The Collapse of World Trade Center 7: Revisited

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Abstract

The catastrophic events of September 11, 2001, stand out as a major motivation for research on improving the understanding of structural behaviour in fire. These events included the first complete collapse of a tall steel framed structure solely due to fire. World Trade Center 7 (WTC7) was a 47-storey office building within the WTC complex that collapsed due to a fire initiated by debris from the collapse of WTC1. In the following years, detailed investigations were carried out by expert teams to pinpoint the cause of the progressive failure of WTC7. Each of the expert teams analysed the fire and structure and made varying conclusions with regards to the mechanisms responsible for initiating and propagating the collapse of the building. This paper revisits the collapse of WTC7 and its investigation, and then explores the hypothesis that a potential hydrocarbon fire may have compromised the large transfer structure within the mechanical space of the building. This is done via two OpenSees finite element models. The first model explores the thermomechanical response of the mechanical floors to a potential diesel fire, and the second investigates the response of the structure to a failure caused by that fire. The outcome of the analyses shows that it is feasible that a mechanical room fire could lead to a failure in the transfer structure, which would then result in the loss of support to at least two columns within the building core. The failure of these columns may unbrace the eastern-most core columns and precipitate in the failure of the structure as observed on 9/11.

Keywords: *progressive collapse, fire resilience, world trade center, case study, finite element method.*

1. Introduction

The World Trade Center (WTC) complex was a collection of seven high-end medium and high-rise buildings within the financial district in Lower Manhattan, New York City. WTC7 was the latest addition to the complex and had 47 storeys. During the events of September 11, 2001, WTC towers 1 and 2 were each struck by a commercial airliner. With all WTC buildings in relatively close proximity, the debris from the twin towers caused severe damage to the surrounding structures many of which collapsed. WTC7 was located north of WTC1 and suffered some damage that did not directly jeopardise its structural integrity. The debris, however, set over ten floors alight within WTC7 in a time where the attention of first respondents was more needed away from the building [1]. After seven hours of multi-floor traveling fires, the structure finally collapsed at 5:21 PM.

WTC7 was the first tall steel-framed building to collapse solely due to fire. Following the disaster, more emphasis was placed on the design and construction of fire resilient structures [2]. It was even emphasised by the National Institute of Standards and Technology (NIST) that structures must be able to withstand burn-out without either localised or progressive failure [1]. This naturally led to additional research attempting to better understand the response of steel-framed and composite construction to fire, such as [3–7], and has inspired significant research into the behaviour of steel connections in fire [8,9]. However, even after years of investigation by some of the world leading experts in both fire and structural engineering, the collapse of WTC7 remains unresolved. Different investigations have proposed specific hypotheses for the failure, but they are not consistent in either detail, cause or mechanism [1,10,11]. As one of the most important collapse events in the field of structural fire engineering, the fire-induced failure of WTC7 ought to be more thoroughly examined.

WTC7 was erected atop an existing power substation. Therefore, the superstructure of the building had to be interfaced with the existing structure of the power substation by a set of large transfer trusses. Trusses 1 and 2 were two major transfer trusses located within the double-height mechanical room on the fifth floor of the building. This space coexisted on the fifth floor with a number of power generators, some of which were fed by a continuous pressure loop linking to 12,000 gallons of diesel fuel at the base of the building. This was one of two primary fuel storage systems for WTC7, and was added to the building in later years after completion [12]. All fuel, about 23,000 gallons, from the other storage was recovered but the 12,000 gallons of diesel linked to the fifth floor were never retrieved. According to NIST's investigation, no fuel residue was found around the storage area ruling out spillage, and their interview with the fuel supplier indicated that the fuel storage ought to have been full [1,12]. As a double height-space with access to ventilation from adjacent louvers, it is possible that all the missing fuel was available for combustion at the fifth floor.

This work explores the hypothesis that a fire within the mechanical floor fuelled by the missing diesel fuel may have contributed to the collapse of WTC7. The paper begins by introducing the WTC7 structure, its unique backup energy systems, and the events that led to its collapse. An overview of previous expert investigations and their results is then presented, with focus on how they analysed the fire and the structure. This is followed by exploring the potential for a diesel fuel fire within the mechanical space on the fifth floor of the building, and how that could have compromised the structure. Two models of WTC7 are then presented for the analysis of the response of the structure to a potential fire within the mechanical room. The first model deals with the thermomechanical response of the building, and the second addresses the potential collapse propagation. The results of the analyses show that that a mechanical room fire may have caused a failure within the transfer structure located between the fifth and seventh floors. This failure would destabilise two of the columns within the building core.

The second model showed that the collapse of these columns causes concentrated damage and reinforcement strain at the edge of the core (column line 79-80-81). Coupled with the observed multi-floor extreme fires WTC7 was subjected to, this may have unbraced the eastern-most core columns over multiple floors and initiated the collapse of the building. While the presented preliminary analyses point to this hypothesis, further study needs to be done in the future to validate it. In the meantime, the open-source tools used for the analyses performed herein need to be improved so that they are more efficiently capable of dealing with large models, and so that they can consider more sophisticated details such as joint failure which the current study does not account for.

2. Background

2.1. The design of WTC7 and its subsequent collapse

WTC7 was 186 m high, which made it the third tallest building in the WTC complex after WTC1 and 2 respectively. Its footprint was larger than WTC 1 and 2, with a trapezoidal plan with dimensions of 100 m × 75 m × 43 m × 45 m as shown in Fig. 1 (a). The building was erected atop the Consolidated Edison power substation which fed Lower Manhattan. The first four floors from the ground contained the power substation, the lobby of the building, as well as other functional space such as a cafeteria and a conference room [1]. In addition, the first floor also contained the pumps for the emergency power system [12]. The fifth and sixth floors were primarily mechanical spaces containing ventilation equipment, power generators, and other equipment. Floors above the sixth consisted mainly of both open-plan and traditional office spaces. The gravity system of the building consisted of wide flange beams and columns supporting a 140 mm thick concrete-on-steel-deck composite slab. Lateral load resistance was provided by moment resisting connections along the perimeter frame and by two-storey high belt truss systems between the 5th and 7th floors, and the 22nd and 24th floors. Core bracing below the 7th floor resisted lateral loads transferred to it by the floor diaphragm. Most floor framing connections consisted of various types of shear connections including fin plate, knife, and multiple variants of seated connections [13]. Spray applied fire resistant materials (SFRM) fire protection was applied to the structural components with a 3-hour rating for the columns and 2-hour rating for the steel decking and beams, which exceeded the required fire protection of 2 and 1.5 hour-rating for the columns and beams respectively [13]. The entire building was also protected by sprinklers fed from the city mains from the ground to the 20th floor, and complemented with water tanks that serviced the 21st floor and above. To transfer the load from the structure into the columns and foundation while bridging over the Con Edison power substation below, a transfer system was used which included three trusses, eight cantilevered built-up girders, and three transfer girders all between floors 5 and 7. The part of the transfer system in the east of the building is shown in Fig. 1 (b)

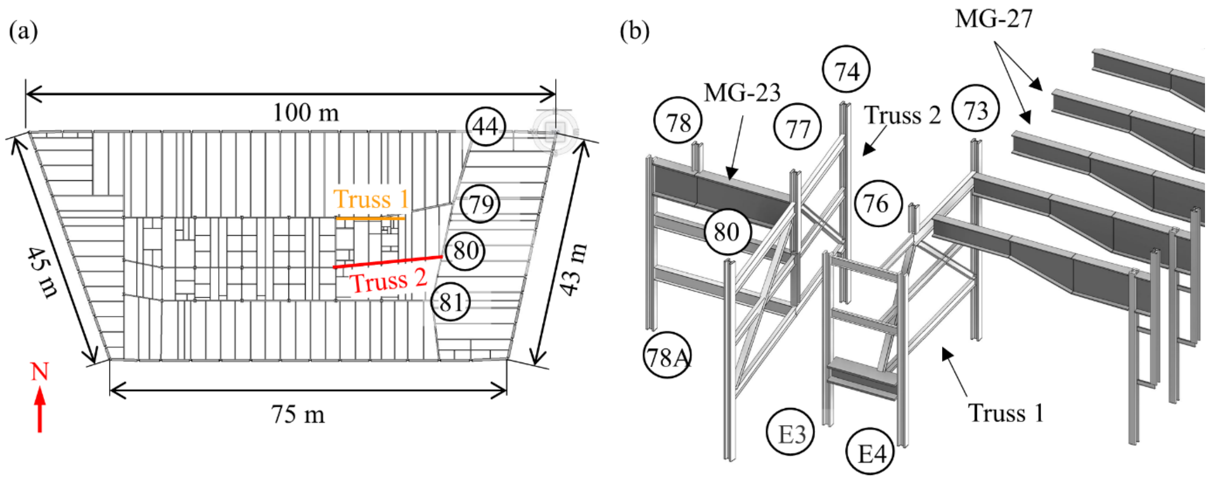

Fig. 1. Typical floor layout of WTC7

To ensure continuous operation of the primary functions of the building, an emergency power system was installed. This system had a total diesel fuel capacity of about 31,000 gallons which fed through several pump sets that powered generators located on the 5th, 7th, 8th, and 9th floors. Separate from this system was the Salomon Brothers emergency power system which had its own subgrade diesel tanks with a capacity of just 12,000 gallons, and nine power generators all located on the 5th floor [12]. About 23,000 gallons of the fuel in the primary emergency system storage tanks was recovered a few months after the collapse, which is within the expected residual amount. None of the fuel within the Salomon Brothers system was recovered, however, and there were no traces of leaked residue within the proximity of the then-damaged tanks. Since there was a pre-existing 275-gallon day tank on the 5th floor as part of the primary emergency power system, the Salomon Brothers system had to be designed without a day tank. The system, therefore, was designed to supply fuel continuously to the generators as long as they were operational. The rate at which fuel was pumped to the generators was controlled by a regulator downstream. As long as the pressure downstream from the generators is low, fuel would continuously circulate to power up the generators which in turn powered up the pumps [12].

