Preliminary Stage OpenSEES Simulation of the Collapse of Plasco Tower in Fire

Ramakanth Domada, Tejeswar Yarlagadda, Liming Jiang, and Asif Usmani

Abstract The Plasco tower, built in 1962, was the tallest building with 17 storeys in Iran at the time of its construction and was considered as an iconic high-rise dominating the Tehran skyline. In January 2017, a fire started on the 10th floor which eventually led to its collapse and caused many deaths and injuries. The building was used as a residential and commercial building, with a major shopping centre on its ground floor, a restaurant on its upper floor, and several clothing workshops. It was a steel structure with built-up sections fabricated using standard European channels and angles without any fire protection. The tower had four strong core columns to transfer the load from primary beams to the foundation and relatively closely spaced interconnected columns along the periphery. This exterior framing is designed to be sufficiently strong to resist all lateral loads on the tower, thereby allowing the interior of the tower to be simply framed for gravity loads. For modelling, OpenSEES fibre-based sections and displacement-based beam-column elements are used. The thermal properties and elevated temperature mechanical properties are as recommended in the Eurocodes. As documented, the fire started at the 10th floor and then involved stories 11–14 as a result of a horizontally and vertically spreading fire. The thermo-mechanical analyses are performed assuming no variation of temperature across the thin sections. Based on the best available information, the floor in plan is believed to be structurally divided into nine individual blocks by two centrally running primary truss beams in both directions. This leads to an understanding that each block is structurally isolated except at the peripheral beams and central core columns, however, if the reinforced concrete floor slab is composite with steel beams of the floor system, this will not be the case. This paper presents the structural response of the tower over a single floor as a preliminary analysis.

Keywords Plasco tower · Steel building · OpenSEES for fire · Thermo-mechanical analysis · Fire-induced collapse

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1 Introduction

The Plasco tower was a 16-storey high-rise steel structure dominating the Tehran skyline after its construction during 1960s. Until its collapse, it was the tallest building in Tehran, the capital city of Iran. On 19th January 2017, a fire accident occurred on the 10th floor which eventually resulted in the collapse of the building after four hours. The accident resulted in loss of many lives and billions of dollars in economic losses. It was reported that the fire was caused by a short circuit in the electric system. For decades, it remained famous for being the tallest high-rise structure in Iran and dominated Tehran's skyline. Because of that, the fire and its consequence drew lot of attention.

The Plasco building was designed as braced steel frame where many light columns are placed peripherally and sturdier columns at the core of the tube. The light exterior columns are intended to resist the lateral loads, whereas the central strong columns to safely transfer the gravity load of each floor to the foundation. The Plasco building was never designed to withstand fire. There was no sprinkler system, no fire alarm system and no fire protection on the steel frame whatsoever. Unlike the WTC towers, only limited structural drawings are available and most of the information obtained was by post-collapse inspection. Therefore, there hasn't been much research which focused on what exactly caused the collapse.

The overall aim of this paper is to carry out preliminary structural analysis of Plasco tower under fire during initial stage of fire using the Open System for Earthquake Engineering Simulation (OpenSEES) which is a software framework for simulating the seismic response of structural and geotechnical systems. The fire module in OpenSEES has been developed by Jiang et al. [2–4] and his students over past 10 years at the University of Edinburgh, UK and this work is being carried forward by his current students at PolyU.

Accidents offer structural engineers an opportunity to learn and improve. The Broadgate Phase 8 fire which occurred in 1999 is a modern steel fast track building which incorporated steel deck/concrete floor construction [6]. There was no collapse, but the examination of damage provided an opportunity to consider the validity of design codes which had been developed from small-scale fire test data. This eventually led BRE and erstwhile British Steel to carry out six full-scale fire tests on an eight-storey steel frame composite structure at the BRE Large Building Test Facility at Cardington (Bedfordshire, UK). The collapse of the WTC Buildings in 2001 following a terrorist attack also offered valuable lessons about adequate preparedness with fire safety and structural stability in fire. Initial attempts to analyze the WTC collapse indicated that, although there had been considerable progress, there were too many counterintuitive and subtle phenomena in the thermo-mechanical response of large frame structures to fires which were not well-understood, even by experts in the profession. Therefore, multiple explanations for WTC collapse surfaced. Kotsovinos and Usmani [5], Usmani et al. [9] and his research team at the University of Edinburgh found that the WTC towers had an unusual vulnerability to large fires. The team was able to produce a credible scenario of collapse, which did not depend upon any gross assumptions about the fire or failure of connections or even structural damage. A clean stability failure mechanism was evident from a simple computational analysis. Not only this, the analysis was entirely consistent with the fundamental principles developed previously during the simulation of Cardington tests led by Usmani et al. [7].

