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ABSTRACT

Since the Ronan Point collapse in the UK in 1968, the progressive collapse analysis of residential buildings has gradually drawn the attention of civil engineers and the scientific community. Recent advances in computer science and the development of new numerical methodologies allow us to perform high-fidelity collapse simulations. This paper assesses different scenarios which could have hypothetically caused the collapse of the Champlain Tower South Condo in Surfside, Florida, in 2021, one of the most catastrophic progressive collapse events ever occurred. The collapse analysis was performed using the latest developments in the Applied Element Method. A high-fidelity numerical model of the building was developed according to the actual structural drawings. Several different collapse hypotheses were examined, considering both column failures and degradation scenarios. The analyses showed that the failure of deep beams at the pool deck level, directly connected to the perimeter columns of the building, could have led to the columns' failure and subsequent collapse of the eastern wing of the building. The simulated scenario highlights the different stages of the collapse sequence and appears to be consistent with what can be observed in the footage of the actual collapse. To improve the performance of the structure against progressive collapse, two modifications to the original design of the building were introduced. From the analyses, it was found that disconnecting the pool deck beam from the perimeter columns could have been effective in preventing the local collapse of the pool deck slab from propagating to the rest of the building. Moreover, these analyses indicate that enhancing the torsional strength and stiffness of the core could have prevented the collapse of the eastern part of the building, given the assumptions and initiation scenarios considered.

Keywords: Numerical Simulation, Progressive Collapse, Structural Failure, Applied Element Method.

PRATICAL APPLICATIONS

Building catastrophic collapses can cause significant lives and economic losses. Poor design and maintenance, in combination with aging, will more likely increase, in the next years, the number of buildings potentially vulnerable to the risk of collapse, due to either seismic, accidental, or degradation actions. This research focuses on the analysis of the Champlain Tower South condo collapse, which occurred in the City of Surfside in 2021. Different hypothetical collapse scenarios were simulated, comparing the analysis results with the actual evidence of the collapse. The analyses have shown that the degradation of the pool deck slab, due to corrosion, may have contributed to the collapse of the building. Finally, two different minor revisions of the original design of the building were analyzed to reduce the risk of failure and understand how the collapse of similar residential buildings could be prevented.

INTRODUCTION

Over the last decades, the number of publications on themes related to the progressive collapse of buildings has exponentially increased (Gerasimidis and Ellingwood 2023). The attention to the disproportionate effect of a local failure dates back to 1970, when the first regulation related to accidental load was introduced in the UK code, as a consequence of the partial collapse of the Ronan Point building in London (Vrouwenvelder 2021). However, it was after the tragic terroristic attack on the World Trade Center in 2001 that the progressive collapse of structures captured the interest of the academic community (Lalkovski and Starossek 2022). Several definitions of Progressive Collapse were proposed by different authors over the last decades; ASCE 7-05 (ASCE 2005) defines progressive collapse as "*the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it"*. Three common points can be identified among the different proposed definitions: the initial failure is local, the failure spreads to other structural members, and the final collapse is disproportionate to the initial failure (Kiakojouri et al. 2021). *In the last decades, several countries introduced specific regulations to address the risk of progressive collapse.* In the US, the General Service Administration (GSA) code (GSA 2013) was developed for government buildings, while the UFC 4-023-03 code (DoD 2016) was introduced for military buildings. In Europe, Annex A in Eurocode 1 was introduced accounting for the first time for Accidental Actions (CEN 2006).

In contrast with the seismic design of structures, which is largely addressed in worldwide regulations through a more prescriptive code compliance approach, progressive collapse design often requires a performance-based approach, by considering a series of "what if" scenarios (Fiorillo and Ghosn 2022).

