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Preliminary Modelling of Plasco Tower Collapse

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Abstract

In a recent tragic fire incident, the Plasco Tower collapsed after an intense outburst of fire lasting for three and a half hours and claiming the lives of 16 firefighters and 6 civilians. This paper will present continuing collaborative work between Hong Kong Polytechnic University and Queen's University in Canada to model the progressive collapse of the tower. The fire started at the 10th floor and was observed to have travelled along the floor horizontally and through the staircase and windows vertically. Plasco Tower was steel structure and all the steel sections were fabricated by welding standard European channel or angle profiles and no fire protection was applied. Four internal columns carried the loads transferred by the primary beams, and box columns were constructed along the perimeter of the building as a braced tube for resisting seismic loading. OpenSees fibre-based sections and displacement-based beam-column elements are used to model the frames, while shell elements are used for the reinforced concrete floor slabs. The thermal properties and elevated temperature mechanical properties are as recommended in the Eurocodes. The results in this preliminarily analysis are based on rough estimations of the structure's configuration. The ongoing work looks at modeling the Plasco Tower based on the most accurate findings from reviewing many photographs and collected data.

Keywords: Progressive collapse, Structures in fire, Plasco Tower, OpenSees simulation

1. Introduction

Constructed in 1962, the Plasco building consisted of a 16-storey tower and a 5-storey podium (Fig. 1). Although the tower was the tallest structure ever built in Iran at the time, it was constructed incorporating some advanced construction methods and elements such as post-tensioned floor trusses, a lateral-load bracing system, and composite concrete floors. Fig. 2(a) shows the flooring trusses in the tower during construction and Fig. 2(b) shows the cross bracing system on the sides of the Plasco tower. Initially, the low-rise podium part of the building and the lower floors of the tower were used as a shopping centre while the upper floors of the tower were occupied as offices. At the time that the fire occurred, however, most of the units in the tower were used as garment manufacturing units.

Fire safety measures were not adopted in the building regardless of the extreme vulnerability of the structural steel components to heat. Neither passive nor active fire safety measures were implemented in the building. There was an ongoing attempt to improve the fire safety of the building, but it was postponed because of financial constraints and the resistance of the tenants to pause their activities during the proposed retrofitting work. Hajiloo et al. (2017) studied various aspects of the Plasco fire including the condition of the tower before the fire, initiation of the fire, the evacuation process, and the progressive collapse of the building. It was shown that the fire could have been extinguished earlier and the catastrophic loss of the firefighters could have been prevented if an expert knowledge of the performance of steel structures was used. The authors have not conducted an independent site investigation, and the work presented in this paper was developed based on the information in the available reports (Hajiloo et al., 2017; Plasco, 2017; Shakib, 2017) photographs, and videos of the fire event.

The Open System for Earthquake Engineering Simulation (OpenSees) was developed at the University of California, Berkeley (Mckenna, 1999). It was initially designed to simulate non-linear response of structural frames under seismic excitations. OpenSees is written in C++ and has an object-oriented architecture, which enables structural engineers to focus on specific objects comprising structural models that have their own attributes and functions rather than just data. Major attributes such as elements, materials, analysis procedures, and solution algorithms are designed as individual objects and they can be added into the framework freely by anyone anywhere. In 2009, OpenSees was adopted at the University of Edinburgh to further develop it to perform structural fire analysis (Usmani et al., 2012). Significant contributions in terms of heat transfer and thermo-mechanical classes have been made to the framework in developing the Thermal version of OpenSees, where a loose coupling method is adopted in order

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(a) (b)

Figure 1. (a) Steel frame and bracing system around all sides of the Plasco tower; (b) the attached 5-storey shopping centre (all images courtesy of IRNA).

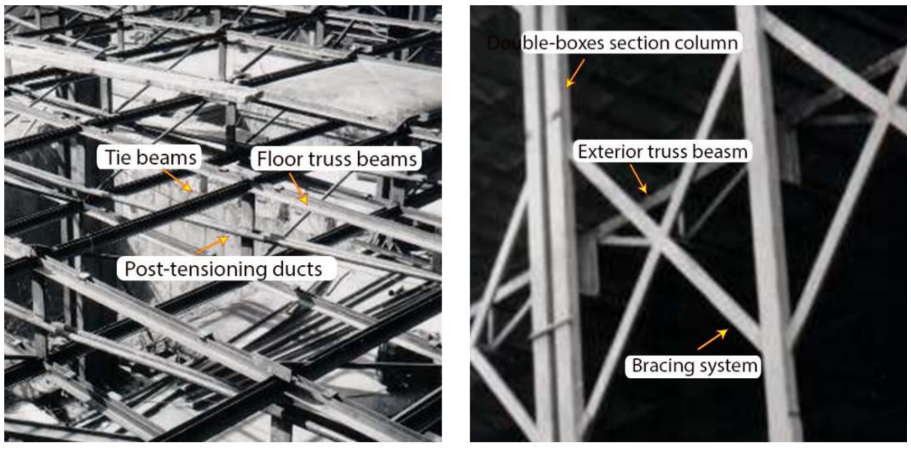
to separate the heat transfer analysis from the thermo-mechanical analysis. For the latter, temperature dependent formulations have been incorporated for frame element and shell elements to account for the thermal effects (Jiang and Usmani, 2013; Jiang et al., 2013), while new temperature dependent material models for steel and concrete based on Eurocodes (CEN, 1993, 2004) are also added to the material library.

2. Engineering Details of the Plasco Tower

2.1. Preliminary Analysis

A preliminary analysis of the performance of the Plasco tower was conducted by Tehran's city council technical committee (Shakib et al., 2017) that was formed to investigate the Plasco fire incident. The committee's analysis was performed with the commercial program SAP2000. First, the tower was evaluated against the gravity load and seismic loads. The tower was adequate for gravity load but failed to meet the requirements of the current seismic demands in Tehran's seismic zone. Then, progressive collapse of the tower was analysed using the "Staged Construction" feature of SAP2000. The procedure proposed by the US Department of Defence to simulate the progressive collapse of structures (DOD, 2016) was adopted. This guideline allowed a linear static analysis because of the regular plan of the tower. The assessment of the collapse process using the visual evidence (Shakib et al., 2017) showed that the most probable cause of the progressive failure was the collapse of one beam on the 12th floor.

When linear static analysis was used, the actions in the structure were classified as deformation-controlled or



(a) (b)

Figure 2. The Plasco tower under construction: (a) the flooring system; (b) the bracing system on the sides of the tower (adapted from (Shakib et al., 2017)).

force-controlled (DOD, 2016; ASCE 41, 2007). The gravity load was applied to the structure using the relevant load cases for deformation-controlled and force-controlled actions. Concurrently, the gravity load on the areas affected by the removed elements was increased by a load increase factor of 2 for force-controlled actions. The procedure assigned each structural component an m factor that was determined from ASCE 41 (2007). Then, the primary elements were checked against the acceptance criteria for force-controlled and deformation-controlled actions.