After WTC1, the Pentagon, and WTC2 were attacked, all 4,000 occupants of WTC7 evacuated the building [1]. When WTC1 began to collapse at 10:28 AM, debris launched by the collapse damaged and penetrated the southwestern face of WTC7 severing the perimeter columns over floors 7 through 17 along the building corner. Large areas of the southern façade glass were also broken. Nonetheless, the debris damage to the building and its structure did not pose an immediate threat to its stability. The water supply to the sprinklers from the city mains was disrupted due to the collapse of WTC1 and 2, and the early evacuation of building occupants made the building a low priority for the fire department which was preoccupied with rescue and firefighting operation elsewhere. Multiple fires erupted as a result of the burning debris that made it into the building, with visible fires over floors 7 through 9 and 11 through 13 burning uncontrolled for about 7 hours. The fires burned throughout the lower fire floors (floors 7, 8, and 9) spreading in a clockwise manner and travelled counter clockwise in the upper fire floors 11, 12, and 13 [14].

2.2. Expert investigations into the collapse

The collapse of WTC7 destroyed the electrical substation below it, which was a logistical disaster for the city during a very difficult time. A lawsuit was later filed in which Consolidated Edison along with several insurance firms sued the designers, contractors and operators of WTC7. This resulted in several expert teams investigating the collapse and each providing a plausible scenario by which the building may have failed. Additionally, NIST were funded by congress through the Federal Emergency Management Agency to do an independent study for public interest [1]. In this section, the various collapse scenarios are introduced and discussed to give a historical and technical background of the analyses that have been performed. In all these studies, detailed arguments were presented in support of their hypothesis and among those arguments were parallels between model performance and evidence collected during the fires. While all this information is of importance, it is clear that images are very limited in the information they can provide, in particular when it comes to complex structural behaviour. Thus, the focus here will be on the modelling and these comparisons will not be presented here, the reader is directed to the original studies for that information.

2.2.1. NIST

The investigation by NIST commenced after WTC7 debris was cleared, and so the primary investigation material was videos of the incident and the information known about the structure as per the drawings available [15,16]. To analyse the failure, NIST considered four major factors: 1. Structural damage from the debris of WTC1, 2. Fire propagation as seen in the videos, 3. Computational fluid dynamics (CFD) modelling of the fire calibrated with the videos, and 4. Nonlinear static and dynamic finite element (FE) models of the building.

According to the NIST investigation, the most severe fires were those occurring on floors 7 through 9 and 11 through 13 [1]. Fires on other floors may not have had a significant effect because they only burned briefly and are thus not visible on video. Floors 7 through 9 comprised mostly of open plan offices with a fuel load of 20 kg/m^2 mainly consisting of office equipment and paper. This fuel load was determined indirectly by considering the fire spread rate from the videos and comparing with the load-dependent calculated rate [17]. The upper fire floors, 11 through 13, had more traditional offices separated by partitions and joined with false ceilings. The paper load on these floors was higher and so the fuel load NIST arrived at was 32 kg/m^2 [1]. With this data, NIST used Fire Dynamics Simulator (FDS) to produce a model of each of the fire floors individually. The models for floors 7, 8, and 12 were calibrated based on the visibility of flame and glass breakage in the videos. The models for the fire on the 9th floor was derived from the 8th floor model, and the models for floors 11 and 13 were based on the 12th floor model due to lack of sufficient footage for calibration of each individual model. Using FDS derived gas temperatures, NIST then calculated the temperatures in the structural members and varied them by ±10% to create three scenarios that envelope the potential building fires and account for uncertainties in the modelling [14,18].

The structural analysis of the building was performed in two stages: stage 1. Implicit pseudo-static analysis of the bottom 16 floors of the building considering the heating effects, and 2. Dynamic explicit model of the entire building to investigate debris impact, heating damage, and collapse propagation. The first model was created in ANSYS and accounted for shear stud connection failure in the eastern part of the fire floors, lateral torsional buckling of floor beams, and removal of slab sections that had reached large strains. Connections were modelled as collections of springs in series or parallel depending on the type of connection modelled, and shear studs were modelled as 'break elements' that would be removed once they reached their capacity. The three fire scenarios (computed temperatures, computed temperatures + 10%, computed temperature -10%) all produced the same structural behaviour with only a variation in the time of occurrence of the failure mechanisms due to the ±10% variance in applied temperatures. The thermo-mechanical damage generally resulted from connection failure which led to loss of vertical support of several floor beams. Lateral torsional buckling also occurred in some members where loss of shear studs was considered. The analysis was stopped after 3.5 to 4 hours of heating, and results transferred over to a dynamic explicit model built in LS-DYNA [1].

The second model was analysed in incremental steps, starting with gradual application of dead load to avoid introducing undue vibrations into the structure. The impact and damage from the WTC1 debris was then added instantaneously, and then the model was allowed to stabilise. The addition of the debris impact showed that the structure could redistribute its load-bearing capacity and avoid disproportionate damage or collapse. The temperature of the structure was then gradually increased up to the levels reached in the ANSYS model to induce restrained expansion effects, before finally applying material and component damage instantaneously to the model. After the damage was applied, the model showed that the structure would undergo progressive collapse.

According to the NIST models and investigation, the collapse of WTC7 primarily initiated because of connection failure. The connection between the internal column 79 and the girder linking it to perimeter column 44 failed by girder walk-off during the heating phase. To the east of column 79, many of the floor beams were lost to either loss-of-vertical-support or lateral torsional buckling. The severe heating and restrained expansion of the floor slab caused what remained of the floor to lose its stiffness by concrete crushing. This severely weakened the fire floors causing partial collapse which could not be impeded and resulted in cascading floor failure. Column 79 was thus left unsupported over 9 floors causing its buckling failure which caused the kink in the penthouse observed in the collapse video. The buckling of column 79 was followed by similar failures of columns 80 and 81, and then the progressive collapse of the adjacent column lines. The collapse of the interior of the building was followed by perimeter failure starting at a location adjacent to the entry point of the majority of the WTC1 debris damage in the southwest corner of the structure. The perimeter columns failed in quick succession and the complete failure of the building occurred [14].

2.2.2. Arup & Guy Nordenson and Associates

Arup and Guy Nordenson took a different approach to the analysis of the WTC7 collapse. First, Arup performed a series of FDS simulations to characterize the temperature evolution of different compartments. These temperatures were used in a simplified heat transfer analysis to determine structural temperatures. Finally, the structural temperatures were imposed on an FE model to predict the collapse initiation event. After the Arup analysis, GNA performed a series of nonlinear studies to study the progression of collapse from its initiation point and projected the results on a large linear FE model.

Similar to NIST, Guy Nordenson Associates (GNA) performed a photographic study of the debris damage and fire in WTC7 [19] and Arup performed a large scale FDS study of the 12th floor, which was one of the six floors (7th-9th, 11th-13th) where severe fires occurred. Arup's FDS model was calibrated with the observed fire behaviour, and the window breakage was programmed following the photographs and videos [20]. The fuel load used was 36.6 kg/m^2 which correlated well with the values provided in the NFPA Handbook for private and government office fuel loads [20,21]. No vertical fire spread was allowed, mechanical ventilation was assumed to have been off, and the building core was not breached and remained free from fire throughout the simulation [11]. The simulation results were in very good agreement with the observed fire in the north and east faces of the building, but less so with that in the west face. However, as Arup had determined that the critical fire space was to the east of the core, the FDS results were deemed acceptable [11]. The FDS simulation was simplified for structural analysis, resulting in a characteristic heating regimen of 1 hour heating at 800 °C and 1 hour cooling at 20 °C [20]. This approach was based on the ceiling-level temperatures at multiple monitoring points throughout the area east of the core, and despite being simple, it accounted for the traveling nature of the fire, preheating, and conservation of energy. The heat transfer analysis was performed at the University of Edinburgh. Two heating scenarios were considered: the aforementioned characteristic heating at 800 °C for 1 hour followed by cooling for another hour, and a less severe scenario where the peak heating temperature was reduced to 700 °C. In addition, two fire protection configurations were considered: one in which the space between the steel decking and the floor girder was filled with SFRM, and another where these 'flutes' were left unfilled. The filling or not filling of the flutes with SFRM resulted in significant differences in the thermal load on the structure, with the latter being a potentially critical link in the chain of failures leading to progressive collapse."

The structural model used by Arup was built in Abaqus/Explicit [22] and represented the east half of a typical floor, with symmetry boundary conditions representing the other side as shown in Fig. 2. The ribbed slab was abstracted with uniform-depth shell elements including the steel decking and using Concrete Damaged Plasticity [22,23] to consider material nonlinearity. While most of the steel framing was idealised using beam elements, columns 44 and 79 as well as girders 79-80, 76-79, and 44-79 were modelled with a higher grid resolution using shell elements [24]. Likewise, both connections of girder

44-79 and all connections into column 79 were modelled in full detail using shell elements to represent the various plates, tie elements to model the bolts, and rigid links to represent the welds. Shear studs were only modelled in the northeast corner of the floor, and assumed to be rigid elsewhere. Their force-slip relationship was temperature dependent, and 'broke' beyond a slip limit of 6 mm as proposed by [25]. Details of the model as built by Arup can be more clearly seen in Fig. 2 [11]. Both mass and time scaling were used to ensure an efficient solution, while inertial forces were monitored to maintain reasonably accurate results [24]. The Arup analysis predicted that failure could indeed commence in the vicinity of column 79 in either heating or cooling and only if the flutes between the girder and slab were unfilled. The expansion of the beams in the northeast, as well as the breakage of the studs over the girder, results in the girder severing its connection bolts and being pulled off its seat. Given this initiating event assessed by Arup, GNA investigated collapse propagation using a full-scale elastic model built in SAP 2000 [26]. The model was supported by multiple nonlinear sub-models that would manually inform the progression of the analysis such as removing particular columns if they were prone to buckling. None of the models used by GNA were thermo-mechanically loaded and instead the analysis focused on collapse progression rather than assessment of thermal damage which was assumed to have been adequately addressed by the prerequisite Arup analysis.