Given that the objective of progressive collapse design is to ensure that a structure can withstand a certain level of local damage and avoid collapse propagation, it is understandable how the prediction of the initial damage effects and its possible propagation can be crucial as well as numerically challenging. Over the past decade, several researchers working on progressive collapse design suggested the introduction of robustness indexes. They can either be based on analytical or simplified numerical approaches, such as alternate load-path methods and push-down analyses (Praxedes and Yuan 2021). While both these approaches can be effective in assessing the risk of progressive collapse of relatively symmetric and homogenous structural systems, the progressive design of complex structural systems may require a more advanced methodology, such as the creation of high-fidelity numerical models (Sadek et al. 2022). While this approach was considered prohibitive in the past, because of the required computational effort, non-linear dynamic analyses of high-fidelity numerical models are now feasible thanks to the latest advancements in hardware computational capabilities and numerical methodologies (Stylianidis and Nethercot 2021) (Le and Bazant 2022). Among the numerical approaches to progressive collapse analysis, Finite Element Method, FEM, is widely adopted in several published studies. The FEM method can be efficiently used in progressive collapse analysis of frame structures, especially in code-based procedures (Kiakojouri et al. 2020). However, because the FEM solver is based on equilibrium equations, the solution cannot automatically implement element separations. Because of that, the capability to simulate the entire collapse of the structure is limited. Nevertheless, several strategies were developed in recent years to overcome FEM limitations in the analysis of large displacement problems. For example, the smeared crack technique was developed to allow for crack propagation in FEM analyses (Petrangeli and Ožbolt 1996). FEM application to progressive collapse analysis of entire structures often considers bi-dimensional frame elements to reduce the computational burden (Alashker et al. 2011). However, researchers also developed a component-level, and multi-scale models approach assuming the refined 3D modeling of only a portion of the structure (Li and Hao 2013), (Mpidi Bita et al. 2022). Lastly, the recent development of FEM coupling methodology (Lu et al. 2009), and refined numerical procedure for element removal, such as the degree-of-freedom (DOF) release (Xu et al. 2018), overcome the FEM limitations to progressive collapse simulation.

The Discrete Element Method, DEM, was also employed in progressive collapse analysis (Lu et al. 2018). Based on the compatibility of displacement, the DEM solver can account for element separation and rigid body collision (Hakuno and Meguro 1993); however, DEM requires large computational efforts, in particular when dealing with a comprehensive numerical model of the entire structure. To reduce analysis time and increase the accuracy of the results, several FEM-DEM methodologies were also developed over the years (Lu et al. 2009).

Among the numerical methodologies for structural analysis, the Applied Element Method (AEM) is considered one of the most efficient numerical approaches to collapse analysis and simulation (Grunwald et al. 2018). The methodology can automatically account for the formation of plastic hinges, development, and propagation of cracks, 3D load redistribution, as well as yielding and failure of reinforcing bars until element separations occur (Domaneschi et al. 2020).

This work focuses on the progressive collapse analysis of the Champlain Towers South condominium using the AEM method. The 2021 collapse of the Champlain Towers (Surfside, Florida) was one of the most catastrophic collapses that ever occurred to reinforced concrete (RC) residential buildings. Built-in 1982 as a part of a three-building complex, namely the Champlain Towers North, South, and East, the Champlain Towers South consisted of an L-shaped, twelve-story RC structure with flat slabs and a basement floor covering the entire footprint of the building area. What makes this event particularly interesting from the point of view of progressive collapse analysis, is that the evidence infers that the collapse was caused by a localized failure of a singular structural element. Specifically, the failure of a slab due to punching shear would spread to the center of the building first, and then to the eastern wing a few seconds later (Lu et al. 2021).

To simulate collapse scenarios and investigate the behavior of the building, a high-fidelity AEM numerical model was developed. Several sensitivity analyses and different collapse scenarios were replicated to study the collapse behavior of the building and evaluate the most probable reason for its collapse. Finally, the progressive collapse performance of the structure was enhanced by introducing two different modifications to the original design, which could have prevented the collapse, under the studied hypotheses.

It should be noted that the causes of the collapse are currently unknown and a comprehensive failure investigation by an agency of the US government is underway to provide a definitive answer as to its causes. The present work is based only on publicly available material, which mostly refers to the original drawings of the structure without considering eventual discrepancies in the final realization of the building. In addition, the analyses presented in this work are based on assumed loads, and degradation conditions which have not been verified.

THE APPLIED ELEMENT METHOD

In the AEM, the structure is discretized in a series of six-degree-of-freedom rigid eight-node elements connected by zero-volume springs representing both the linear and non-linear behavior of the constitutive material (Fig. 1, a). The interface springs, uniformly distributed along the element's surfaces, describe stresses and deformation of a certain volume δV. A geometrical relation is determined between the centroid of the eight-node element and the contact point in which the surface spring is located (Fig. 1, b). The axial stiffness, (k_n) and shear stiffnesses, ($k_{s,1}$, $k_{s,2}$) of the interface springs are determined based on the given elastic moduli, E and G, and the area (d·t) and length (l) of the represented *i*-volume, as per the following equations:

$$k_n = Edt/l; \; k_{s,1,2} = Gdt/l \qquad \text{Eq. 1}$$

More details about the methodology can be found in Tagel-Din and Meguro (2000).