Greater understanding and knowledge of structural frame performance in fire can only be gained by rigorous analysis, just as it is customary to determine the response of structures to earthquake or wind loading.

2 Plasco Tower Description

The Plasco tower, which was located at a densely populated area, was initially intended to be a light commercial centre but eventually transformed into a major clothing distribution centre at the time of its collapse. The building had never been designed according to fire safety regulations; and the change of usage of the building had significantly increased the fire load. It had no sprinkler system, alarm and not even an emergency evacuation plan. The electrical wiring was outdated. The recommendations of Tehran Fire Safety Department (TFSD) were completely ignored.

The building had two separate structural blocks—the main tower which is 17-storeyed standing besides a five-storeyed building. The five storey building has 105 m long in along the North–South direction and has approximately 3200 m^2 floor area. The 17-storey building was 30 × 30 m in plan with two floors below the ground level with height of the ground floor as 6.3 m and other floors being 3.8 m tall.

The light columns were put along the periphery to resist lateral loads like wind and seismic, and the central bulky columns to safely transfer the gravity loads to the foundation. The peripheral columns had been connected via radial foundation. The four central core columns had been raised up to ground level using strong pedestals. The perimeter columns on the south side had not been continued to the foundation level, in order to give free access to give adjoining five-storey part of the structure.

As the structural drawings and other information, had been lost during the fire accident, many field trips were needed to establish the dimensions of all structural members.

All the structural members were welded built-up sections, meaning that they had been made by adding some light steel profiles such as U- and L-shaped profiles. The flooring system contained a concrete slab with a thickness of 120 mm, which was rigidly connected to a series of ceiling trusses placed over both perpendicular directions. The details about the structure have been discussed in detail by Behnam [1].

2.1 Fire Description

It was reported by TFSD that the fire started on the 10th storey of the building at northwest corner. Then the fire had travelled to the 11th floor and eventually involved 11–14 storeys.

In this work, the behaviour of the structure during the initial stages of fire spread has been considered. From the visual evidence, it has been observed that fire which started at 7:50 am in a single block had spread to many other blocks in the same floor by 8:10 am. The more information about the fire accident and post-inspection is available in Tehran municipality report and Plasco national report.

Four fire loads for the analysis are considered which involve one, three and four blocks and the entire floor as shown in Fig. 1.

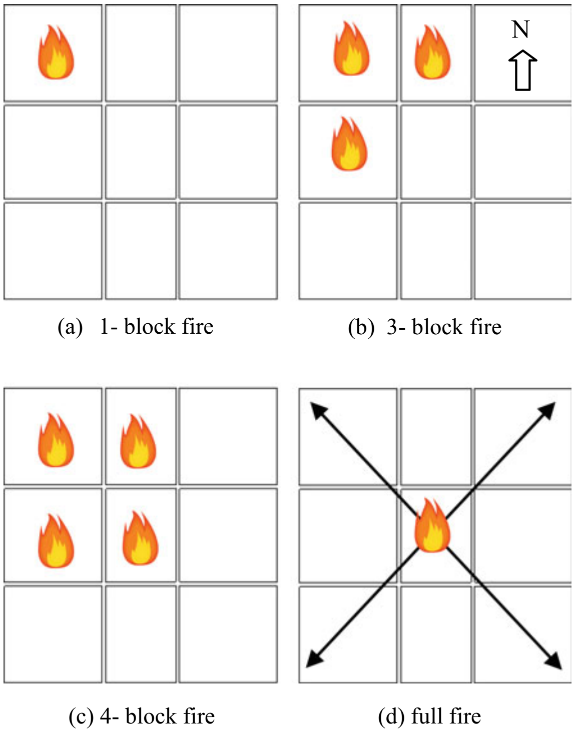

Fig. 1 Considered fire load scenarios with block-wise extents (secondary beams are hidden in these representative pictures)

3 Model Description

The GiD preprocessor has been used to model the Plasco tower. To reduce the size of the problem, only the top 10 floors of the 17 floors have been considered in the FE model.