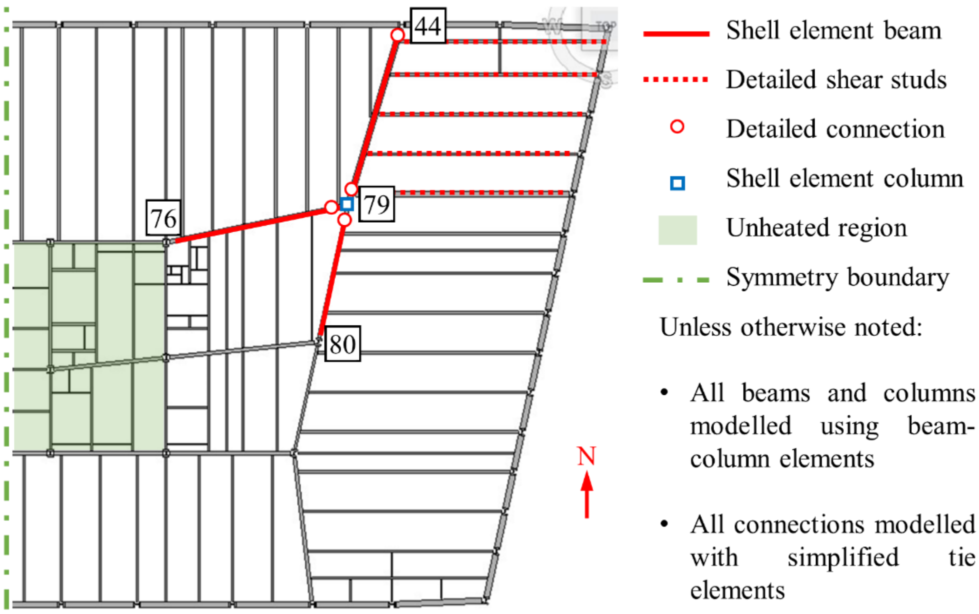

Fig. 2. The Arup model details [11]

The Arup-GNA investigation concluded that several factors including some design and construction defects contributed to the collapse of WTC7. While the debris from WTC1 contributed to the collapse by starting the fires, the impact and structural damage caused by debris was largely inconsequential. Despite the fires burning uncontrollably for 7 hours throughout the structure, their effect only jeopardised the building after reaching the eastern face. There, the high loads borne by column 79, the asymmetry of floor framing into girder 44-79, and most importantly the un-filled flutes (no fire protection for the top flange) caused girder 44-79 to be pulled off of its seat at its connection to column 79 [27]. From that initiating event, the analysis performed by GNA showed that the sequence of events following would lead to global collapse. Just after the failure of girder 44-79, the northeast corner of the floor slab would collapse over multiple floors causing column 79 to lose its support and thus buckle. The failure of column 79 and its supported floor area then left column 80 unbraced and its buckling then caused the kink observed in the penthouse as seen in the collapse videos. The falling debris then damaged the transfer trusses and resulted in the failure of the next column line followed by pulling-in of the remaining core and causing the failure of interior of the building. Finally, the perimeter frame buckled resulting in total collapse of the building.

The analysis presented by this group also included a study by the BRE Centre for Fire Safety Engineering and the University of Edinburgh on the alternative scenario of a fire in the mechanical room. This will be discussed in detail in a subsequent section.

2.2.3. Weidlinger Associates and Hughes Associates

Weidlinger Associates Inc. (WAI) and Hughes Associates produced two reports that were based on their examination of the incident, one regarding the fire and another dealing with the structural response. Unfortunately, the former, produced by Hughes Associates, was never made public and so little about it can be said.

Despite the WAI report focusing on the structural and thermomechanical behaviour, some important insight about the fire is given. First, it is assumed that the debris damage not only started the fires over multiple floors, but also inhibited the fire prevention measures in the building [10]. It is assumed that most compartmentation was lost and that the sprinklers were destroyed due to the debris impact. It is also noted that the presence of widespread fires on affected floors resulted in a severely reduced ability to arrest potential progressive collapse by weakening the floor capacity to resist the impact of a falling floor.

WAI used an explicit dynamic model to study the collapse of WTC7. The analysis performed by WAI relied on one full building model, and two sub-models representing the eastern bay of one and two floors respectively. The sub-models were used to study the thermo-mechanical response of the east bay, while the full building model was used to study the collapse propagation. All models were built using WAI's inhouse nonlinear FE software 'FLEX' and included a very high level of detail. All shear studs, welds, connections and slab-beam contact were modelled explicitly in the sub-models. Moreover, all beams, girders and columns were modelled using shell elements. The global model was less detailed with floor framing connections assumed to be pinned, and others built as consolidations of uniaxial spring based on connection capacity estimation using the component method [28]. Beam-column elements were used to represent the steel framing [10].

Mechanical load was applied to the models by increasing the gravity and the density incrementally until full quasi static load was applied. Thermal loading was also a quasi-static phase employing time scaling and incurring few dynamic effects. The thermal loading continued until failure initiation, at which point dynamic analysis proceeds until re-stabilisation or total collapse.

The WAI analysis also showed that the collapse initiated in the eastern part of the building. However, the initialising event did not occur in the vicinity of column 79 but in the proximity of column 80. Uncontrolled heating in the eastern bay had led to floor deflections in the order of 1/15 of the span, which resulted in high tensile forces in the floor beams. At the same time, the uncontrolled heating had resulted in high connection temperatures. This caused the nearest connection to column 80 to sever, which resulted in zipping off of adjacent beam-girder joints. With this, the eastern portion of the floor would collapse impacting a similarly weakened floor below and resulting in progressive failure of the east section of the floor of WTC7. Columns 79, 80, and 81 are then left unsupported over multiple floors which causes them to buckle thus generating the kink in the penthouse. The collapsing debris then severely damaged the transfer structure on the fifth and sixth floors and resulted in westwards internal collapse propagation. The unbraced perimeter columns finally buckled and the structure collapsed [10].

2.2.4. The potential for a mechanical room fire

As previously mentioned in section 2.1, the Salomon Brothers emergency power system was designed as a pressurised loop where fuel would continuously be pumped from the storage tanks as long as the generators were operational. As there was no indication of any automatic leak-detection in the Salomon Brothers system, NIST judged that all available fuel could have been available at the fifth floor to feed a potential fire [14]. NIST dismissed this scenario, however, based on three reasons:

1. A pool fire would have raised the temperature beyond the generator operational limit thus cutting the power supply to the fuel pumps and effectively cutting the loop.
2. Even if a fire had occurred, it would not have heated the structure to critical levels.
3. A critical fire around the generators would have resulted in significant exhausted smoke visible from the outside of the structure but none were seen.

This assessment was based on FDS results assuming a pool fire occurring in the vicinity of column 79 just outside the mechanical room located on the fifth floor [14]. However, a fire could have arisen in the mechanical room if: (a) a breach in the fuel line and masonry wall of the mechanical room may have led the fuel line to leak inside of it, or (b) a fire started just outside the mechanical room and propagated into it through damage in the separating partition. Both scenarios would have heated the large transfer trusses in the mechanical room potentially resulting in critical damage to the structural integrity of the building.

Such a hydrocarbon fire would likely generate large amounts of smoke that should have been visible outside the east face of the structure. During the last hours of the fires of WTC7, however, smoke had spread extensively, and visibility was poor. It is therefore difficult to judge whether smoke was emanating from the mechanical room or elsewhere in the building. Furthermore, the part of the seventh floor atop the mechanical room had multiple large openings that may have provided an avenue for the smoke to propagate elsewhere into the building and potentially exhaust from other openings. It is noted by Mowrer [20] that dark smoke indicative of hydrocarbon combustion was observed in the videos and witnessed by first responders after 3:30 PM just outside the northeast corner of the building as seen in Fig. 3 [19]. This may potentially have been smoke exhausted due to a fire in the mechanical room.

Assuming a pool fire did indeed start in the mechanical room, an FDS analysis was run by the BRE Centre for Fire Safety Engineering [29]. Several ventilation conditions were assessed considering the opening of the mechanical room to the plenum east of the building. The results were then compared to the Thomas [30] curve since the fire had reached steady-state combustion within 300 – 600 seconds in all the assumed ventilation test cases. All FDS simulations gave predictions below the Thomas curve indicating that the analyses were conservative and within the range of recorded experimental data [29]. If such a fire did indeed occur, then the members within the mechanical space may have been thermally compromised.

This is particularly important because the mechanical room contained a portion of Truss 2, one of the two major transfer structures in the east of the building. Truss 2, as shown in Fig. 4 (a), supported two core columns and had a large built-up girder (MG-23) framing into it from the south. Fig. 4 (b) through (c) shows a potential failure mechanism that may have been induced by heating of the truss. As the diagonal members are under high compressive loading due to restrained thermal expansion, the heated girder MG-23 applies out-of-plane load to the truss. Unable to sustain the loads, the truss may fail and affect the overall stability of the building. The hypothesis that the fire may have compromised the transfer structure which would in turn affect global stability will be investigated in the coming sections.

Fig. 3. Smoke outside the northeast corner of WTC7 between 4:15 and 4:38 PM. From [20]

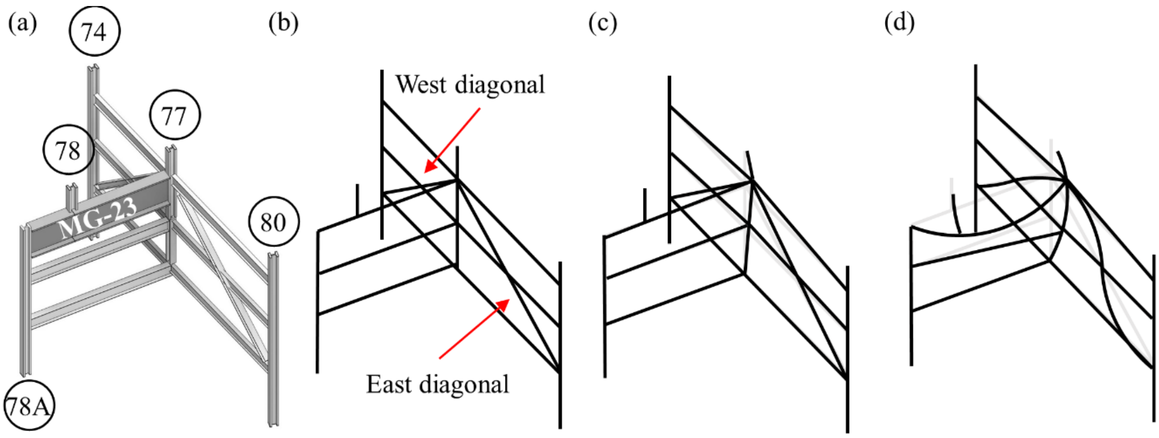

Fig. 4. The potential collapse of Truss 2 (a) Truss 2 column and girder designations, (b) Truss 2 line representation, (c) As MG-23 heats up it pushes on Truss 2 out-of-plane, and (d) due to P-δ effects Truss 2 diagonals buckle and vertical support is lost to columns 77 and 78

3. Modelling the WTC7 response to a potential mechanical room fire

3.1. Modelling considerations and simplifying assumptions

As per the description given in section 2.2.4, it is possible that a downstream leak in the Salomon Brothers auxiliary power system may have gone undetected and resulted in a pool fire within the mechanical room. A diesel fuel fire in this double-storey open space may result in gas temperatures in excess of 1000 °C after reaching steady-state combustion. FDS analysis performed by the BRE Centre for Fire Safety Engineering team had shown that such a pool fire would reach steady-state within 5 to 10 minutes [29]. Based on this, the models presented in this paper will utilise the temperatures of the standard hydrocarbon curve to represent a mechanical room fire lasting up to 3 hours. While this is a coarse approximation of the possible fire, it is a good starting point for the preliminary assessment performed in this paper.