In the AEM approach to the analysis of RC structures, the mechanical behavior of the concrete material is represented by a series of springs distributed along the interface between the two elements (Fig. 1, c). The contribution of steel rebars embedded in the material can be explicitly accounted for by coupling the mechanical contribution of additional springs representing the steel reinforcement. The steel springs are placed in their actual position in the cross-section of the considered structural element (Fig. 1, d). As the springs consider the axial stiffness k_n, and the shear stiffnesses $k_{s,1}$ and $k_{s,2}$, the contribution of both longitudinal and transversal reinforcing bars, for the given constitutive laws, is automatically accounted for in the numerical analysis. In this study, the Maekawa and Okamura (1985) model is considered for representing the axial behavior of concrete (Fig. 1, e), while a linear relationship up to failure is assumed for the behavior of concrete subject to combined shear and compressive loads (Fig. 1, f). Finally, the Menegotto and Pinto (1973) model is adopted for representing the nonlinear behavior of steel reinforcement (Fig. 1, g).

Fig. 1. AEM discretization approach of RC assemblies and the corresponding constitutive laws for concrete and steel.

The commercial software Extreme Loading for Structures (ELS) developed by Applied Science International (ASI) was employed in the present study (ASI, 2021).

THE AEM NUMERICAL MODEL OF THE CHAMPLAIN TOWERS CONDO

Structure description and material properties

The Champlain Towers South structure consists of RC flat slabs supported by RC columns. The thickness of the slab is 23 cm (9") on the basement floor, 24 cm (9 ½") at the Lobby level, and 20cm (8") for typical floors. Different concrete compressive strengths were considered in the design of the building: columns and shear walls were designed with strength varying from 41 MPa (6000 psi) to 28 MPa (4000 psi), while the slabs were designed with compressive strength varying from 28 MPa (4000 psi) to 21 MPa (3000 psi), (Fig. 2, a). The longitudinal reinforcement of columns is varying from size Ø36 mm (#11) at the lower floors to Ø25 mm (#8) at the upper floors. The reinforcement of the two shear walls includes two columns at each edge and a reinforcement Ø13 mm (#4) mesh, spaced at 30 cm (12"). Ø13 mm (#4) stirrups were used for Ø36 mm (#11) longitudinal reinforcement while Ø10 mm (#3) stirrups were used for the rest of the bar sizes, (Fig. 2, b).

Fig. 2. Color map of concrete strength in columns, shear walls, and slabs (a) [MPa *(ksi)*], and diameter of reinforcement bars implemented in the numerical model (b) [mm]

Table 1 shows the concrete properties considered for the AEM numerical model.

Table 1. Concrete material properties introduced in the AEM numerical model [Stresses in MPa (*ksi)*, Elastic Modulus in GPa (*Mpsi)*]

The bottom reinforcement of the flat slab consists of a uniform rebar mesh of Ø13 mm (#4) spaced at 30 cm (12") in the basement and Lobby floors, and 33 cm (13") at the 2^{nd} and typical floors. The punching shear reinforcement at the top side of the slab consists of Ø16 mm (#5) rebars with variable spacing. The area covered by the punching shear reinforcement also varies based on the column's section and location. Rebars having a diameter of Ø13 mm (#4) and different spacings were provided, in one direction only, at the top side of the slabs, in the transition zones between the areas covered with punching shear reinforcement (Fig. 3).

Fig. 3. AEM numerical model view of the punching shear reinforcement in the lobby slab, basement, 2nd floor, and typical floor.

RC girders can be found in the Lobby and 2^{nd} floor only. On the Lobby floor, 30cm (12") width girders with various depths are connecting the Lobby RC slabs at different elevations (also referenced as "slab-drops" in the original drawings of the structure). On the 2^{nd} floor, 91x107 cm

(36”x 42”) transfer girders are supporting 30,5x61cm (12”x24”) columns elevating from the 2^{nd} floor to the roof.