The Plasco tower floor plan can be seen as nine individual blocks separated by two primary beams running in both the directions. The only structural continuity among the blocks was due to the ceiling trusses spanning in both NS and EW directions.

Trusses had been provided along the EW directions and Vierendeel beams along the NS direction. The plan looks almost square except that the distance in the NS direction is slightly longer than the EW directions (Fig. 2).

The fibre section approach has been followed in the modelling of sections. Each fibre section object is composed of fibres with thermal properties, with each fibre containing a uniaxial thermal material, an area and a location (y, z). The *fibreSec-Thermal* object class enables the user to apply thermal load at 2, 5 or 9 points across the depth of the section. Since the thickness of member sections used in the Plasco tower were very thin, it has been assumed that there is no temperature variation across the depth of sections; in other words, the temperature gradient is taken as zero.

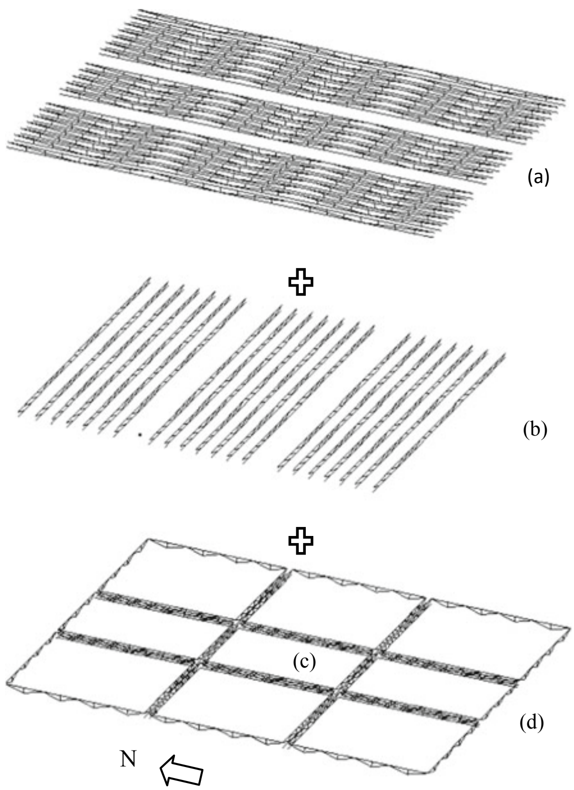

Fig. 2 Plasco tower floor assembly is made of **a** and **b** Secondary truss beams running in both directions, **c** Primary beams, **d** Edge beams

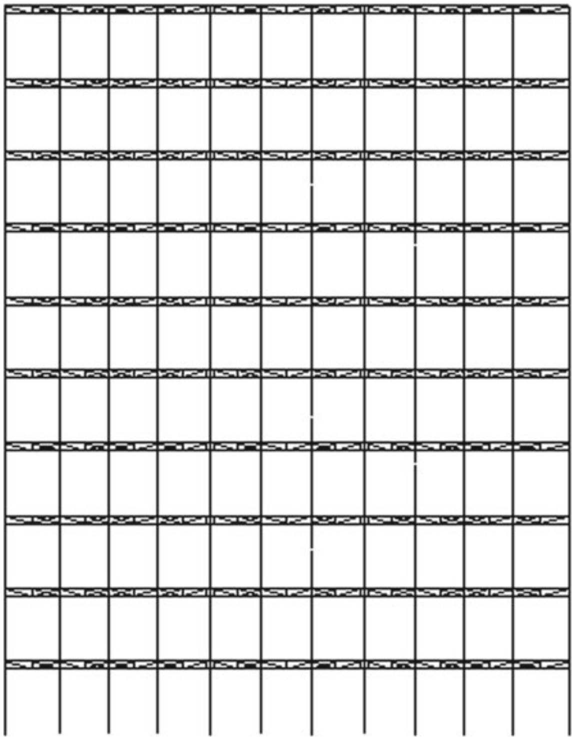

Fig. 3 Finite element model of Plasco tower visualized in OpenSEES GiD preprocessor

A UDL of 20 kN/m representing dead load, live load and super imposed dead load has been applied on all top cords of ceiling trusses. The element type of *dispBeamColumnThermal* has been used in this analysis which is a modification of existing *dispBeamColumn* element to accommodate thermal properties (Fig. 3).