OpenSees has been extensively developed and validated as a powerful open-source tool for modelling structures in fire [31–39]. It has also been used as the heat transfer and structural analysis solution engine of the Integrated Simulation Environment (ISE). The ISE is a recent open-source software framework for efficient model building and analysis of large structures in fire discussed in great detail in [40]. The ISE is centred around the GiD pre and post processor, and offers a myriad of key functionalities for structural fire analysis atop the original GiD+OpenSees interface [41–43]. For example, the ISE allows for importing a frame model of WTC7 from Revit, assigning OpenSees elements and loads to it, and automatically performing heat transfer analysis given a specified fire scenario. The results of the heat transfer analysis are then automatically applied to the OpenSees elements, followed by thermomechanical analysis in OpenSees and post processing in GiD. Because of this, both WTC7 models presented in the next section will be built in the ISE and analysed using OpenSees. In both models, the beams and columns are represented using displacement-based beam-column elements with corotational transformation to include the effects of geometric nonlinearity. The beam-column sections are fibre-based so that the spread of material nonlinearity is considered throughout the section, and across the integration points of the element. Floor slabs are abstracted using the geometrically nonlinear multi-layered thermomechanical NLDKGQ shell elements [33,44,45]. The concrete layers are represented using the plane stress concrete damage plasticity material, and the reinforcement bars are modelled using the 'rebar mesh' approach for the first model, and the 'steel layer' approach for the second. In short, the rebar mesh approach uses uniaxial steel material to model smeared reinforcement acting across either of the two planar dimensions thus requiring two layers to represent transverse and longitudinal reinforcement bars. The steel layer approach uses a plane-stress implementation of the J2 plasticity material to model a smeared biaxial layer of reinforcement bars. A layer of J2 steel material is also used to represent the steel of the composite slab decking, and a layered section with J2 steel layers was used to represent the large built-up girder in the mechanical room. Connections were assumed to be fully fixed and not modelled explicitly in either model. This choice was made because it was aimed to explore global failure modes and see whether strong-floor or weak-floor failure mechanisms would present [46,47]. Using fixed connections has been shown to be effective when modelling the other WTC towers [48,49], and was thus chosen for the preliminary analyses performed here. Likewise, all shear studs were modelled using beam-type rigid links that transfer all translations and rotations and do not fail.

The concrete compressive strength was set to 24.1 MPa as per the erection and structural drawings [15,16]. The tensile strength was specified as 2.41 MPa, Young's modulus set to 18 GPa, crushing strain set to 0.002, and tensile and compressive damage multipliers m and n set to 10. These values were chosen based on previous experience as discussed in previous work [33,45]. The tested material properties, as reported in [24] and shown in Table 1, were used for the various steel grades used in WTC7. The temperature-dependent mechanical and thermal properties are based on the provisions of the Eurocodes such as the EN1993 and EN1992 [50,51]. Details of the implementation of the mechanical models are presented in depth in [33,36,40] while the details of the thermal material models are discussed in [35,52]. The details of the heat transfer analysis undertaken within the ISE are the same as that presented in [40].

Table 1. Steel grade, yield strength, and ultimate strength used in the models. Based on tested values as reported in [24]

Steel grade	Yield strength	Ultimate strength
A36	314 MPa	505 MPa
A572 – Gr 50	413 MPa	573 MPa
A572 – Gr 42	348 MPa	573 MPa

The SFRM applied to WTC7 was Monokote MK-5 [53]. Based on the test data from NIST, the conductivity and specific heat capacity relationships shown in Fig. 5 were implemented in OpenSees. The figure includes both the test data from NIST and the linearised expressions implemented in OpenSees and used for heat transfer analysis. Heat transfer analysis in the ISE is performed using predefined section abstractions utilising a mesh with element size of no more than 15 mm and at least four elements across the width of the steel section [40].

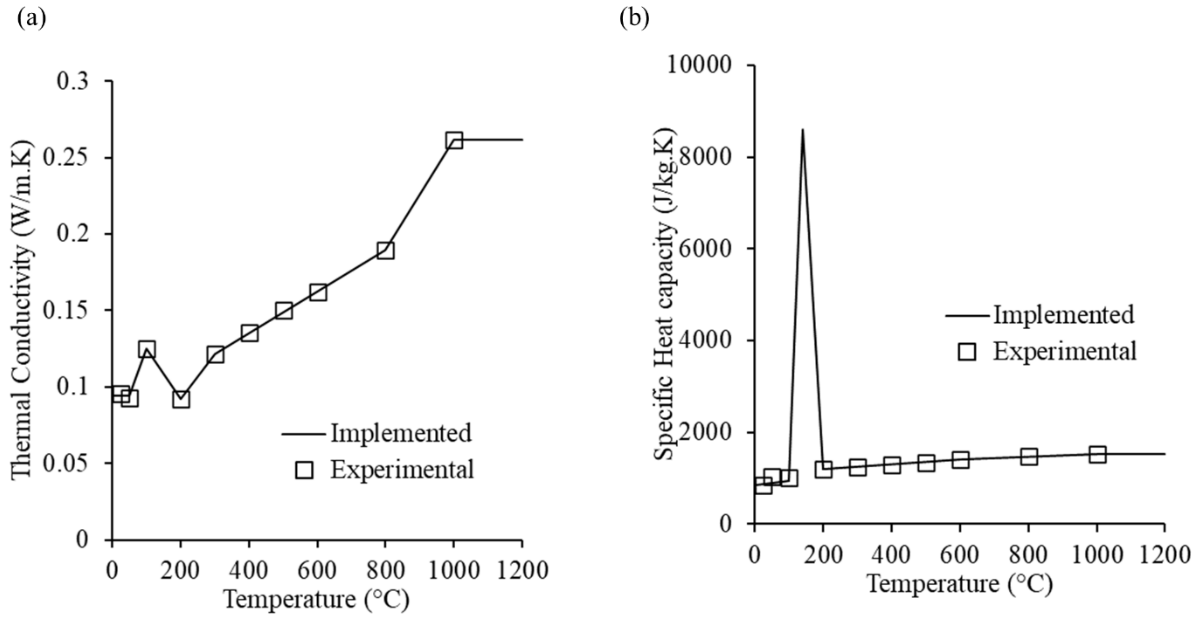

Fig. 5. Experimental and linearized properties for Monokote MK-5 [53] (a) conductivity, and (b) specific heat

3.2. Model 1: thermomechanical model of mechanical floors

The first model aimed to examine whether a failure was likely to occur due to a hydrocarbon fire within the mechanical room. The mechanical room was located in the east of the building and spanned between the 5th and 7th floor. Therefore, only the eastern part of the building was modelled as demarked in Fig. 6 (a). The extent of this model encapsulates the double-height mechanical space highlighted in Fig. 6 (b) as well as the floor slab of the 6th, and 7th floors. The figure shows the Revit model that was used to build the OpenSees model of WTC7 within the ISE.

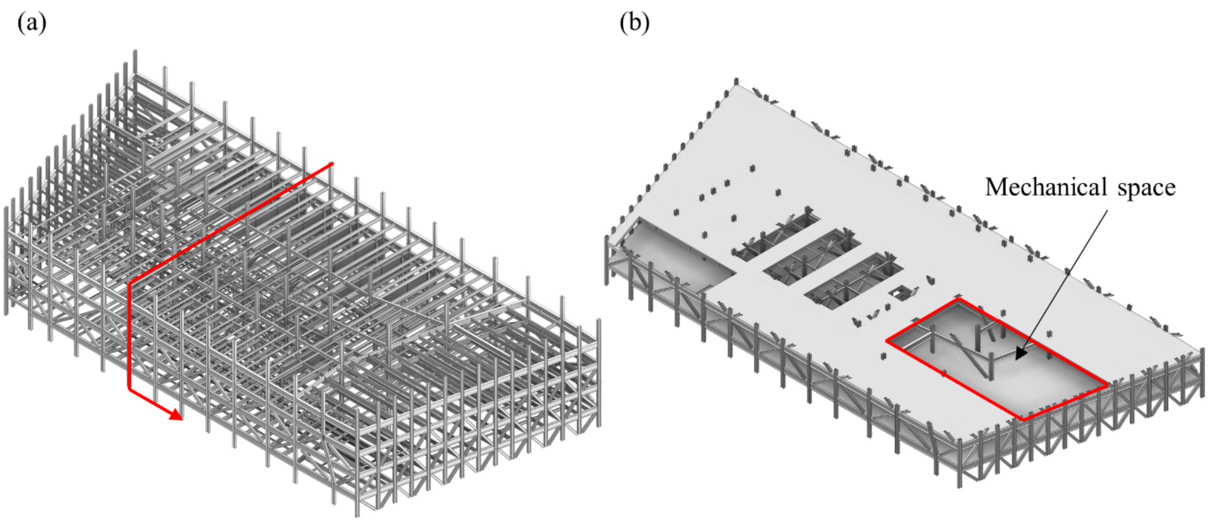

Fig. 6. Revit model of WTC7 (a) extent of floor included in the GiD+OpenSees model, and (b) location of the mechanical space showcasing the double-floor height

After importing the Revit model into GiD as shown in Fig. 7 (a), the section designations and floor and column loads were applied following the erection drawings [15]. The columns extended from the 4th floor where they were assumed to be fixed to the 8th floor where they were allowed to translate vertically as shown in Fig. 7 (b). The edge at which the model is discontinued was fixed in all degrees of freedom, as it is away from the location of the mechanical room fire and thus expected to be cool and stiff. As will be shown in the results and discussion, this does not seem to affect the overall structural behaviour as the effects of the fire in the mechanical space remained localised. The mesh size of the fire-affected 7th floor was approximately 0.5 m, while the 6th floor slab was meshed with element size of about 0.7 m. These values are of the same order of magnitude as NIST's thermomechanical model which had element size of 0.9 m for the floor members [18]. The model had a total of 22,823 nodes and 22,978 elements.