As specified in the as-built drawings notes (William M. Friedman & Associates Architects 1979), reinforcing bars meet ASTM A-615 Grade 60 criteria, with yield strength equal to 414 MPa (*60 ksi*), (Table 2).

Table 2. Steel material properties introduced in the AEM numerical model [Stresses in MPa (*ksi)*, Elastic Modulus in GPa (*Mpsi)*]

The final developed model, employing 5 matrix springs per element’s face, resulted in 7.5 million matrix springs representing the different concrete materials and additional 0.85 million equivalent springs representing the different reinforcement for more than 900,000 degrees of freedom.

The non-linear dynamic analyses were performed considering a time step equal to 0.001 s, using a 3.5 GHz 12 cores processor and requesting approximately 30 Gb of memory. With the given hardware, the AEM solver produced the analysis output of 1 sec in approximately 3 hours of calculations, resulting in overall 48 hours needed to complete one entire collapse simulation of the duration of approximately 16 sec.

Loads

The dead load of the structural elements explicitly introduced in the numerical model is automatically accounted for in the analysis based on the volume and density of the concrete. In addition, the weight of non-bearing walls, finishes, furniture, and any other elements not directly introduced in the numerical model was assumed as distributed on the floor area. As this work aims to compare the numerical results with the actual evidence of the Champlain Tower South collapse, no code-based load combinations are considered in the analysis. In fact, with respect to the DoD and GSA provisions, in which a factor of 1/2 is applied to the prescribed Live Load, LL, only a fraction of 1/4LL is assumed to be in place at the moment of the collapse. The assumption is consistent with

ASCE 7-22 Commentary Table C4.3-2, which suggests a mean sustained Live Load of 0.3kN/m^2 *(≈6 lb/ft^2*), (ASCE 2021). In addition, because of uncertainties on apartment' finishes, walls and ceilings composition and materials, and overall actual loads at the moment of the collapse, sensitivity analyses were carried out with different loading assumptions, considering a cumulative distributed load (DL+LL) varying from 1.5 kN/m^2 (≈*30 lb/ft^2*) to 3.0 kN/m^2 (≈*60 lb/ft^2*). In this work, only the analyses with the load assumptions reported in Table 3 are considered for the sake of brevity.

Table 3. Loads [kN/m^2 (*lb/ft^2)*]

For the Typical Floor, the following loads were assumed based on the typical weights of the building materials (Breyer et al. 2020): dead load in addition to slab self-weight, 1 kN/m^2 (*20lb/ft^2*), accounting for the floor finishes, ceilings, façade elements, windows, doors, railings, MEP systems, and any additional load not explicitly introduced in the model; dead load of walls & partitions, 0.5 kN/m^2 (*10lb/ft^2*); live load, 0.5 kN/m^2 (*10lb/ft^2*); the total considered distributed load results 2 kN/m^2 (*40lb/ft^2*).

In addition to the distributed load, an ornamental plant load, estimated on a soil density equal to 16 kN/m^2 (≈*100 lb/ft^3*), was introduced in the numerical model at the pool deck level based on the actual plant arrangements at the time of the collapse.

NON-LINEAR DYNAMIC ANALYSES AND COLLAPSE SCENARIOS

Two of the most credited hypotheses raised by media in the aftermath of the collapse of the Champlain Towers South attribute the cause to either differential settlement in the foundations or localized structural failure. In the first stage of this work, several column removal scenarios were carried out to evaluate the sensitivity of the structure to column failure and its consequent load redistribution capacity.

Column removal scenarios

Column removal scenarios were implemented at the locations where the initial failure was observed, considering both perimeter and inner column removal scenarios. To simulate a hypothetical foundation settlement, columns are removed at the foundation pile level, below the basement slab. Thus, the basement slab contributes to the load redistribution till punching shear failure occurs. The column's removal is performed using non-linear static analysis, so the overload determined by the column's loss is redistributed incrementally to the surrounding structural elements. The two considered scenarios, loss of center columns and loss of perimeter columns, were defined to identify the most probable area where the initial failure occurred. Each of the two scenarios was repeated considering the loss of one column first, and an adjacent one after, keeping removing columns till collapse is reached.