According to the material tests that were conducted on the coupons taken from the various structural elements, the average tensile yield strength of the steel was 290 MPa. As such, the steel was considered equivalent to ASTM A992. In the analysis, the temperature of the steel structural elements was roughly assumed to be above 600°C and thus the steel was assumed to have lost half of its strength and stiffness. Although it was an unrealistic assumption that the temperatures at all structural elements above the 11th floor would all be the same, the authors (Shakib et al., 2017) claimed it did not influence the results. Their analysis (Shakib et al., 2017) showed little dependence of the progressive collapse on the temperature of the elements. According to the report [3], the impact of the collapsing floor on its lower floor was the most destructive factor. Although the analysis was performed using linear static analysis, the effects of dynamic impacts were taken into account by the load increase factors. The accumulating weight of the upper collapsing floors on the lower floor caused the progressive collapse.

2.2. Gravity (Dead and Live) Loading Details

The gravity dead load consists of the weight of the composite concrete floor (4.30 kN/m^2) and the partitioning walls (1.0 kN/m^2) which are distributed uniformly on the floors. The external walls were heavier, and their weight is applied on the external perimeter beams. The weight of the external walls on the east and west sides was 6.8 kN/m, and 4.5 kN/m on the north and south sides.

Because most of the commercial units in the upper floors were wholesale stores with some manufacturing purposes, it is assumed that they contained a considerable amount of materials. According to ASCE-7 (2005), the storage live load of typical wholesale stores is 6.0 kN/m^2. Although it is typically assumed that only a portion of the live load (e.g., 25%) is present in a building during a fire incident, the whole live load is considered to be present in the Plasco Tower in this analysis because the building was heavily used for storage.

2.3. The fire

Few people were in the Plasco tower when the fire broke out at 7:50 AM local time on the 10th floor in the northwest corner of the tower. There were no fire extinguishers nor sprinklers in the tower, and by the time that the firefighters arrived at the location, the 10th floor was already filled with smoke. Fig. 3(a) shows that the flames were put out at the end of the first attempt at firefighting, but the fire again erupted and spread to the upper floors as shown in Figs. 3(b) and (c).

3. Opensees Modelling

3.1. General Modelling Philosophy and Aims and Scope of the Simulation

The aim of this work is to understand the reasons for the total collapse of the Plasco Tower through subjecting a reasonably comprehensive 3D finite element model of the building to the type of fire loading it experienced during the incident that caused its collapse. The long-term aim is to simulate the progressive collapse of the tower with respect to the travelling fire phenomenon as observed during the fire disaster. The fire started on the 10th



(a) (b) (c)

Figure 3. The extent of fire: (a) at early stages; (b) spreading in the northwest side; (c) extending throughout the entire floor levels (all images courtesy of ISNA NEWS Agency).

floor and was observed to have travelled along the floor horizontally and through the staircase and windows vertically. By comprehensively studying the photos taken from different sides of the building during the fire, various potential trajectories of fire were identified, and will eventually be implemented in the analysis enabled by OpenSees-SiFBuilder (Jiang et al., 2013). The advanced travelling fire models available in the fire library of OpenSees-SiFBuilder will be used to model the fire evolution in the Plasco tower. The horizontal travelling fire behaviour will be adopted based on the model proposed by Dai et al. (2017) and the vertical travel will be represented by time-delays as assumed by Kotsovinos et al. (2013). The SifBuilder tool will orchestrate the analysis sequence, from the fire models which provide heat flux boundary conditions to the structural members. Information from interviews with the fire crew and the residents will be combined with the observations of the extent of the fire in the photos to generate the fire models. This will be followed by a heat transfer analysis that will produce the temperature evolution history for each structural member. The temperature history will then be applied as a load in the thermo-mechanical analysis to simulate the non-linear structural response to the fire. For the preliminary stage of this work, the team has focused on understanding the structural system of the Plasco Building and its capacity for fire resistance. This paper presents the progress of the work achieved so far.

3.2. Structural Details of the Building

The plan of the Plasco tower is shown in Fig. 4. Four internal columns carried the loads transferred by the primary beams and box columns were constructed along the perimeter of the building. All the steel sections were fabricated by welding standard European channel or angle profiles and no fire protection was applied. OpenSees fibre-based sections and displacement-based beam-column elements are used to model the frames, while shell elements are used for concrete slabs. The material properties of the steel at room temperature were taken from experiments performed on the coupons gained from the crumbled structural elements (Shakib et al., 2017).

3.3. Structural Model of the Floor System (Preliminary Assumption)

In the modelling of Plasco tower, a non-composite grillage floor system was assumed where the primary trusses are simply supported and the secondary floor beams were assumed continuous over the primary beams. The information on the conditions and the structural configuration of the Plasco tower is not quite adequate for modelling the Plasco tower with absolute certainty (Hajiloo et al., 2017). Limited information is available on the connections of the trusses to the columns. The beam-column connections are recommended to be considered semi-rigid in the report (Plasco, 2017), and as a start, the beams were considered continuous. Fig. 5 – a shows the OpenSees floor model based on a preliminary interpretation of photos from the construction phase of the building (e.g., Fig. 2). The floor system is essentially a grillage structure. The East-West direction in the plan (Fig. 4) is spanned by a system of secondary tie trusses (Fig. 5(h)) and