(a) (b)

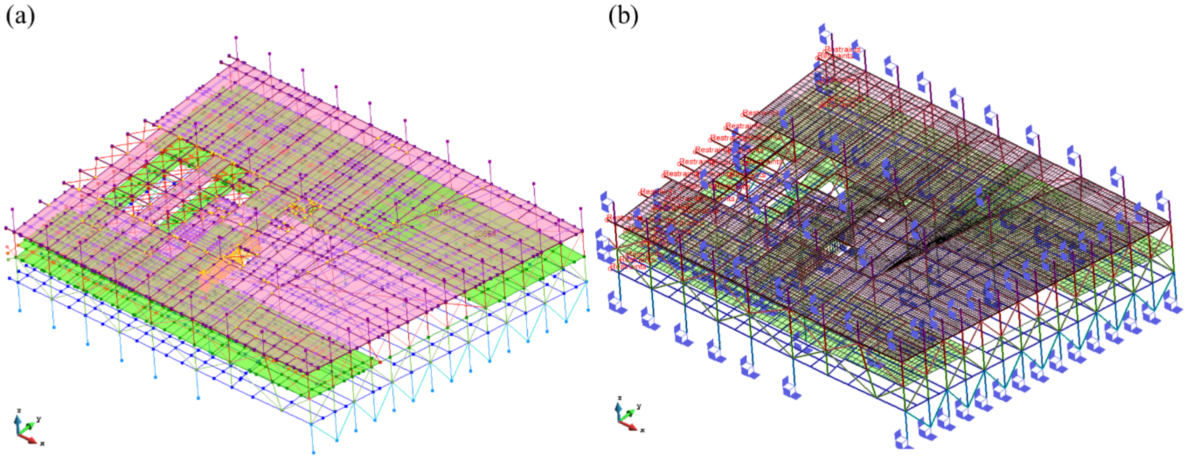

Fig. 7. Model of WTC7 mechanical floors (a) geometry imported from Revit, and (b) mesh and boundary conditions

The analysis was performed over two intervals: a static interval for application of ambient dead loads, and a dynamic pseudo-static interval for thermomechanical analysis. The static interval used load-controlled integration with the UMFPACK system of equations, the reverse Cuthill-McKee numberer, transformation constraints handler as recommended by Lu et al. [54], and the Krylov Newton algorithm [55]. The transient interval used the same solver parameters but performed the integration with the Newmark method with the average acceleration method ($\gamma = 0.5$, $\beta = 0.25$). Rayleigh damping was also applied with the mass and stiffness damping factors set to 0.05 and 0.005 respectively. Significantly less stiffness-proportional damping was applied because of the stiffness degradation expected at elevated temperatures. The step size was 0.0005 seconds, and the total analysis time was set to 1.08 second. The time scaling factor was 10,000 so that the 1.08 second analysis duration corresponded to 3 hours of fire time. Dynamic events were not crucial in this model as the thermomechanical response is expected to be pseudo-static during most of the fire.

3.3. Model 2: Load redistribution of floors 5 through 14

The second model sought to examine the overall structural behaviour and potential collapse propagation if a local failure was detected in Model 1. The fire scenario is the same as the first model, and the fire on the upper floors is ignored to isolate the effect of the mechanical room fire. The model, shown in Fig. 8, covered the same planar area as Model 1 but was extended up to floor 14 thus incorporating 9 floor slabs in the analysis. The columns up to floor 15 were included and loaded as per the structural drawings. The same boundary conditions as Model 1 were used: the column bases were assumed fixed, and the column tops allowed to translate vertically. Likewise, the western edge was fixed, and this did not affect the overall structural behaviour. The model had a total of 33,814 nodes and 34,484 elements. The mesh was relatively coarse with an element size of up to 1.5 m to allow the model to run at a manageable computational cost. This does not affect the overall behaviour of the structure as the scale of the overall model meant that the mesh was sufficient to accommodate the expected deflected shape.

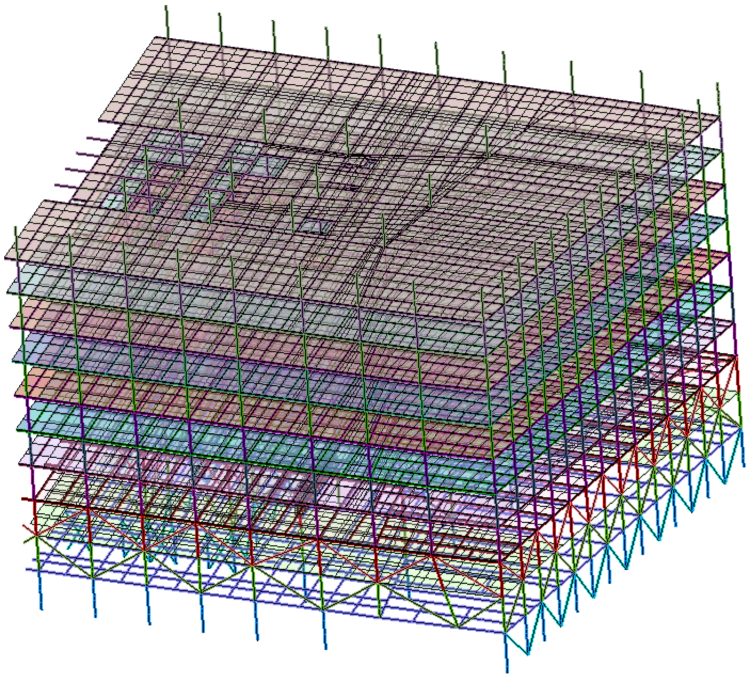

Fig. 8. Model 2 of WTC7 covering floors 5 through 14

The analysis was performed over three intervals: static loading, pseudo-static thermomechanical analysis, and dynamic collapse. The parameters of the analysis were unchanged from Model 1, except that the pseudo-static thermomechanical stage was run to cover only 1 hour of heating (0.36 seconds of analysis time), which was shown by the previous model to be sufficient time for collapse initiation. The last interval was allocated 5 seconds of analysis time (actual time; no time scaling).

4. Results and discussion

4.1. Thermomechanical response of Model 1

Fig. 9 shows the results of the static analysis of the WTC7 mechanical floors model subject to ambient loads (first interval). A maximum deflection of 45 mm was obtained in the northeast corner of the building as shown in Fig. 9 (a). This region was noted to utilise the largest pre-camber values of 57 mm to 63.5 mm which would completely account for the observed deflections. It is shown in Fig. 9 (b) that even under ambient loads some cracking is expected in the top concrete layers of the slab. This occurs over some regions where a negative moment is expected such as over the girders connecting column line 81, 80, 79, and the north perimeter. It is also shown that positive deflections are expected over the girders in the north of the floor due to the hanging loads of the north perimeter columns (see girders MG-27 in Fig. 1 (b)). Cracking is also noted on top of the large built-up girder (MG-23) at the location of Truss 2. This effect is likely an artefact of the large, concentrated load applied by column 78 at girder MG-23 over the slab.

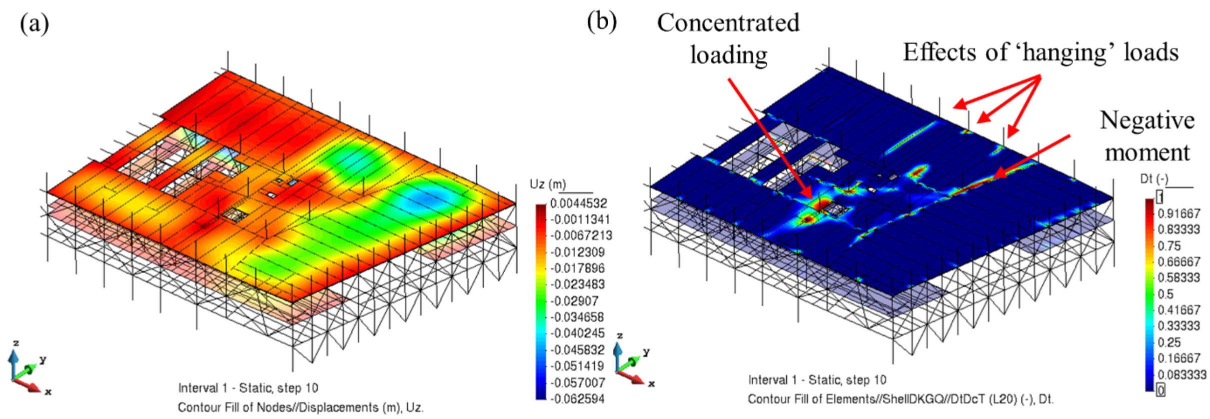

Fig. 9. Results from ambient analysis (a) deflections, and (b) Tensile damage in the top concrete layer

The average temperatures of the most critical members of Truss 2 are shown in Fig. 10, where truss member designations are clarified in Fig. 4. The average temperature histories shown correspond to heat transfer analysis performed in the ISE assuming hydrocarbon gas temperatures with a convective coefficient equal to 50 W/m^{2}K. Two temperature histories are shown for the diagonals assuming they were protected or unprotected. The temperature assuming 22 mm of SFRM, which is the same prescribed protection thickness for the 'heavy' columns of WTC7, produces very low temperatures even after 3 hours of heating. These temperatures are lower than the temperatures for column 77 and for the built-up girder MG-23 both of which were assumed to have been protected with 22 mm and 13 mm of SFRM respectively.