The column removal analyses revealed that the building was more sensitive to the removal of perimeter columns (Fig. 4, scenarios C and D) rather than inner columns (Fig. 4, scenarios A and B). In fact, under the loading assumption and considering the original properties of steel and concrete, without accounting for material degradation, the building was able to redistribute the loads and avoid progressive collapse, even when three inner columns were removed (Fig. 4, Scenario B). Nevertheless, the removal of two perimeter columns was enough to initiate the progressive collapse of the building (Fig. 4, Scenario D).

Fig. 4. Inner (top) and perimeter (bottom) column removal scenario, Vertical deflection.

A load sensitivity analysis was also carried out showing that the scenario of perimeter column removal remains the most critical one regardless of the entity of load.

Localized degradation scenario

Several media discussed evidence of extensive degradation of the pool deck slab in the immediate aftermath of the collapse. Indeed, the area was partially covered by ornamental plants, which, on top of the additional weight, might also have caused corrosion of the slab steel reinforcement due to watering and lack of proper impermeabilization. Static analyses were performed considering only the vertical dead load, as per load assumptions, without accounting for any degradation. These analyses showed that the area of the pool deck was substantially weaker and subjected to higher deflections and stresses than the area within the twelve-story building footprint (Fig. 5, a). For example, the deflection in the pool deck area reaches 2cm, while it resulted in only fractions of a centimeter in the area pertaining to the twelve-story building itself. The larger spans, together with the limited reinforcement, led to stresses in the punching shear reinforcement substantially higher than in the rest of the structure. The mean value of normal stresses in the punching shear reinforcing bars on the ground floor of the twelve-story building was found to be below 100MPa (≈15ksi). In the pool deck area, the normal stresses in the punching shear reinforcing bars reached 200MPa (≈30ksi), which corresponds to half of the yield stress of the steel, 414MPa (60ksi), according to the original design specification (William M. Friedman & Associates Architects 1979). In particular, the area where the initial collapse occurred, showed the highest stresses, specifically at the top side rebars of the pool deck slab (Fig. 5, b).

Fig. 5. Static analysis, Vertical displacement at basement level [cm (*in*)] (a), and normal stresses in top punching shear reinforcement [MPa (*Ksi*)] (b)

To investigate the hypothesis of the pool deck slab degradation, further analyses were performed considering localized steel degradation in the pool deck area (Fig. 6).

Fig. 6. Building plan at basement level with the indication of the twelve-story building's footprint and the degraded area of the pool deck level

Degradation analyses were carried out by introducing a progressive area reduction of the slab and beams reinforcement, up to 90%, until collapse is reached. This degradation analysis approach is widely adopted in the literature. For instance, it was used to analyze the collapse of the Polcevera Viaduct in Genoa, Italy (Domaneschi et al. 2020).

Fig. 7 shows a comparison between the crack distribution resulting from the two considered scenarios, foundation settlement in the center of the building (Fig. 7, a) and degradation of the pool deck slab (Fig. 7, b).

Fig. 7. Lobby level, comparison between cracks distribution in case of foundation settlement scenario (left) and pool deck degradation scenario (right), Principal strains in Dir.1 [-]

Cracks are shown based on the plot of principal normal strains, varying from ε=0.0 to ε=0.1. Considering a mesh dimension equal to approximately 30cm (≈1ft), it corresponds to a maximum crack opening of about 3cm (≈1in). It can be noticed how a diffuse foundation settlement, which should have involved at least four columns to result in the disproportionate collapse of the building (see "Columns removal scenarios"), would have caused widespread cracking and concrete spalling at the basement level that by far exceeds what is described in the reports about the structure (Fig. 7, a). Evidence of linear cracking at the Pool deck slab, outside the actual footprint of the building, was instead reported by some media and found in the degradation analysis scenario also (Fig. 7, b).

In addition, it was noticed that, when applying the degradation to both the spans pertaining to the twelve-story building and the pool deck slab, the pool deck slab area would have shown much more diffuse evidence of cracks compared to the twelve-story building counterpart, because of the inherent lower residual capacity, deriving from ornamental plant superimposed load and larger spans.

Another aspect worth noticing is that the pool deck structure, designed to carry only one floor rather than twelve stories, while also subject to additional superimposed loads and deterioration, was rigidly connected to the main structure through three beams with a depth of 46cm (*18"*).

These beams were generally used at the pool deck slab level to cover for different elevations and steps, and in fact, called "slab drops".