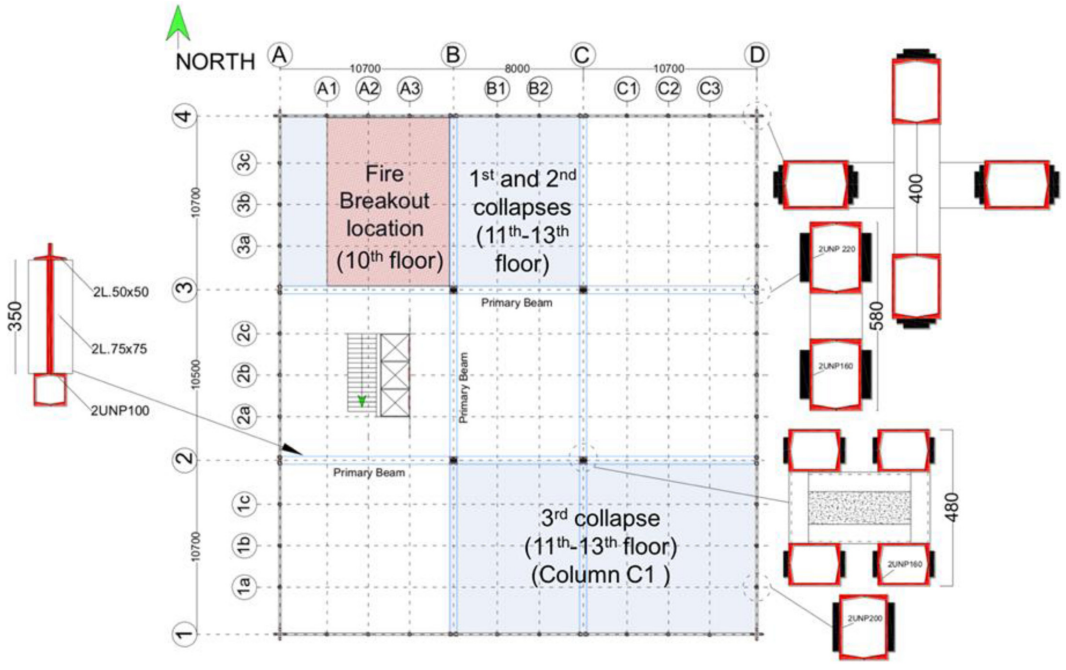


Figure 4. The plan view of Plasco tower and collapse locations (Hajiloo et al., 2017).

the North-South direction is spanned by a system of Vierendeel trusses (Fig. 5(g)), which are also post-tensioned. Primary beams are running over the central columns (B-2, B-3, C-2, C-3, refer Fig. 4). All truss systems are 400 mm deep. The post-tensioning force for the Vierendeel trusses is also unknown at this stage.

3.4. Modelling of the Floor System Assuming Composite Slab

Our preliminary assumption was based on a non-composite floor slab, however, a recent paper (Darvan et al., 2018) suggested that the slab may have been composite to the grillage. Based on this information further analyses have been carried out and reported in subsection 4.2.

3.5. Fire Load Modelling

For the preliminary modelling in this paper, temperatures up to a maximum of 800°C are applied uniformly to all the grillage members in small increments, excluding the concrete slab (which is not explicitly modelled). No heat transfer analysis is carried out at this stage.

A 10-storey multiple floor model is then constructed from the 5th to the 14th floor and subjected to single, double, and triple floor simultaneous fires (subjecting all grillage members to temperatures up to 800°C). The columns are not subjected to fire at this stage. This analysis provides the upper bound for potential failure of the whole structure with the collapse of columns as indicated in Lange et al. (2012). Subsequent investigations will consider realistic fire scenarios based on forensic evidence as mentioned earlier.

4. Modelling Results

4.1. Behaviour of the Building under a Single Floor Fire (Assuming Non-Composite Slab)

Fig. 6 shows the deformed shape (magnified as mentioned) of the EW and NS frames (10-storey model of the Plasco Building) at 350°C and 750°C. Both EW and NS trusses are pushing the columns out (Figs. 6(e), (f)). This behaviour is usually expected in the early stages when thermally induced expansion is dominant. As the heating progresses the trusses begin to pull the columns back in (Figs. 6(g), (h)). This behaviour is due to the softening of floors and when downward deflection dominates the initial expansion due to the self-weight and live loads. As this phenomenon cannot be observed clearly from Fig. 6 due scale issues, Figs. 8 & 9 show the plots of the horizontal displacement of the perimeter columns and the downward deflections of the floor.

Figs. 8(a) & (b) shows the plot of deformations of the columns along the NS direction (grid line – D) and columns along EW (grid line – 1) respectively at the floor level where the fire loading is applied. Figs. 9(a) & (b) shows the downward deflections of the floor and the primary beams respectively for the fire affected floor. The horizontal displacements of the columns and the vertical displacements of the floor and the floor beams are recorded at the corresponding reference points shown in Fig. 7.

Figs. 8 & 9 also shows the comparison between the static and dynamic analysis. The material properties in the static analysis case are considered elastic but temperature dependent, while non-linear temperature dependent material properties are used in the dynamic analysis case. It may be noticed that the static and dynamic analysis results are good in agreement until the temperature reaches 200°C since the material properties are roughly the same until this temperature point. However, as the heating progresses the material softens and shows higher displacements in case of the dynamic analysis.

In Fig. 8 the outward horizontal displacement of columns continues until the temperature reaches 400°C for columns along gridline D and 500°C for columns along gridline 1 (because these columns are pushed by the longer span of the floor). Rapid reduction in the stiffness and strength of the material occurs beyond 400°C leads to the softening of floors that reduces the outward push force caused by thermal expansion and the columns begin to pull inwards. Downward deflection of the floor peaks at approximately 700 mm when the fire temperature is between 600°C to 700°C. The beam deflections peak at approximately 500 mm at a similar stage. The analysis shows a slight recovery in downward deflection of both the floors and beams. This is essentially due to the same reasons that the perimeter columns "pull inwards", i.e., the reduction of the forces causes by restrained thermal expansion (against the perimeter columns) because of the softening of steel.

With further heating the floor may begin to deflect downwards again as it will not remain stiff enough to resist the gravity loading in the bending mechanism only and will begin to deploy its tensile membrane capacity. This will also lead to further pull in of the column, which may eventually destabilise the column and precipitate collapse. This issue has not been investigated in the current stage of this work, however we feel it is unlikely that a single floor fire will be sufficient to initiate global progressive collapse in the structure. Further investigation is needed to determine potential collapse mechanisms from simultaneous fires on two or three floors.

Nomenclature: MF, Magnification factor; St, static; Dy, dynamic; CF, Composite Floor; T1, thermal loading at 5^{th} floor; T3, thermal loading at 4^{th}, 5^{th} & 6^{th} floors.