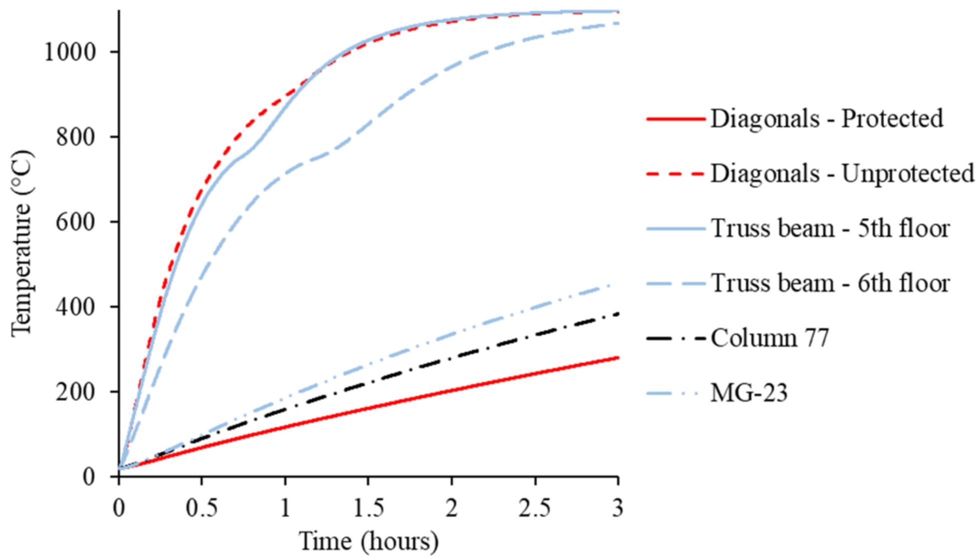

Fig. 10. Average section temperatures for the members of Truss 2 and for MG-23

While the diagonals used the same base section as column 77 (W14 × 730), they were additionally stiffened with two 200 mm × 75 mm steel plates on both sides of the web. This resulted in them being heavier than column 77 and thus explains the difference in temperatures between the protected diagonals and the column given the same exposure conditions and protection thickness. If the diagonals were assumed to have been left bare, then they would reach high temperatures rapidly despite their massive sections. The much smaller section size and thus high thermal thinness of the protected lateral beams of the truss explain their high temperatures, and likewise, the difference in section size and mass between the beams on the 5th and 6th floors explains the difference in their temperature. The 6th floor beam was about 3 times heavier than the 5th floor beam and thus reached lower temperatures.

The thermomechanical analysis was run twice: once assuming the diagonals were protected with 22 mm of SFRM, and once assuming they were left unprotected. With the protected diagonals, the transfer structure did not fail. Multiple unessential members, such as the lateral beams of the transfer structure, buckled. However, their failure did not compromise the overall stability of the structure. Maximum deflections reached during the heating phase of the fire were up to 0.97 m in the east of the mechanical floor. The axial force in the diagonal members against time is plotted in Fig. 11 (a), and the displacement of columns 77 and 78 are shown in Fig. 11 (b). The western diagonal axial force appears to unload, but no large out-of-plane deformations were recorded in it thus ensuring that it did not fail in buckling. This indicates that the reduction in its load is due to the longer eastern diagonal member taking on more of the load and thus effectively unloading it. The diagonals push column 77 upwards as seen in Fig. 11 (b) and thus the axial forces they are subjected to is limited by the vertical load of the column. The tip of column 78, which is supported by MG-23, deflects downward because the girder is thermally bowing due to its thermal gradient. The bottom flange of the built-up girder pushes against the connection between the hanger and column 77, which offers a significantly lower restraint than that provided at the seventh floor by the slab and floor framing, thus allowing the thermal gradient effects to manifest clearly.

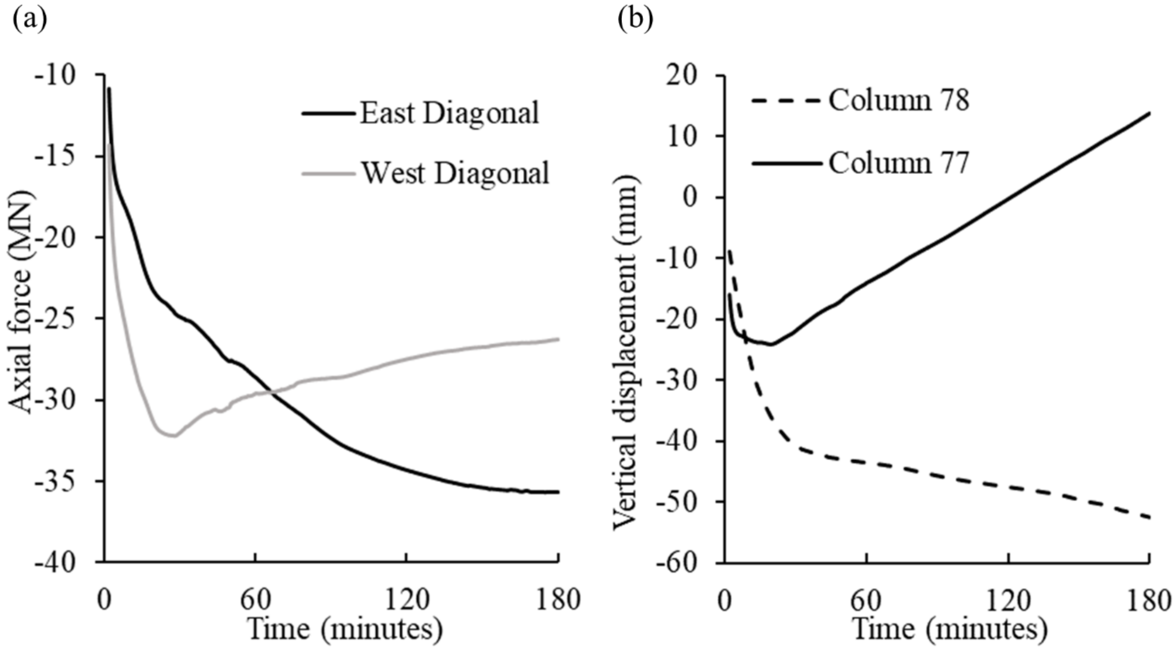

Fig. 11. Thermomechanical response of Truss 2 (a) axial force in the diagonal members, and (b) deflections of columns 77 and 78

Assuming the diagonal members were unprotected, their average temperature would rise to about 300 °C in 15 to 20 minutes, and about 1100 °C after three hours. Model 1 was analysed again under this condition, which results in the deflections shown in Fig. 12 (a), caused by the buckling of the diagonals as shown in Fig. 12 (b). The maximum displacement at the tip of column 77 was 1.59 m. It is noted from Fig. 12 (a) that the vertical deflections are skewed west (deflected surface approaches column line 80-81) but are otherwise centred around column 77 which lost its support. The fixed conditions at the western edge of the model do not affect the overall deflected shape as the thermally-induced deflections near the edge and away from the fire are negligible. The deflections in the east bay of the building were caused by the thermal loading which continued up to about 120 minutes. The thermomechanical analysis was terminated after two hours of heating because the transfer truss had failed resulting in a dynamic collapse event that will be analysed using Model 2.

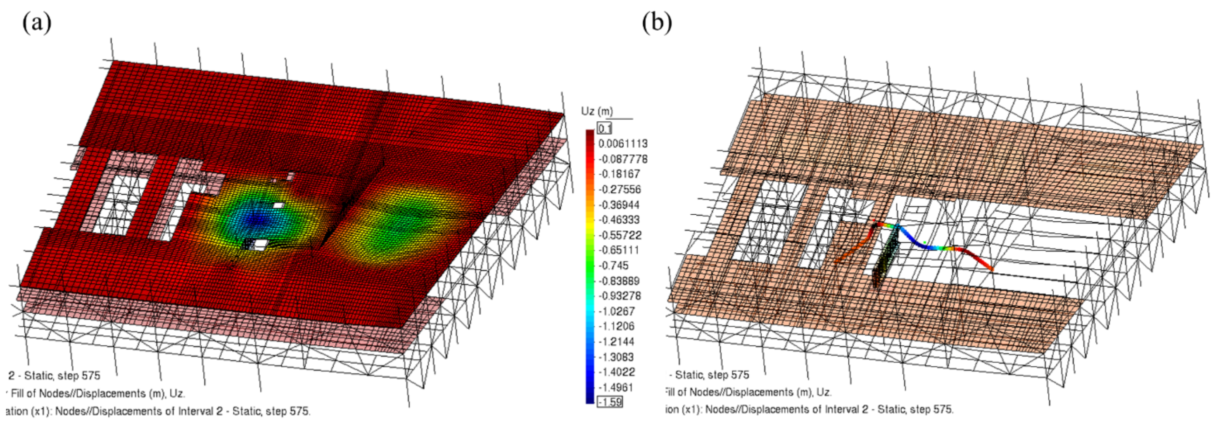

Fig. 12. Deflections due to Truss 2 failure (a) displacement in the slab of floor 7, and (b) buckled transfer structure

The buckling of the diagonal members of the transfer structure is indicated clearly by the axial force developed in them as shown in Fig. 13 (a). The axial force reached about 34 MN in the east diagonal after ten minutes and then was followed by buckling of the west diagonal member some 20 minutes later. Column 77 initially deflects upwards as shown in the vertical displacement of the column tops in Fig. 13 (b). Soon after, however, it began to fall rapidly reaching a maximum deflection of nearly 1.6 m in what is dynamically about 0.5 s but over 1.5 hours of fire pseudo-time. Column 78, partially supported by column 78A (see Fig. 4), had a lower deflection rate but also appears to have failed.

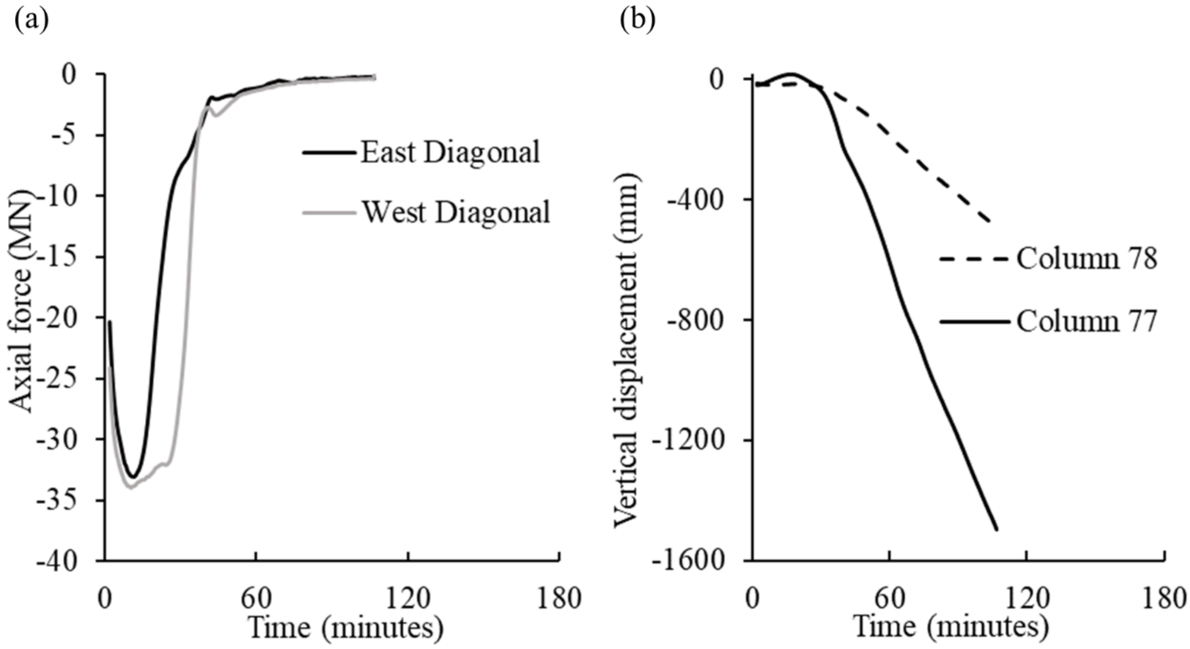

Fig. 13. Response of Truss 2 members and supported columns (a) axial forces in diagonal members, and (b) vertical displacement of columns 77 and 78

As the truss failed, the slab resisted the failure by adopting a compressive ring around the failed region as shown in Fig. 14 (a). Another compressive ring is formed about the heavily heated eastern bay. The concrete damage indices of the top layer of the slab presented in Fig. 14 (b) show that it has cracked within the heated region. Tensile damage also accumulated over the end of the girder MG-23 and towards the south edge of the building.