4 Analysis Parameters

The following settings have been used in OpenSEES analysis. The dead load has been applied in 10 load increments whereas temperature load in 200 steps, i.e. 4 °C in each load increment.

Dead load and thermo-mechanical	
`system`	`UmfPack`
`numbered`	`Plain`
`constraints`	`Plain`
`integrator`	`LoadControl`

(continued)

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Dead load and thermo-mechanical	
`test`	`NormDispIncr`
`algorithm`	`Newton`
`analysis`	`Static`

Element sizes varying from 100 to 300 mm are used for meshing the truss members and 750 mm for columns. The fully meshed OpenSEES model has 199,410 elements with 172,270 nodes.

4.1 Computational Information

The analyses have been performed on a Windows machine with processor Intel® Xeon® W-2155 @3.30 GHz (base speed) equipped with 64 GB RAM.

Analysis time	2.5 h
RAM usage	30 GB
Clock speed	3.30 GHz (81% of maximum overclock speed of 4.1 GHz)
OpenSEES.exe output	7.8 GB
GiD postprocessor	22.6 GB of cached RAM
Post-processing time	0.67 h

5 Results

Observations have been made at the following locations in the floor and locations as shown in Fig. 4.

Ce1, Ce2 represent locations on the less stiff perimeter columns and C1, C2 are for stiffer columns which support primary beams. Beam mid-point deflections under fire have been observed at the locations A, B, C and D. Points A and B are on beams with equal length but they support different secondary trusses. C and D are on interior beams.

As the fire accident happened on the 10th floor, the fire load has been applied to the 11th floor. The analyses have been carried out to simulate the behaviour of the structure under fire during the first 20 min. The fire which broke out in the north-west corner block of the 10th floor had spread to blocks within 20 min before spreading to the 11th floor. The extent of fire during this period is not known, hence three different fire load scenarios have been created to represent the initial stage fire as shown in Fig. 1. A fourth scenario with fire load on the entire floor which is used to represent the worst case.

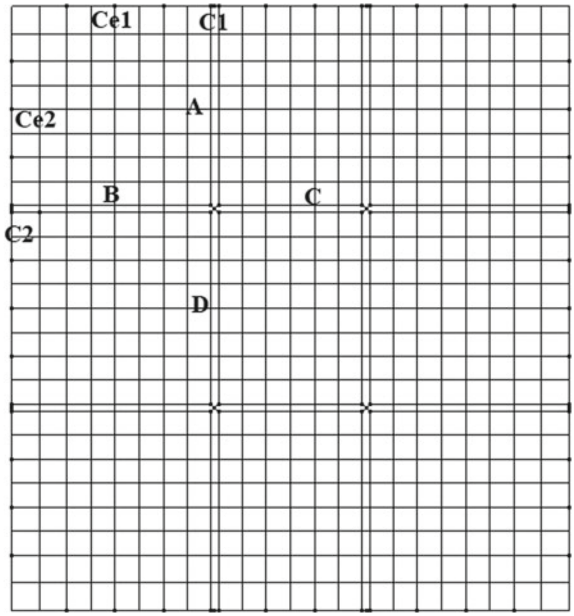

Fig. 4 Points of observation in OpenSEES model (plan)

Figure 5 shows the floor deformations under the fire load scenarios b and c (see Fig. 1).

In the EW direction, the lengths of primary beams longer in comparison with the NS direction, hence the deflections in beams along the EW direction are higher.

In the blocks under fire, the in-plane deformation near the edge is not uniform because of presence of alternate perimeter columns. Because of that, the grid of secondary beams in the top view appeared like waves passing in both the directions.

Figure 6 depicts the interaction of expanding floor with stiff columns and less stiff perimeter columns.

The results in Fig. 7 show push-out deformations of columns C1, Ce1, C2 and Ce2 on 11th under the four considered fire scenarios.

The results in Fig. 8 show the beam deflections in primary beams at locations A, B, C, D under the four considered fire scenarios.