4.2. Behaviour under Single and Multiple Floor Fire Assuming Composite Floor Slab

Fig. 10 shows the horizontal displacements at the 5th floor columns from two dynamic analyses. The former is based on fire loading applied to the 5^{th} floor and the latter is based on fire loading applied simultaneously to the 4^{th}, 5^{th} and 6^{th} floor. For both analyses shell elements are used to model the a 100 mm thick concrete floor slab made

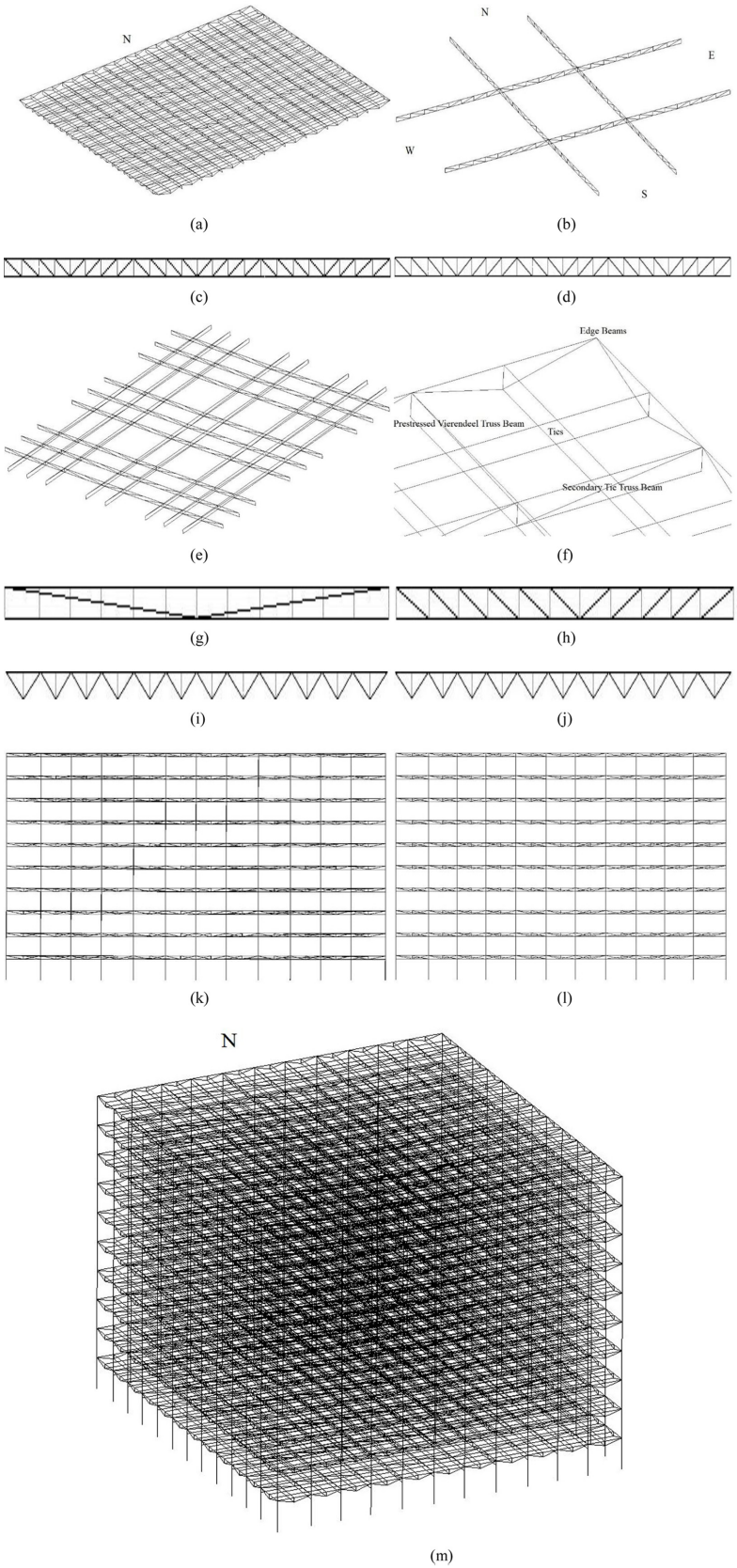


Figure 5. (a) Full Floor; (b) Primary EW and NS Truss; (c) Primary NS Truss; (d) Primary EW Truss; (e) Secondary Beam EW and NS Truss; (g) Secondary Prestressed Vierendeel Truss (NS); (h) Secondary Tie Truss; (i) Edge Beam (NS); (j) Edge Beam (EW); (k) Multi Floor (NS); (l) Multi Floor (EW); (m) Multi Floor Model.

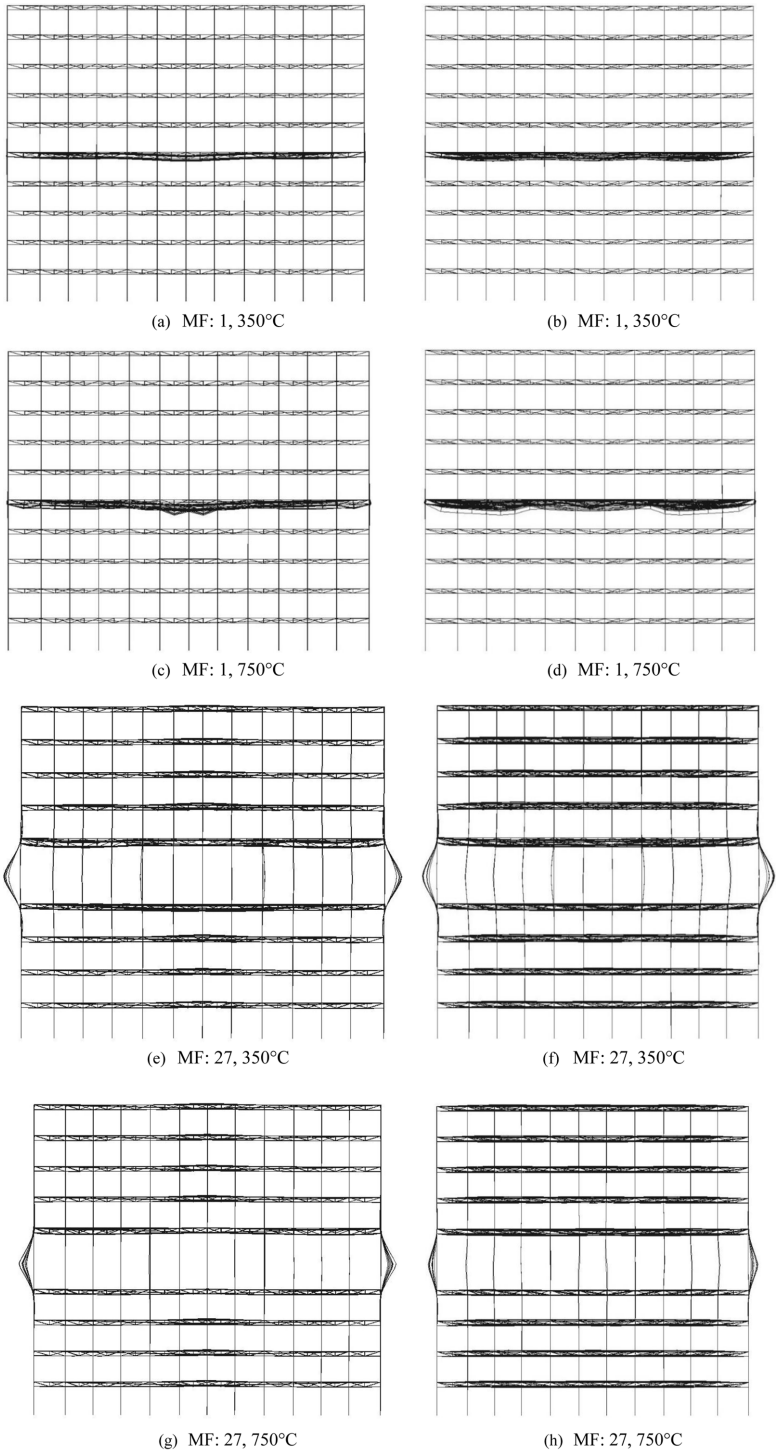


Figure 6. 10-storey model with middle floor heated (MF, Magnification Factor); (a), (c), (e), (g), NS (grid line – D); (b), (d), (f), (h), EW (grid line – 1).