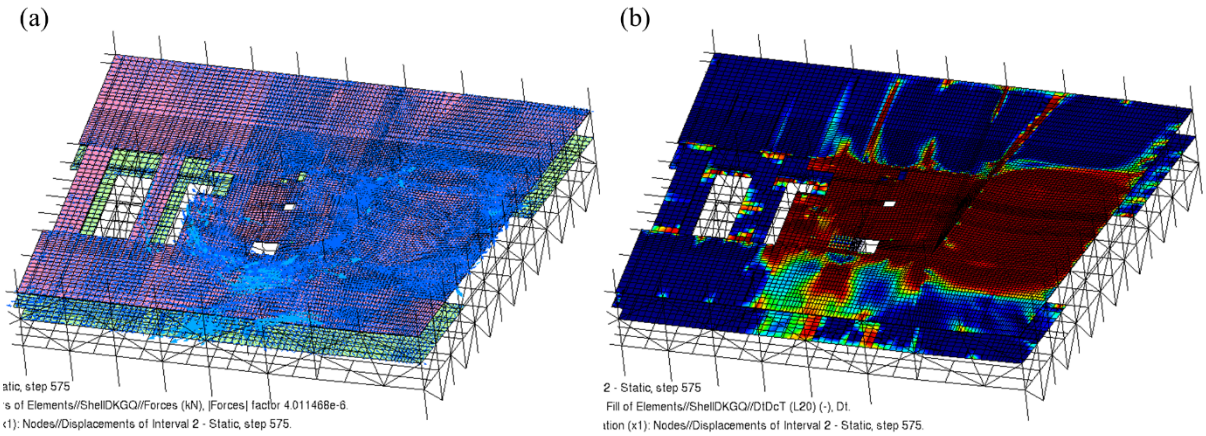

Fig. 14. response of the slab to the failure of Truss 2 (a) tractions, and (b) compressive damage in the top concrete layer

From the results of Model 1, it appears that the failure of Truss 2 is likely if the diagonals were not fire protected. This also points that this failure may also occur if the mechanical room fire burned for longer and thus allowed for higher temperatures in the diagonal members. From the current results, it appears that the collapse of the transfer structure would primarily destabilise columns 77 and 78 which it supports. Model 2 will investigate what the failure of Truss 2 may mean to the rest of the structure and is discussed next.

4.2. Failure propagation as predicted by Model 2

Model 2 has an additional 7 floors for a total of 9 floor slabs spanning floors 6 through 14. The load at the top of the columns is taken from the design drawings, and all floors are also loaded accordingly [16]. After the ambient loading, one of hour of hydrocarbon heating is applied to the same region as Model 1. This results in the diagonal members buckling and losing all capacity as was discussed in the previous section. The thermomechanical stage is followed by a dynamic analysis stage during which no additional thermomechanical load is applied. While the dynamic stage was originally set to analyse a dynamic time of 5 seconds, the analysis had to be terminated after 2.3 seconds were analysed. This was because the result files had reached 400 Gb in size, and the analysis had run continuously for 1 week. For the purposes of the preliminary analysis in this paper, the results for 2.3 seconds of dynamic collapse were sufficient.

The destabilisation of column 77 due to the failure of Truss 2 is resisted by the floor framing and slabs on all floors above. Despite this, the 17.5 MN load applied to the top of column 77 and loss of the main support provided by Truss 2 caused the deflections shown in Fig. 15 (a). The failure propagated from the failed transfer structure up to the 14th floor slab as the figure shows and caused floor deflections in the order of 0.8 m after 2 seconds. Naturally, the deflection of the floor is cantered over the failed transfer structure and columns 77 and 78. The large deflections in the core caused strain concentrations in the floor reinforcement bars over the east and north edges of the core. This is shown in Fig. 15 (b) which visualises the first principal strain in the J2 reinforcement layer.

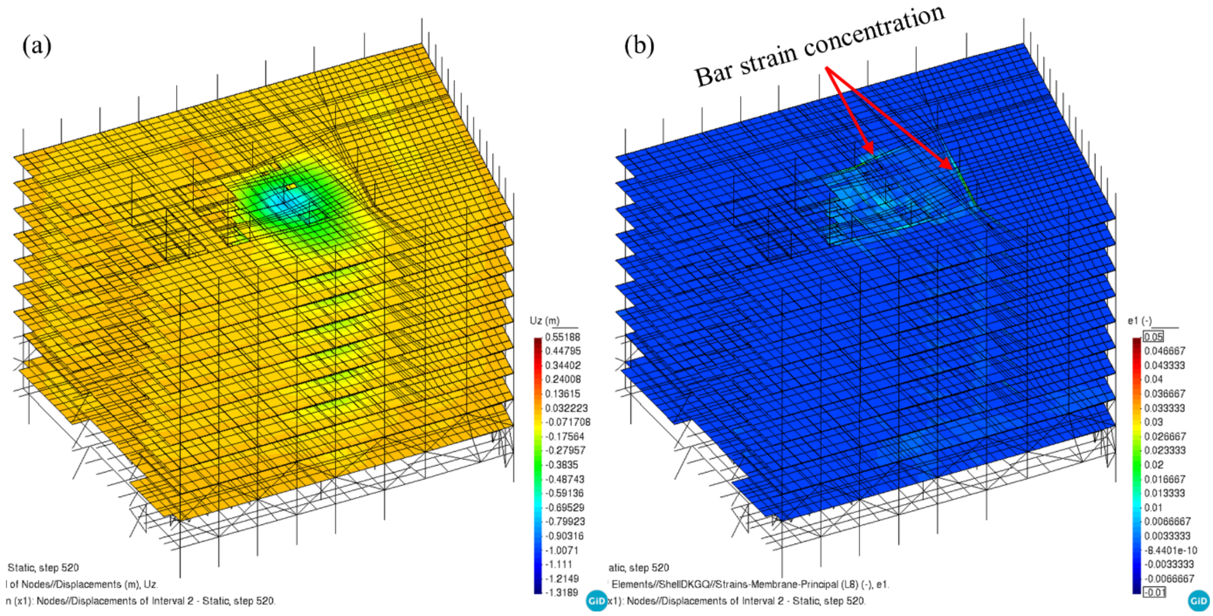

Fig. 15. Model 2 simulation results (a) vertical deflections, and (b) strain in the reinforcement layer

Likewise, the concrete of all floors cracks over the girder at the edge of the core, as well as within the region supported by columns 77 and 78. Fig. 16 (a) and (b) show the tensile cracking in the floors using the damage indices of the concrete damage plasticity material. The cracking of both the bottom and top concrete layers at the edge of the core, and the concentration of the reinforcement strain at this location indicates that the slab may be susceptible to failure at this location.

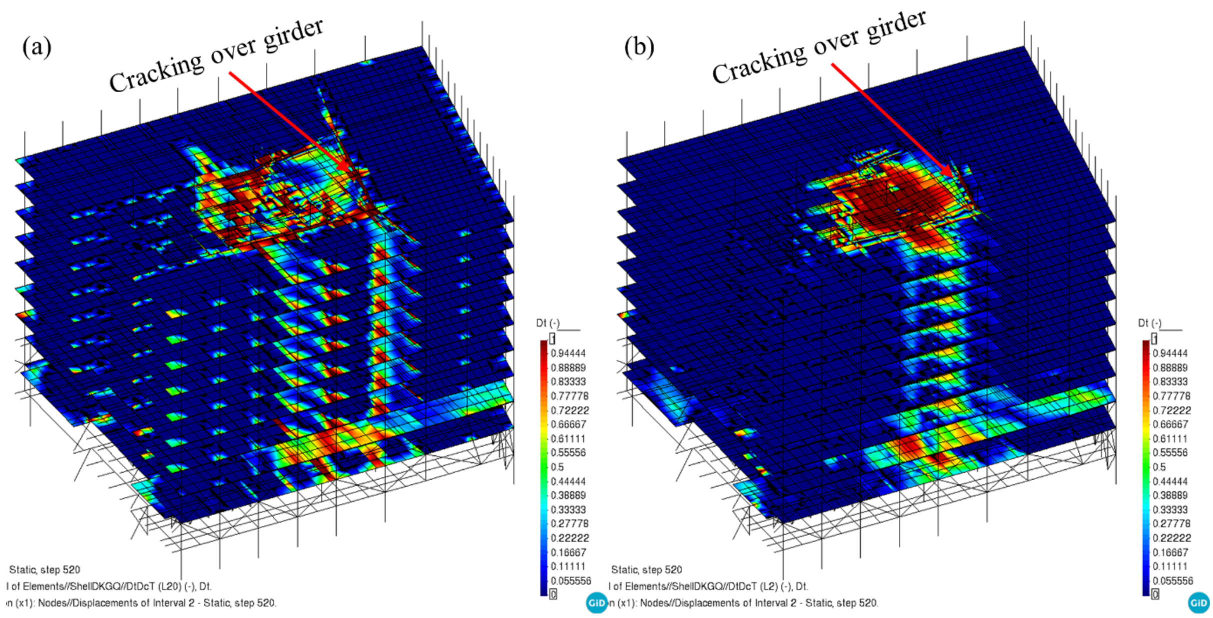

Fig. 16. Tensile damage in slab (a) top concrete layer, and (b) bottom concrete layer

4.3. Potential collapse mechanism

The failure of WTC7, as shown in Fig. 17, appears to have begun somewhere in the north-east part of its internal structure. This is commonly characterised by the distinct 'kink' that appeared in the edge of the east section of the penthouse on the roof as shown in the 3rd frame of Fig. 17. The eastern part of the penthouse completely sinks into the structure by the 6th frame followed by the remaining portion which has disappeared from view by the 8th frame. After the penthouse has completely sunk into the building, the remaining structure progressively collapses. Another kink is observed in the edge of the building as it failed and is clearly visible in the 9th frame. The entire collapse of the 47-storeyed tower happened in less than 14 seconds, and had begun somewhere in the lower floors [14].