The three previously mentioned "slab drops" were originally designed to be 59cm (*23"*), and then reduced to 46cm (*18"*) in a second design revision (William M. Friedman & Associates Architects 1979). The degradation analysis of the pool deck slab shows how the depth of the three girders, resulting from the slab drops, could have played a significant role in propagating the collapse of the slab to the rest of the building. Because the building was particularly sensitive to the loss of perimeter columns, when the slab and connecting beams fail, a concentrated bending moment is transferred to the three perimeter columns, leading to column overload and consequent collapse of the building (Fig. 8).

Fig. 8. Effect of deep beams in causing the instability of the perimeter columns at 2.0s, Principal strains in Dir.1, Scale color red equal to 0.1 Strain [-], and deformed shape scaled by a factor of 2 (left)

Arch and catenary actions: from the failure of the pool deck slab to the disproportionate collapse of the building

This section describes the identified collapse mechanism, starting from the deck failure, followed by the formation of an arch action and subsequent catenary action, until reaching the failure of the perimeter columns.

Fig. 9 shows a time-lapse of the failure at different seconds, describing both the variation of compressive stresses in the concrete, normal stresses in the reinforcing bars, and the variation of internal forces in columns and beams, as the failure progresses. The different instants are identified in the timeline at the bottom of the Figure. The compressive stresses in the concrete are shown in the top row; Also, the stresses in the reinforcing bars are shown in the middle row, together with the related chromatic scale; the compressive stresses in the Finally, the internal forces, bending moment, M, (positive in red and negative in blue), and normal force, N, (in magenta) are shown on the bottom row.

Fig. 9. Column 11.1-L, Compressive stresses in the concrete [MPa *(Ksi)*] (top), normal stresses in beams' and columns' reinforcement [MPa *(Ksi)*], normal forces [kN *(kips)*] in the column and bending moments in the beams [kN*m *(kips*inch)*] (bottom), at different stages of the collapse.

When degradation is introduced, steel rebars start yielding. The slab starts deflecting downwards and the concrete in the perimeter columns reaches its maximum compressive strength at the connection with the pool deck beams (Fig. 9, 1.0s). After the concrete fails, as a consequence of the yielding of the longitudinal reinforcement in the pool deck beams, the bending moment in the column increases till reaching the ultimate capacity of the section for the given combination of axial forces and moment (Fig. 9, 1.3s). At this point the column loses its load-bearing capacity, activating an initial arch action, as can be gathered from the increase of compressive stresses at the top of the perimeter beams (Fig. 9, 1.5s). Consequently, the column-beam connection fails, generating a catenary action in the perimeter beams that results in both top and bottom longitudinal reinforcement subjected to tensile stresses (Fig. 9, 2.0s). At this point, the original degradation introduced in the slab has progressed through slab failure and subsequent column failure, with the only catenary action opposing the propagation of the collapse. Unfortunately, the amount of reinforcement in the beams is not enough to withstand the catenary forces, ultimately leading to the progressive collapse of the building (Fig. 10).

Fig. 10. Distribution of Principal Strains [-] after column' failure (left), and punching shear failure at pool deck slab (right)

Side-by-side comparison of the collapse

Fig. 11 shows a side-by-side comparison between the simulation and the footage of the collapse (Slater 2021). The comparison shows a good agreement between the simulations and the actual collapse, both during the initial failure of the central portion of the building (between Axes G and M, Fig. 6), and the initial torsion of the remaining eastern portion of the building, a few instants later. However, a flexural failure at mid-height of the eastern wing of the structure can be observed only in the numerical analysis, occurring when the remaining portion of the structure starts hitting the ground (frame T7 in Fig.11).

Fig. 11. Side-by-side comparison between the footage of the collapse and the numerical analysis in nine different timestamps; Image courtesy of Slater (2021).

This failure, which takes place approximatively at the middle height of the structure, results in more than half of the eastern core of the building leaning toward the east in the final debris distribution obtained from the numerical analysis. However, this mechanism is not observed in the actual video of the failure, where the eastern wing collapses in on itself, leaning toward the west. The difference in the observed collapse behavior could be explained by a possible divergence of the mechanical properties of the materials, due to either degradation or construction defects, which are not considered in the model.