composite with the top members of the steel grillage. The slab is not subjected to any fire load in these preliminary analyses, while all the steel grillage members are heated to a temperature of 625°C.

It can be seen from Fig. 10 that the floors displace more when the fire loading is applied simultaneously to three floors compared to when the fire loading is applied only to one floor. These sets of analyses show that the reversal of the horizontal outward displacements of the columns occurs at a higher temperature (600°C) for the single floor fire loading case, compared to the non-composite floor analysis discussed in the previous section. No reversal is

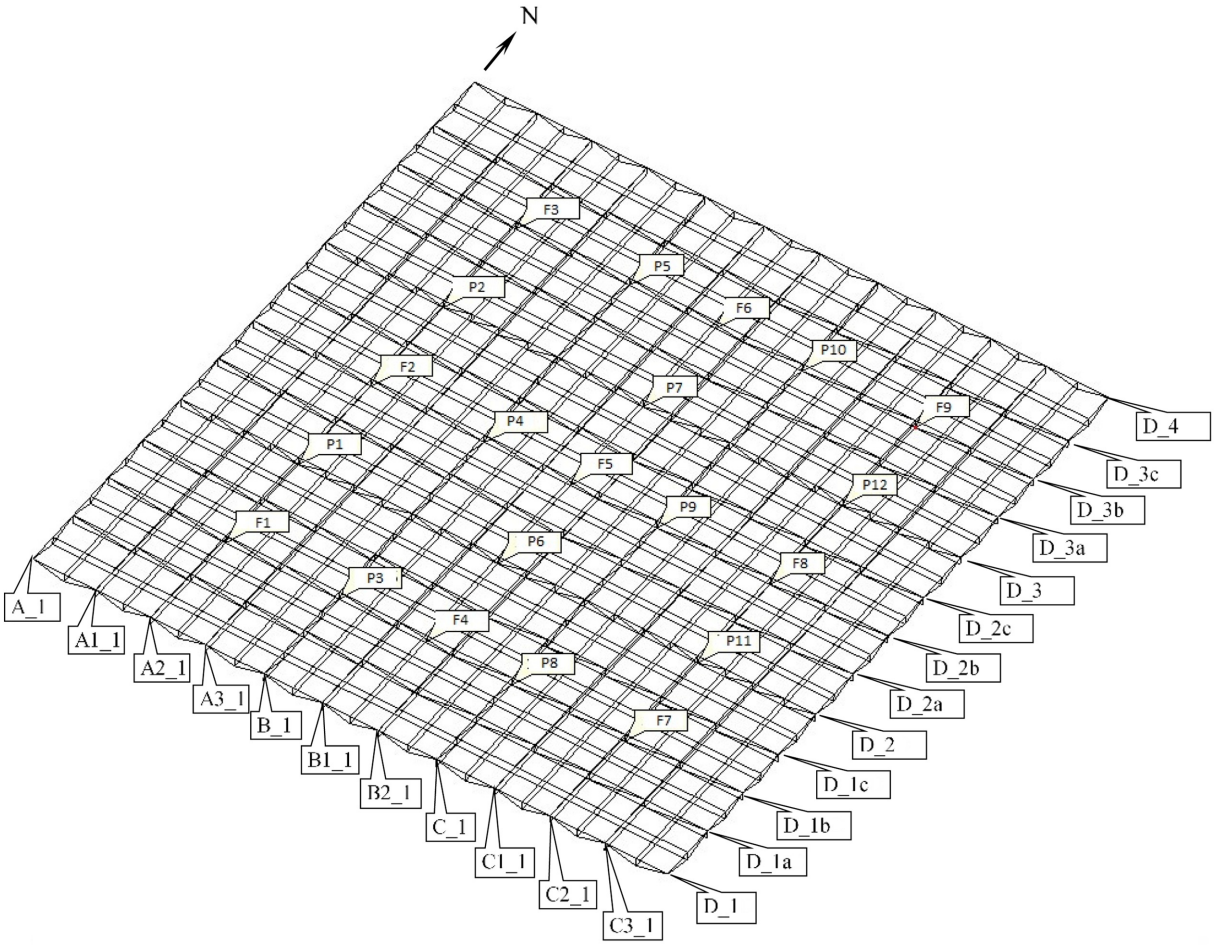
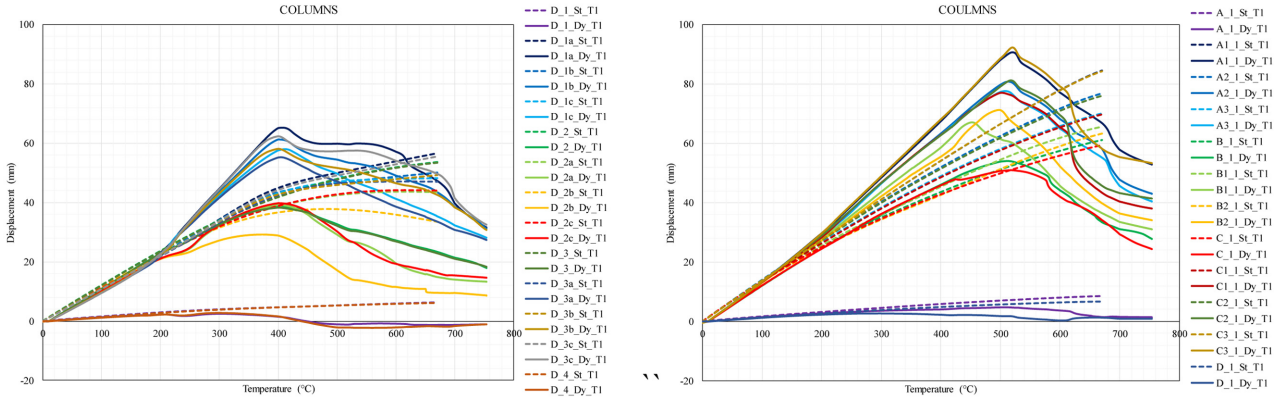