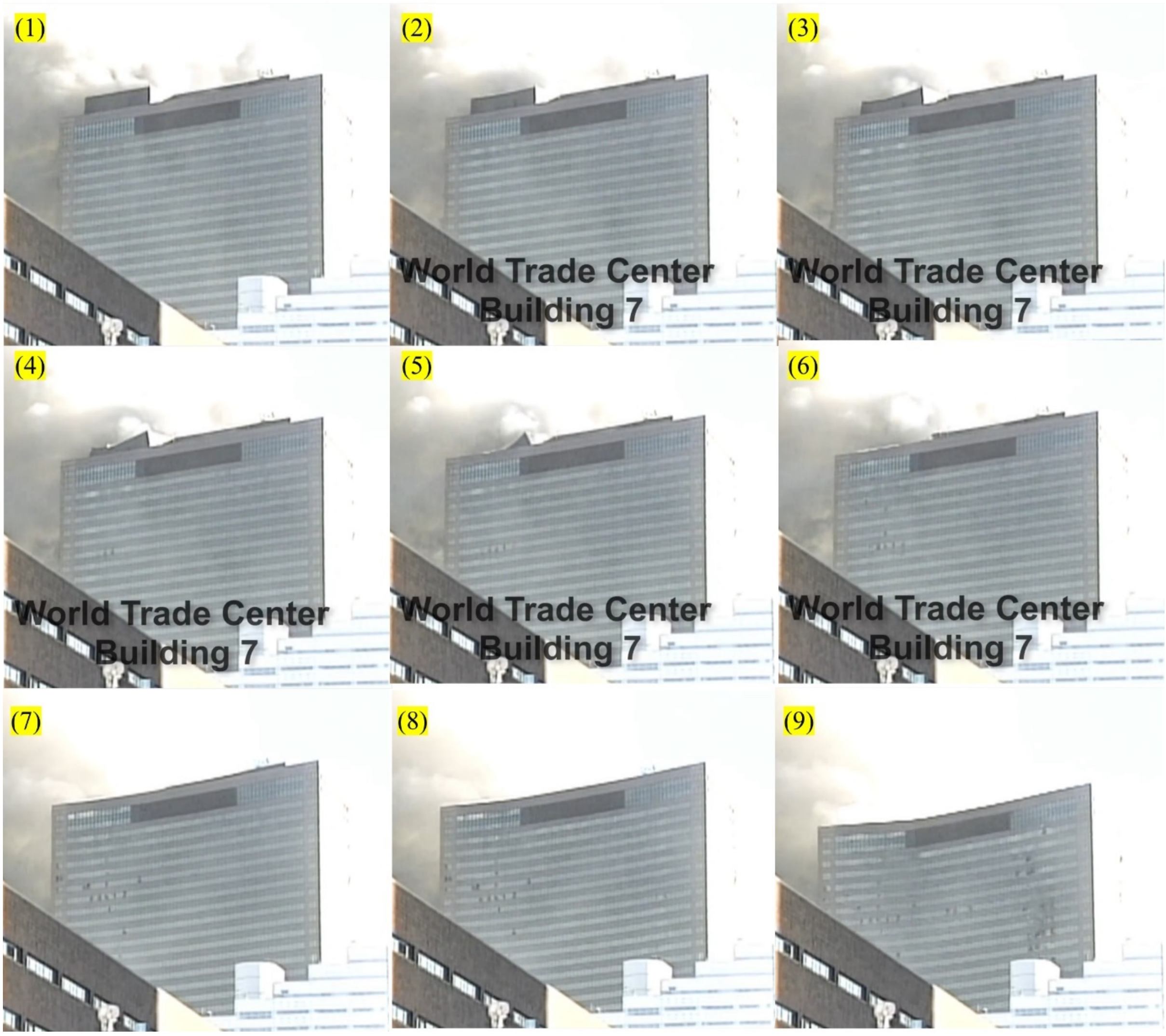

Fig. 17. Early stages of the collapse of WTC7. From NIST video [56] showing the North face of the building

Model 1 showed that an under-protected Truss 2 is susceptible to failure if a hydrocarbon fire had occurred within the 5th floor mechanical room in WTC7. This failure would destabilise columns 77 and 78, which would in turn affect the eastern part of the building core across all floors. The resulting rate of deflections predicted by Model 2 was approximately 0.4 m/s, which appears to be too low to solely explain the progressive failure of WTC7. This would, however, explain that the failure of Truss 2 and loss-of-strength that the core undergoes would not have an immediately visible effect on the outside of the structure.

Instead, the effect of the failure of Truss 2 would be to strain the reinforcement and rupture the concrete of the slab across column line 79-80-81 at the edge of the core. If the upper floors were unheated, then the effects of the collapse of Truss 2 may be either halted or localised. This especially likely given that there were 39 floors above Truss 2, each of which would bridge the forces to the surrounding intact columns thus resisting the failure. It is known, however, that severe fires had propagated on floors 7 through 9 and 11 through 13 on the day of the collapse. It is also argued by all investigators, and clearly shown by photographic evidence, that these fires were within the eastern part of the structure during the collapse. Such fires would have placed additional strains at the connections and slabs at both sides of column line 79-80-81, allowing the failure of Truss 2 to become a triggering event that would rupture the slab and sever these weakened connections. This would result in unbracing columns 79, 80, and 81 from the west. To the east, column line 79-80-81 would only be supported by a number of consecutively severely heated floors. These floors are subject to are incapable of preventing the buckling of the columns due to their thermal degradation and would contribute to column destabilisation because of the restrained thermal expansion of the slabs and framing.

The buckling would likely begin with columns 80 or 79 as they were carrying the heaviest load, and their failure starting on any of the fire floors would rapidly manifest as the kink seen on the east penthouse and shown in frame 3 of Fig. 17. Indeed, one of the few points of consensus among the investigation teams is that the failure of column line 79-80-81 would manifest itself as a kink in the penthouse structure. The initialisation of the failure to the west of the middle of the east penthouse caused by the failure of Truss 2 then results in the sinking in of the entire east penthouse as was clearly observed in frames 4 and 5 of Fig. 17. Without columns 79 through 81, and with the penthouse already sinking into the structure, the building begins to fail. The failure of column line 79-80-81 and the floors they once supported would nullify the damping effect of the upper floors that had resulted in the low deflection rate in Model 2. The absence of load-carrying capacity of columns 78 and 77 should then manifest in the collapse of WTC7 accompanied by a kink in the building edge, which was indeed clearly visible in frames 7 through 9 of Fig. 17.

Future analyses will need to be performed to further investigate this new collapse hypothesis. Currently, the open-source tools used in this study will need to be improved to handle large-scale simulations more efficiently. The analysis of Model 2, for example, had to be terminated early due to the size of the output file and the computational expense involved in running the model and processing the results. Planned future improvements of the ISE and OpenSees for Fire will allow for automatic static-dynamic analysis-type switching for the pseudo-static thermomechanical analysis [57], consideration of connection behaviour and failure, and connection and element removal. The deployment of OpenSees for fire on high performance computing (HPC) facilities is also underway.

Conclusions

This paper gave an overview of the collapse of WTC7, the investigations that ensued after it, and examined the hypothesis that a mechanical room fire may have contributed to the collapse. The three expert investigators believe that the failure initiated at column 79 or 80, but differed in the detail, mechanism, and cause of the failure. Building upon the fire investigation and FDS simulation by the BRE Centre for Fire Safety Engineering and the University of Edinburgh, this study assessed the response of WTC7 to a mechanical room fire.

Given that the fire is expected to reach steady state within 5 to 10 minutes of ignition, the hydrocarbon temperature-time curve was used to simplify the thermal exposure of the mechanical room structural elements. The heat transfer analysis was performed in OpenSees using the integrated simulation environment assuming a convective coefficient of 50 W/m^2K and both protected and unprotected diagonal members of Truss 2. It was shown that if the Truss 2 diagonal members were unprotected then their temperature would rapidly reach about 1100 °C despite their large mass. This temperature would lead to the failure of Truss 2 as explored in a pseudo-static thermomechanical model of the mechanical floors. The failure of Truss 2 causes columns 77 and 78 to collapse and forces the floor to resist the failure by adopting a compressive ring around the failed area. The deflections are centralised over the failed transfer structure, and the boundary condition on the west edge of the model does not seem to get in the way of the analysis. A second model with 7 additional floors was used to the assess the effect of the failure of Truss 2 on the overall structural behaviour. The model showed that under these conditions, the concrete and reinforcement of the slab at column line 79-80-81 are stressed. This allows for building upon the mechanical floor fire collapse hypothesis by considering that the multi-floor fires on the floors up to the 14th may result in unbracing the eastern-most core columns from the west. With only a heated floor on the eastern side, the columns would fail and produce the characteristic kink in the penthouse observed during the collapse. The collapse propagation would then follow, and another kink would form in the edge of the building since columns 77 and 78 had already failed.

Further analyses are needed in the future to examine this hypothesis in more detail. For this, however, the ISE and OpenSees for fire will need to be adjusted to deal with large simulation projects efficiently. The size of the output files needs to be reduced so that larger more sophisticated analyses can be run more efficiently without struggling with input/output bottlenecks. Furthermore, implementation of imperfect and failure-enabled connections, removable elements, and automatic analysis mode switching between static and dynamic analyses will improve the accuracy of the analysis models and the range of possible failure modes they can capture. Once OpenSees for Fire is successfully deployed on the HPC facility available to the research team, it will also be possible to produce larger and more sophisticated models for detailed analyses building upon the preliminary study detailed in this paper.

CRediT authorship contribution statement

Mhd Anwar Orabi: Conceptualization, Investigation, Writing-original draft, Formal analysis. **Liming Jiang:** Supervision, Writing-review & editing, Funding acquisition. **Asif Usmani:** Conceptualization, Supervision, Formal analysis, Investigation, Resources. **Jose Torero:** Conceptualization, Supervision, Formal analysis, Investigation.

Declaration of Competing Interest

The authors declare that there is no conflict of interest.

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