Fig. 12 shows a comparison between the reconstruction of the actual debris distribution resulting after the collapse and the debris resulting from the analysis.

Fig. 12. Comparison between actual debris distribution (a) and analysis results (b); Image (a) reconstructed by the Authors based on available media pictures.

In both cases, the collapse of the building did not spread over the west core of the structure. The initial torsion of the eastern wing is also captured by the numerical analysis, as it can be gathered from the orientation of the slabs, pointing towards the south side. However, because of the flexural failure mechanism described early, which was observed in the numerical simulation only, the final position of the east shear wall core differs, with the half-eastern core leaning east rather than west.

PROGRESSIVE COLLAPSE ANALYSIS UNDER ALTERNATIVE DESIGN SCENARIOS

Investigating how the failure propagation could have been prevented, could help avoid similar catastrophic events in the future. To avoid the disproportionate collapse of the building, two alternative designs were defined.

The first alternative design scenario considers that the pool deck slab is not connected to the main structure through deep beams. Thus, the deep beams connecting the slab to the perimeter columns were removed (Fig. 13, a, b). As it was previously noted in this work, the three deep beams were provided in the design as “slab drop”, to cover the different elevations at the pool deck area.

The slab reinforcement in this area was the same as the surrounding spans, having similar span lengths and loads but with no deep beams. Therefore, the scenario only pertains to the removal of the "slab drops" and related differences in elevation, without modifications to the original design of the slabs.

In this configuration, even when assuming high degradation of the pool deck slab, perimeter beams, and columns, the failure of the slab did not affect the rest of the structure (Fig. 13, c). Because the slab has a substantially smaller depth compared to one of the deep beams, the bending moment induced to the column joint is considerably smaller. Therefore, when the slab fails, punching shear failure occurs and the slab detaches from the perimeter columns without compromising their load-bearing capacity (Fig. 13, d).

Fig. 13. Original design (a), and alternative scenario (b), assuming the absence of beams connecting perpendicularly the pool deck to the perimeter column of the structure, Analysis results, Vertical displacement (c) [cm (*in*)], and Principal strains at slab failure (d) [-]

In terms of progressive collapse design, it is clear how the structural separation between the main twelve-story building and the secondary structure (i.e., one-story basement), could be an effective strategy in avoiding collapse propagation, especially when the secondary structure is naturally subject to deterioration (e.g., terraces, pool decks, etc.).

The second alternative design scenario considers the introduction of additional shear walls in the East core of the building (Fig. 14, a, c). The objective is to reduce the torsional behavior observed during the collapse of the eastern wing of the building and avoid its failure (Fig. 14, e).

The reason why, after the collapse of the central portion of the building, the collapse propagated to the eastern wing only, is related to the different torsional stiffness resulting from the structural layout. While the West core of the building consisted of RC shear walls oriented both in the North-South and East-West directions, the East RC core of the structure was composed of only one shear wall oriented in the North-South direction. Therefore, the torsional capacity of the western core was significantly larger than the one of the eastern core. When the central block of the building starts failing, it was observed that slabs were detaching from the western and eastern wings at the interface of the shear wall. Afterward, the eastern wing fails, due to the lack of torsional capacity. In light of that, two additional shear walls were introduced in the East-West direction (Fig. 14, c). Results show that the change in the design is effective in avoiding the collapse of the eastern wing (Fig. 14, d, f). The difference in terms of rotation observed both in the original model and in the new design configuration can be evaluated from, Fig. 14, e, f.

Fig. 14. Original design (left), and alternative scenario (right), assuming the presence of two additional shear walls

In terms of progressive collapse design, these results highlighted that providing RC cores with enough torsional capacity can be effective in preventing collapse propagation between different parts of the structure.

CONCLUSIONS

In this work, a series of progressive collapse analyses were performed to analyze the collapse of the Champlain Towers South in 2021 (Surfside, Florida). An AEM high-fidelity numerical model was developed and employed to investigate the possible causes of the collapse. Different column removal scenarios were modeled to simulate foundation settlement, while structural degradation was modeled progressively reducing the reinforcement cross-section area. Column removal analyses revealed that the building was particularly sensitive to the loss of perimeter columns, whose failure could easily propagate to the central block, due to the lack of load redistribution capacity at the perimeter of the structure. In addition, the degradation analysis at the pool deck level showed that the initial failure of the pool deck slab could have caused relevant damage to the connection between the perimeter columns and the pool deck beams, leading to the failure of the perimeter columns. Indeed, the global static analysis revealed that the one-story structure of the pool area was subjected to higher deflections and stresses than those found in the main building. The side-by-side comparison between the simulated collapse through nonlinear dynamic analysis and the actual footage of the event showed a reasonable match.