The plot in Fig. 9 highlights results of the analysis carried out assuming three floor fire on north side of the structure. The two graphs show (a) the in-floor deformations in case of three floor fire (b) column outwards deformations.

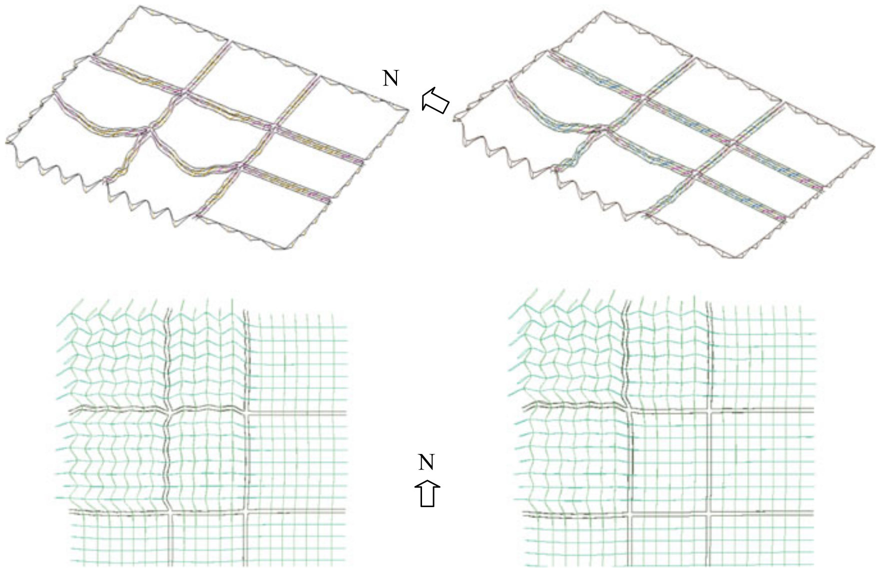

Fig. 5 Floor deflections under the 3-block and 4-block fire scenarios in perspective (secondary beams hides) and in the plan view

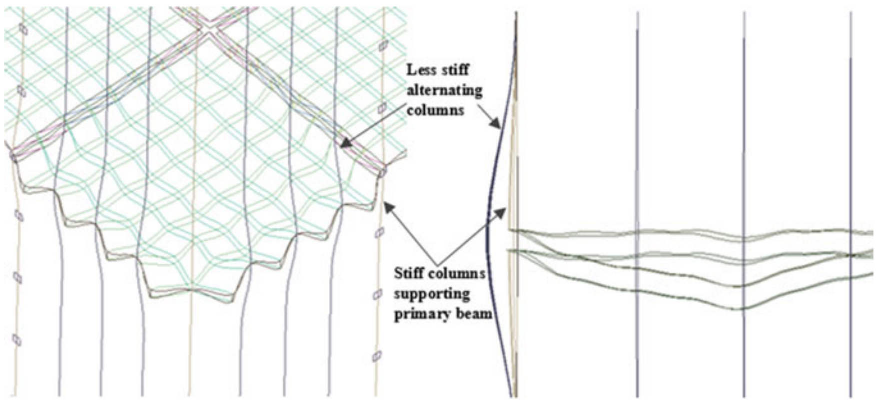

Fig. 6 Visualization of columns getting pushed by the expanding floor

The development of compressive forces in the top chord members of primary beam trusses at locations A and B during the 1-block fire and 4-block fire scenarios are shown in Fig. 10. As location 'B' is on the beam whose supporting column is relatively stiffer, higher compressive forces are observed in the beam because of the higher lateral restraint. The extent of fire is also a factor influencing the forces, because lateral displacement restraints are higher in the 1-block fire than the 4-block fire.