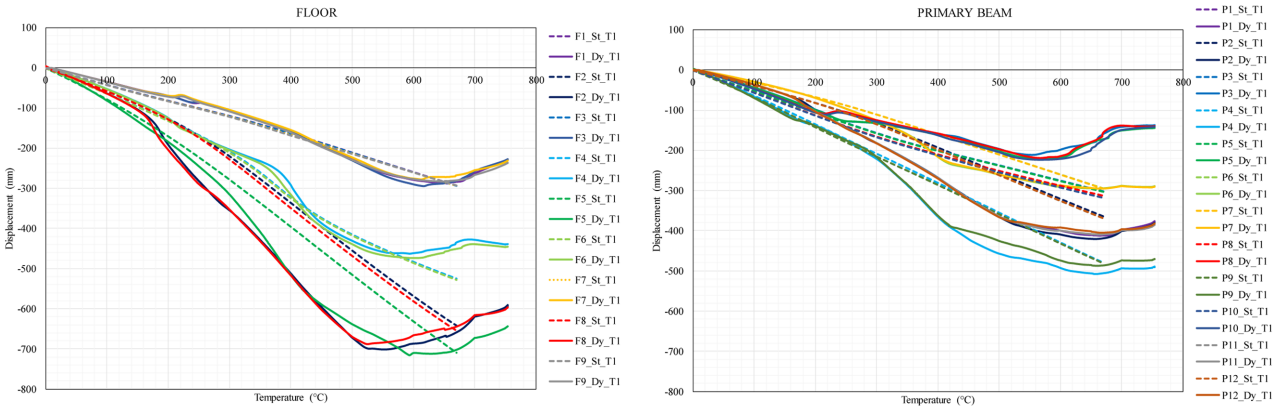
Figure 7. Reference points in the floor for horizontal and vertical deflections.



(a) Columns along grid line – D(NS) (b) Columns along grid line – 1(EW)

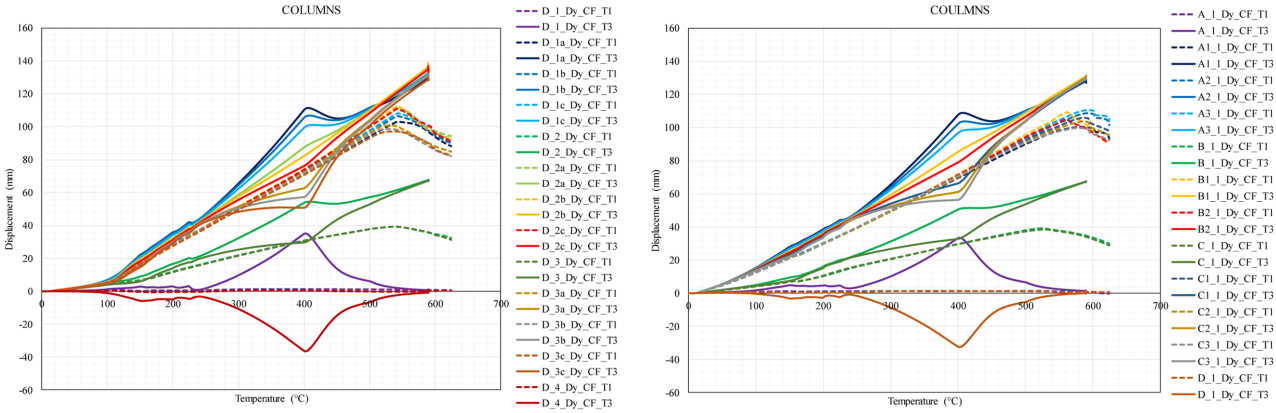
Figure 8. Horizontal displacement of columns at 5th floor.

seen for the three-floor fire loading case. This is counter-intuitive as composite floors should induce greater thermal bowing in the floor (because of the cold slab and hot grillage) and cause a reversal of the outward displacements earlier than the non-composite case. This issue will be investigated further in future work. The corner columns suggests an asymmetric deformation pattern of the composite floor system for the three-floor fire loading case. Fig. 11 shows the floor downward deflections for the composite floor slab and the primary beams of the grillage



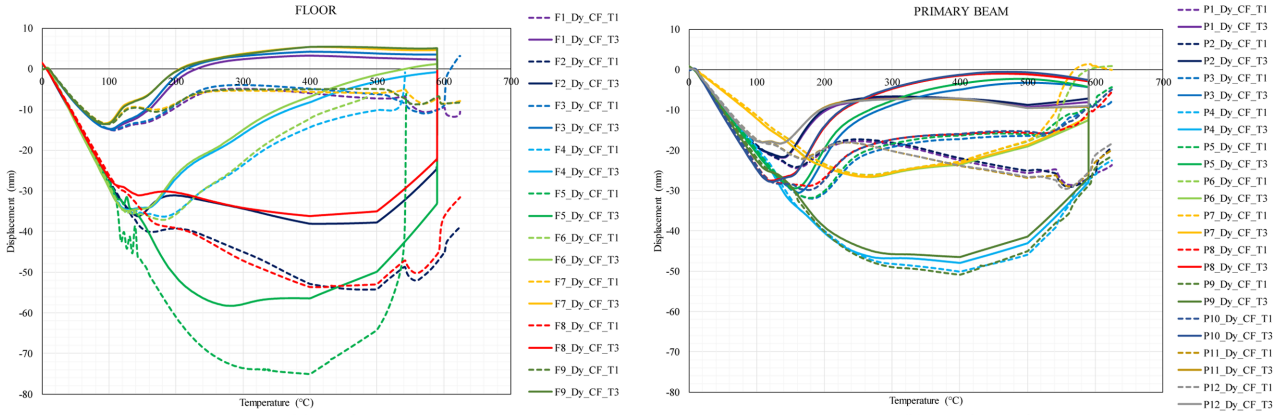
(a) Floor (b) Primary beam

Figure 9. Vertical displacement of floor at 5th floor.



(a) Columns along grid line – D(NS) (b) Columns along grid line – 1(EW)

Figure 10. Horizontal displacement of columns at 5th floor (single floor fire vs three floor fire).