Two variations to the original design were introduced aiming at avoiding the collapse propagation from the pool deck to the main structure and the collapse of the eastern wing of the building. It was found that the removal of the deep beams connected to the perimeter columns at the pool deck level was effective in preventing the initial failure of the pool deck slab from propagating to the rest of the building. In addition, increasing the torsional capacity of the eastern wing by introducing two shear walls oriented in the East-West direction, was proven to be effective in preventing its collapse. The overall separation between structures of different natures, such as one-story basements and twelve-story buildings, and RC cores consisting of shear walls oriented along the two principal directions, are the main lessons learned from the progressive collapse analysis of the tragic Champlain Towers South collapse.

DATA AVAILABILITY STATEMENT

Some or all data, models, or codes that support the findings of this study are available from the corresponding author upon reasonable request.

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NOTATIONS

The following symbols are used in this paper:

E = elastic modulus (GPa);

fc = compressive strength of concrete (MPa);

ft = tensile strength of concrete (MPa);

G = shear modulus (GPa);

G_i = element's centroid;

$k_{c,n}$ = normal stiffness of concrete springs

$k_{c,s}$ = shear stiffness of concrete springs

$k_{r,n}$ = normal stiffness of reinforcement springs

$k_{r,s}$ = shear stiffness of reinforcement springs

RTF = reinforcing bars;

ui = element's degree of freedom;

εu = ultimate strain of steel;

μ = friction coefficient;

σ_y = yield stress of steel (MPa);

σ_u = ultimate stress of steel (MPa);

τs = shear strength of concrete (MPa);

SUPPLEMENTAL DATA STATEMENT

Figs. S1–S14 are available online in the ASCE Library (ascelibrary.org).

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List of Tables

Table 1. Concrete material properties introduced in the AEM numerical model [Stresses in MPa (*ksi*), Elastic Modulus in GPa (*Mpsi*)]

Concrete	f_c	$f_t^{(a)}$	$\tau_s^{(b)}$	μ [-]	$E^{(c)}$	$G^{(d)}$
6000psi	41 (*6*)	4 (*0.6*)	13 (*1.9*)	0.8	32 (*4.7*)	13 (*1.9*)
5000psi	34 (*5*)	3 (*0.5*)	12 (*1.8*)	0.8	29 (*4.3*)	12 (*1.8*)
4000psi	28 (*4*)	3 (*0.4*)	11 (*1.6*)	0.8	26 (*3.8*)	11 (*1.6*)
3000psi	21 (*3*)	2 (*0.3*)	10 (*1.5*)	0.8	23 (*3.3*)	9 (*1.4*)

Note: [a] $f_t = f_c/10$; [b] $\tau_s = 3.8 f_c 0.33$; [c] E_c = 5000*(fck^(1/2)), [Mpa]; [d] G= E/(2(1+v)), v=0.2

Table 2. Steel material properties introduced in the AEM numerical model [Stresses in MPa (*ksi*), Elastic Modulus in GPa (*Mpsi*)]

Steel	σ_y	σ_u	ε_u	E	G
Grade60	414 (*60*)	579 (*84*)	1	200 (*29*)	80 (*12*)

Table 3. Loads [kN/m^2 (*lb/ft^2*)]

Floor	Dead load in addition to slab self-weight	Dead Load of walls & partitions	Live Load	Total per floor
Typical	≈1.0 (*20*)	≈0.5 (*10*)	≈0.5 (*10*) [a]	≈2.0 (*40*)
Lobby	≈2.0 (*40*)	≈0.5 (*10*)	≈1.0 (*20*)	≈3.0 (*60*)
Basement	-	-	≈1.0 (*20*) [b]	≈1.0 (*20*)

Note: [a] 1/4 of 2.0kN/m^2 (*≈40lb/ft^2*) design load for residential buildings; [b] 1/2.5 of 2.5kN/m^2 (*≈50lb/ft^2*) design load for garages.

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