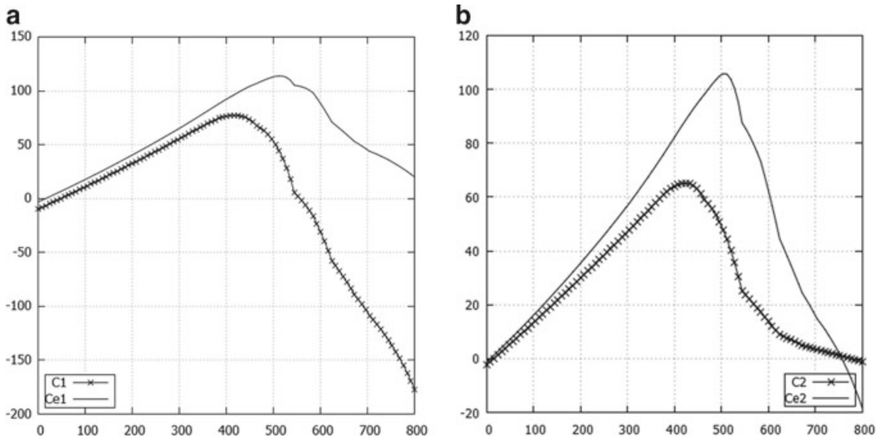

Fig. 7 Temperature versus horizontal displacement plot for column **a** C1, Ce1 and **b** C2, Ce2

6 Conclusions

(a) The fire zone involving 3 or 4 blocks of the plan is enough to cause a similar magnitude of floor deflections as caused by the full floor fire.
(b) The behaviour of the structure is very sensitive to fire load in the core region.
(c) The ceiling trusses are slightly longer along the NS than the EW direction, hence the horizontal displacement of perimeter columns because of thermal expansion of floor is higher in NS direction.
(d) When three blocks are under fire, the relative lateral column deflections are highest along the EW direction. See Fig. 10a. After 700 °C, the difference between C2 and Ce2 plots is greater in comparison with C1 and Ce1.
(e) The secondary truss and Vierendeel beams in both directions are restrained by perimeter columns support in an alternating fashion, i.e. every second secondary beam is restrained. This causes uneven lateral deflection of edge beams. The edge beams which are supporting walls on east and west faces cannot be seen a stable support system.

7 Further Research

(a) Fire effects on the columns will be considered
(b) Diaphragm action of the slab will be included
(c) Analyses for later stage of the fire will be done
(d) Realistic fire scenario will be taken.

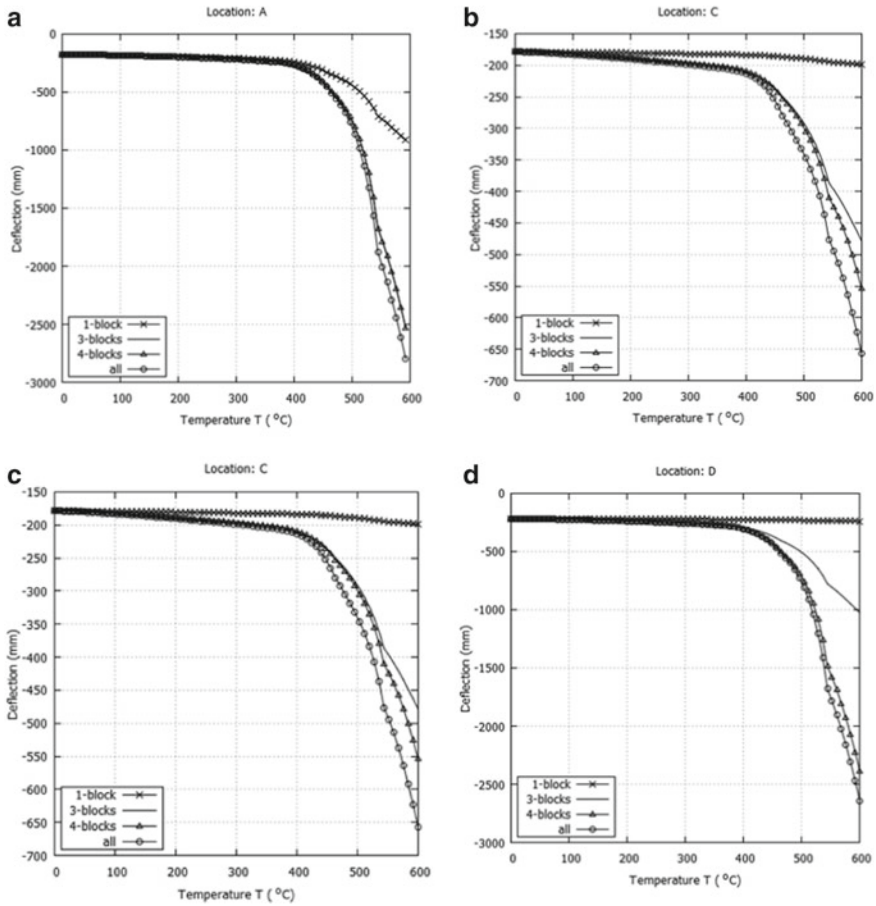

Fig. 8 Comparison of mid-point displacements in beams A, B, C and D under the four fire zone extents