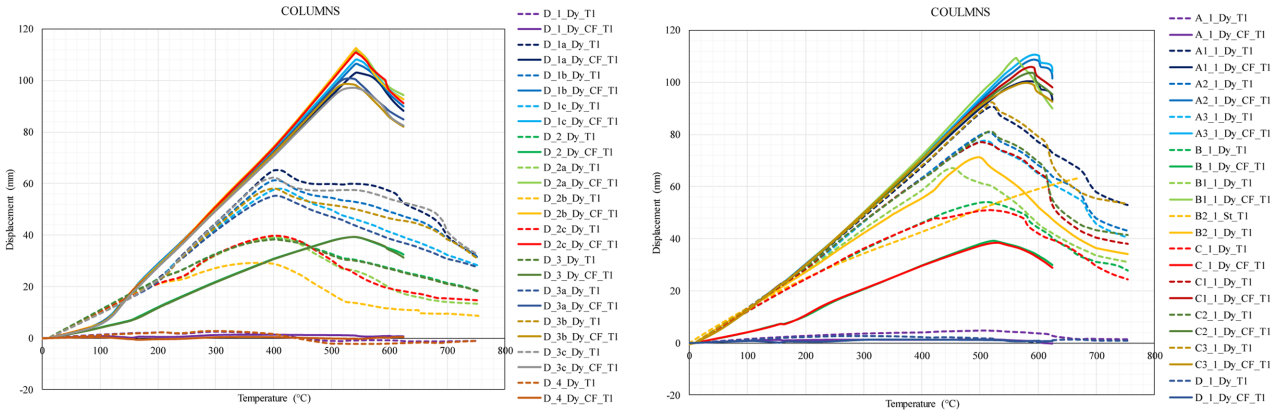
(a) Floor (b) Primary beam

Figure 11. Vertical displacement of floor at 5th floor (single floor fire vs three floor fire).

floor system. These deflections are an order of magnitude less than those of the non-composite floor (shown in Fig. 9). Fig. 11 also shows a sudden recovery of the floor deflection at the end of the analysis (at 625°C), when the analysis aborts, possibly due to the sudden rebound.

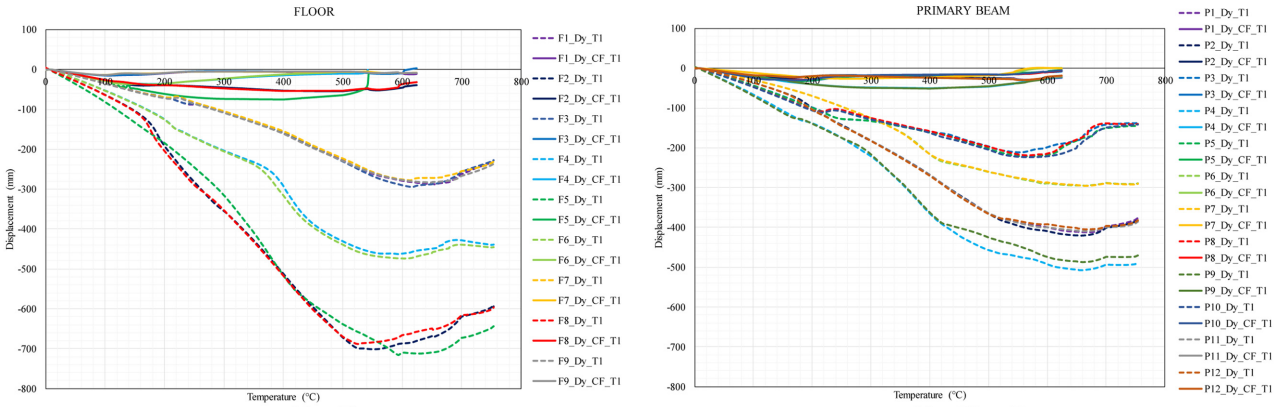
4.3. Non – Composite *vs* Composite

Fig. 12 shows the comparison between the horizontal



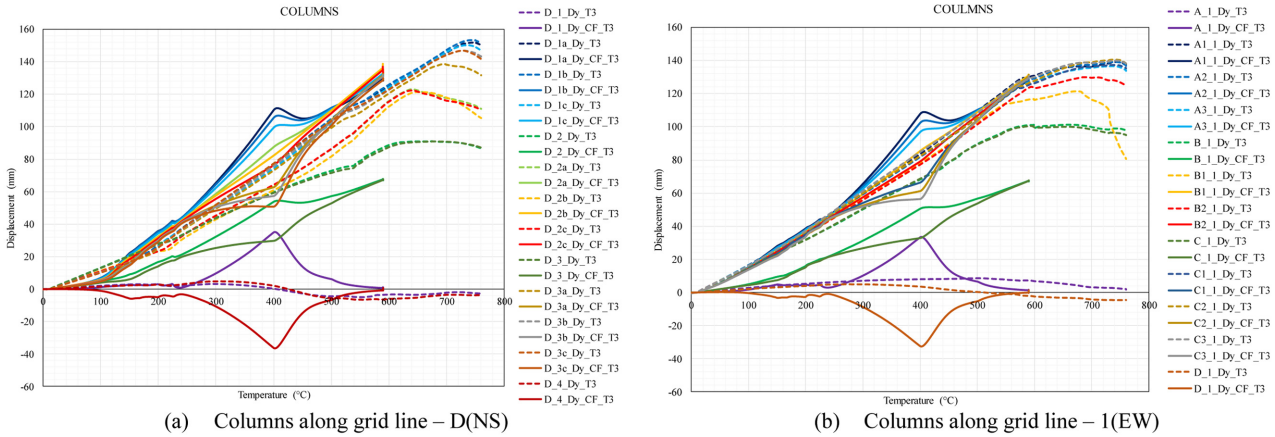
(a) Columns along grid line – D(NS) (b) Columns along grid line – 1(EW)

Figure 12. Horizontal displacement of columns at mid-floor height (Thermal loading at 5th floor).



(a) Floor (b) Floor Beam

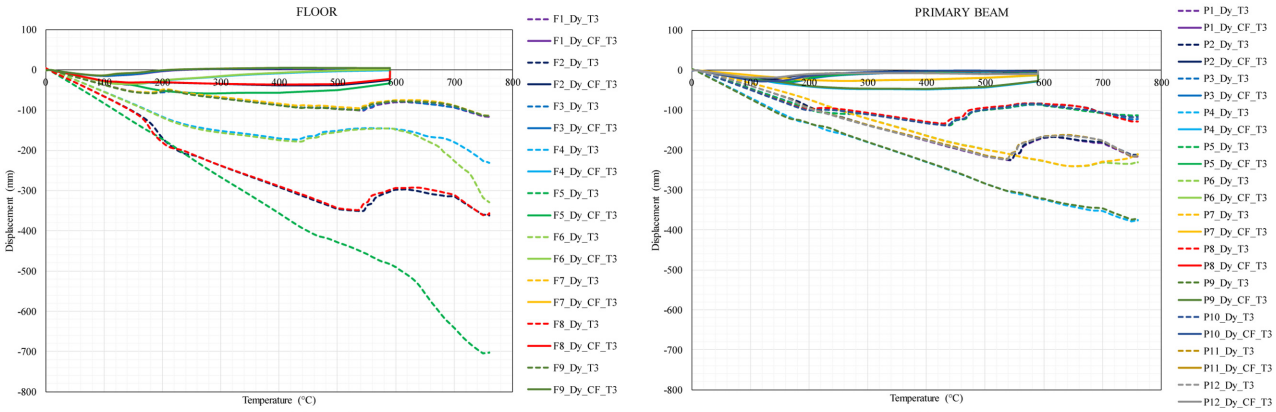
Figure 13. Vertical displacement of floor at 5th floor (Thermal loading at 5th floor).