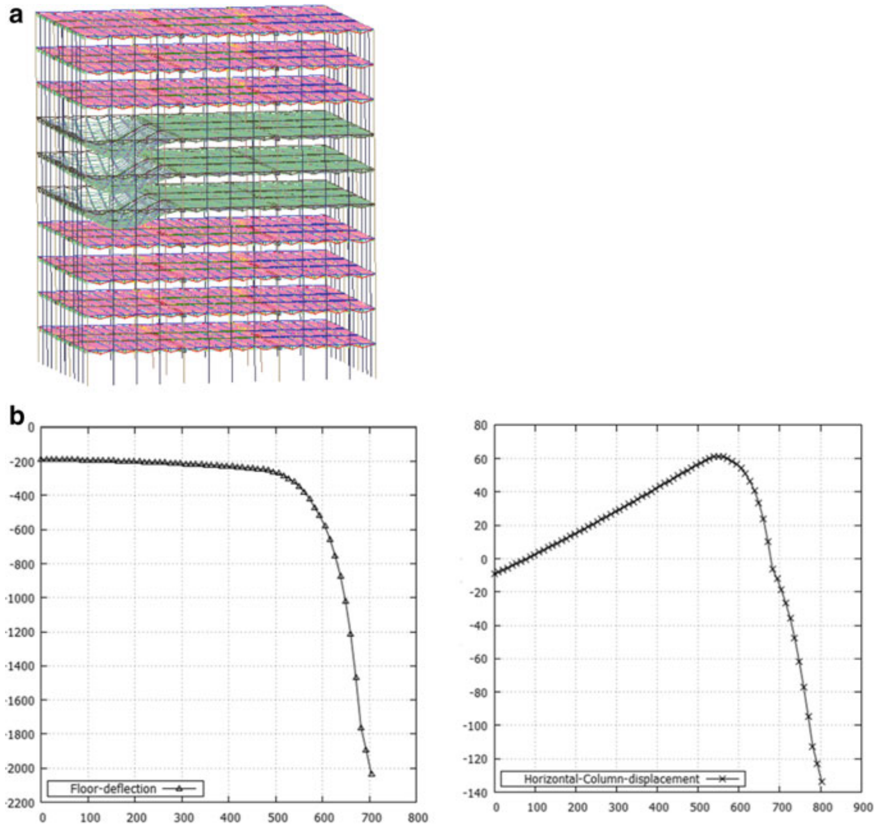

Fig. 9 **a** Horizontal displacements in and **b** Difference in deflection between Ce2 and C2 columns under critical 4-block fire loading

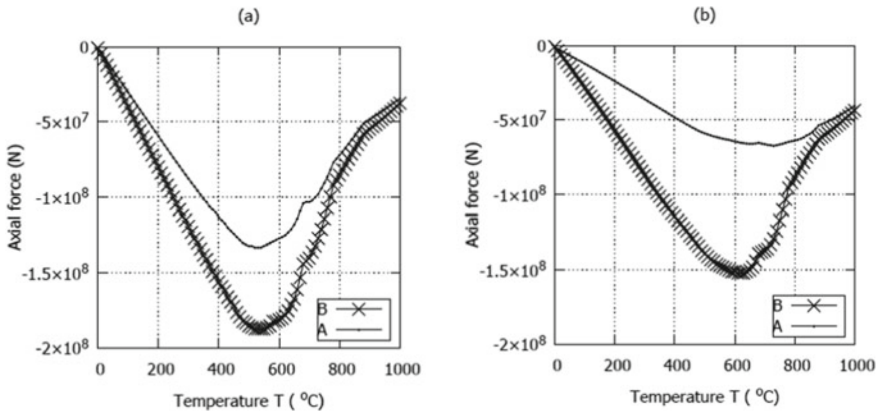

Fig. 10 Comparison of top chord axial forces at the mid-sections of primary beam at locations A and B during **a** 1-block fire and **b** 4-block fire

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