(a) Columns along grid line – D(NS) (b) Columns along grid line – 1(EW)

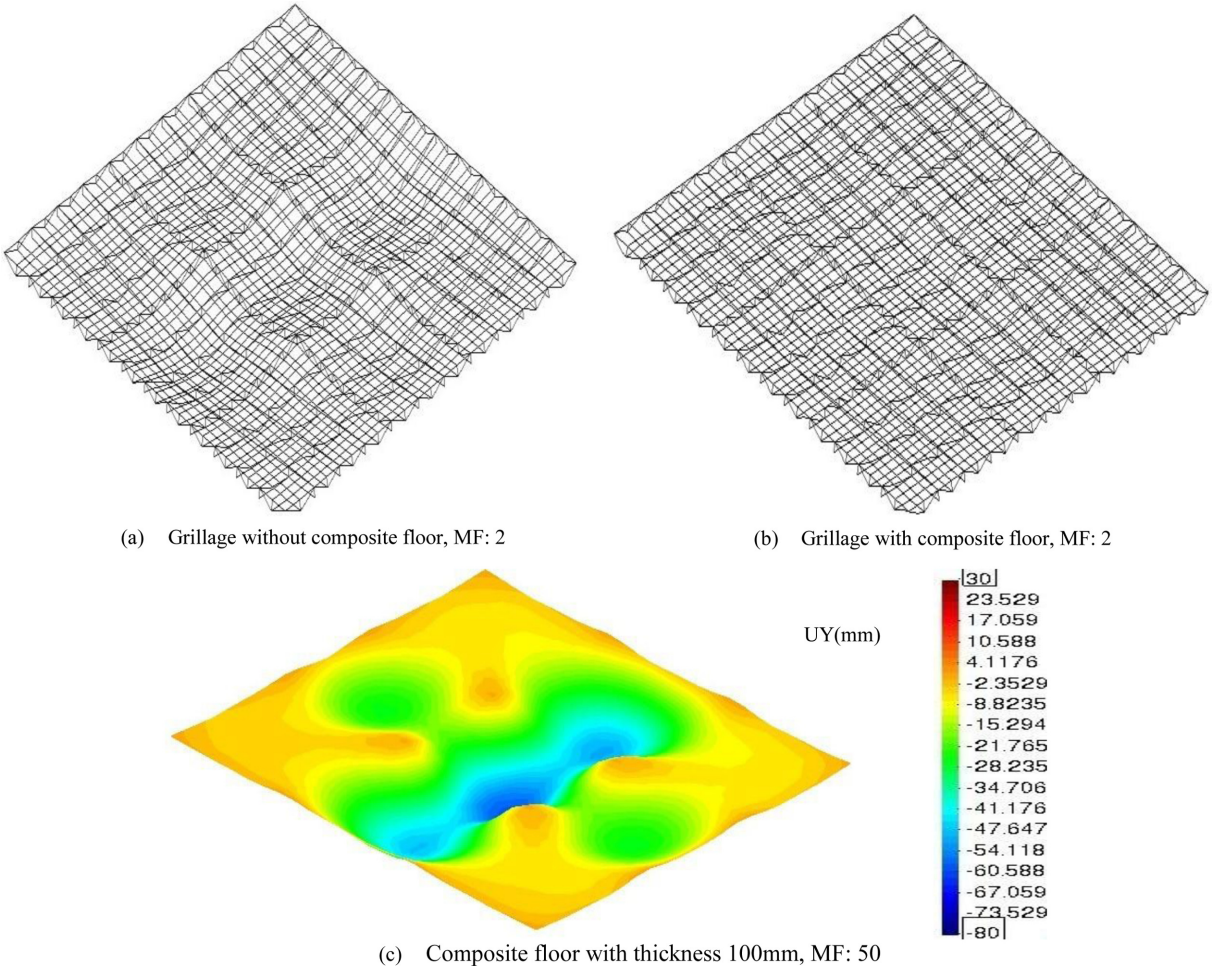
Figure 14. Horizontal displacement of columns at 5th floor (Thermal loading at 4th, 5th & 6th floors).

displacements of the composite and non-composite floor systems obtained for the single floor fire loading case. As noted earlier the horizontal displacement of the perimeter of the composite floor is greater than that of the non-composite floor. Fig. 13 provides direct comparison between the magnitude of the floor downward deflections for the composite and non-composite floors, with former showing much lower deflection.



(a) Floor (b) Floor Beam

Figure 15. Vertical displacement of floor at 5th floor (Thermal loading at 4th, 5th & 6th floors).



(a) Grillage without composite floor, MF: 2 (b) Grillage with composite floor, MF: 2

(c) Composite floor with thickness 100mm, MF: 50

Figure 16. Floor Deformations at 5th floor, 350°C (Thermal Loading at 5th floor).

Fig. 14 shows the horizontal displacements of the 5th floor for composite and non-composite floor systems for the three-floor fire loading case. The non-composite floor analyses are able to run to higher temperatures that the composite floor analyses, otherwise their magnitudes are broadly similar. Furthermore, the non-composite floor fire does not show any asymmetry in the floor deformation unlike what the plots from Fig. 14 show for the composite

floor. Fig. 15 shows the downward deflections of the composite and non-composite floors for the three-floor fire case, which shows much lower deflections for the composite floor. Fig. 16 provides the final deflected shapes of the composite and non-composite 5th floor system.

5. Conclusions

The authors believe that it is extremely important for structural engineers to analyze major failures with great care and dedication, such as is routine in the aerospace industry. This will help the profession learn from these failures and help improve the robustness and resilience of buildings and urban infrastructure that may be similarly vulnerable through appropriate strengthening and retrofit. While this particular analysis is not conclusive at this initial stage of investigation, the authors feel that the structural system of the Plasco Tower has considerable similarity to that of the WTC Towers, specifically the two floor systems, and collapse mechanisms described by Lange et al. (2012) could very well be responsible for the collapse. However, so far this has not been found and significant further studies are required.

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