

Progressive Collapse Analysis of the Champlain Towers South in Surfside, Florida

*Original*

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24 in Surfside, Florida, in 2021, one of the most catastrophic progressive collapse events ever occurred.  
25 The collapse analysis was performed using the latest developments in the Applied Element Method.  
26 A high-fidelity numerical model of the building was developed according to the actual structural  
27 drawings. Several different collapse hypotheses were examined, considering both column failures  
28 and degradation scenarios. The analyses showed that the failure of deep beams at the pool deck level,  
29 directly connected to the perimeter columns of the building, could have led to the columns' failure  
30 and subsequent collapse of the eastern wing of the building. The simulated scenario highlights the  
31 different stages of the collapse sequence and appears to be consistent with what can be observed in  
32 the footage of the actual collapse. To improve the performance of the structure against progressive  
33 collapse, two modifications to the original design of the building were introduced. From the analyses,  
34 it was found that disconnecting the pool deck beam from the perimeter columns could have been  
35 effective in preventing the local collapse of the pool deck slab from propagating to the rest of the  
36 building. Moreover, these analyses indicate that enhancing the torsional strength and stiffness of the  
37 core could have prevented the collapse of the eastern part of the building, given the assumptions and  
38 initiation scenarios considered.

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

## 41 **PRATICAL APPLICATIONS**

42 Building catastrophic collapses can cause significant lives and economic losses. Poor design and  
43 maintenance, in combination with aging, will more likely increase, in the next years, the number of  
44 buildings potentially vulnerable to the risk of collapse, due to either seismic, accidental, or  
45 degradation actions. This research focuses on the analysis of the Champlain Tower South condo  
46 collapse, which occurred in the City of Surfside in 2021. Different hypothetical collapse scenarios  
47 were simulated, comparing the analysis results with the actual evidence of the collapse. The analyses

48 have shown that the degradation of the pool deck slab, due to corrosion, may have contributed to the  
49 collapse of the building. Finally, two different minor revisions of the original design of the building  
50 were analyzed to reduce the risk of failure and understand how the collapse of similar residential  
51 buildings could be prevented.

## 52 **INTRODUCTION**

53 Over the last decades, the number of publications on themes related to the progressive collapse  
54 of buildings has exponentially increased (Gerasimidis and Ellingwood 2023). The attention to the  
55 disproportionate effect of a local failure dates back to 1970, when the first regulation related to  
56 accidental load was introduced in the UK code, as a consequence of the partial collapse of the Ronan  
57 Point building in London (Vrouwenvelder 2021). However, it was after the tragic terroristic attack  
58 on the World Trade Center in 2001 that the progressive collapse of structures captured the interest of  
59 the academic community (Lalkovski and Starossek 2022). Several definitions of Progressive Collapse  
60 were proposed by different authors over the last decades; ASCE 7-05 (ASCE 2005) defines  
61 progressive collapse as “*the spread of an initial local failure from element to element resulting,*  
62 *eventually, in the collapse of an entire structure or a disproportionately large part of it*”. Three  
63 common points can be identified among the different proposed definitions: the initial failure is local,  
64 the failure spreads to other structural members, and the final collapse is disproportionate to the initial  
65 failure (Kiakojoury et al. 2021). *In the last decades, several countries introduced specific regulations*  
66 *to address the risk of progressive collapse.* In the US, the General Service Administration (GSA)  
67 code (GSA 2013) was developed for government buildings, while the UFC 4-023-03 code (DoD  
68 2016) was introduced for military buildings. In Europe, Annex A in Eurocode 1 was introduced  
69 accounting for the first time for Accidental Actions (CEN 2006).

70 In contrast with the seismic design of structures, which is largely addressed in worldwide  
71 regulations through a more prescriptive code compliance approach, progressive collapse design often

72 requires a performance-based approach, by considering a series of “what if” scenarios (Fiorillo and  
73 Ghosn 2022).

74         Given that the objective of progressive collapse design is to ensure that a structure can  
75 withstand a certain level of local damage and avoid collapse propagation, it is understandable how  
76 the prediction of the initial damage effects and its possible propagation can be crucial as well as  
77 numerically challenging. Over the past decade, several researchers working on progressive collapse  
78 design suggested the introduction of robustness indexes. They can either be based on analytical or  
79 simplified numerical approaches, such as alternate load-path methods and push-down analyses  
80 (Praxedes and Yuan 2021). While both these approaches can be effective in assessing the risk of  
81 progressive collapse of relatively symmetric and homogenous structural systems, the progressive  
82 design of complex structural systems may require a more advanced methodology, such as the creation  
83 of high-fidelity numerical models (Sadek et al. 2022). While this approach was considered prohibitive  
84 in the past, because of the required computational effort, non-linear dynamic analyses of high-fidelity  
85 numerical models are now feasible thanks to the latest advancements in hardware computational  
86 capabilities and numerical methodologies (Stylianidis and Nethercot 2021) (Le and Bazant 2022).  
87 Among the numerical approaches to progressive collapse analysis, Finite Element Method, FEM, is  
88 widely adopted in several published studies. The FEM method can be efficiently used in progressive  
89 collapse analysis of frame structures, especially in code-based procedures (Kiakojoury et al. 2020).  
90 However, because the FEM solver is based on equilibrium equations, the solution cannot  
91 automatically implement element separations. Because of that, the capability to simulate the entire  
92 collapse of the structure is limited. Nevertheless, several strategies were developed in recent years to  
93 overcome FEM limitations in the analysis of large displacement problems. For example, the smeared  
94 crack technique was developed to allow for crack propagation in FEM analyses (Petrangeli and  
95 Ožbolt 1996). FEM application to progressive collapse analysis of entire structures often considers  
96 bi-dimensional frame elements to reduce the computational burden (Alashker et al. 2011). However,

97 researchers also developed a component-level, and multi-scale models approach assuming the refined  
98 3D modeling of only a portion of the structure (Li and Hao 2013), (Mpidi Bitu et al. 2022). Lastly,  
99 the recent development of FEM coupling methodology (Lu et al. 2009), and refined numerical  
100 procedure for element removal, such as the degree-of-freedom (DOF) release (Xu et al. 2018),  
101 overcome the FEM limitations to progressive collapse simulation.

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

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

113 This work focuses on the progressive collapse analysis of the Champlain Towers South  
114 condominium using the AEM method. The 2021 collapse of the Champlain Towers (Surfside,  
115 Florida) was one of the most catastrophic collapses that ever occurred to reinforced concrete (RC)  
116 residential buildings. Built-in 1982 as a part of a three-building complex, namely the Champlain  
117 Towers North, South, and East, the Champlain Towers South consisted of an L-shaped, twelve-story  
118 RC structure with flat slabs and a basement floor covering the entire footprint of the building area.  
119 What makes this event particularly interesting from the point of view of progressive collapse analysis,  
120 is that the evidence infers that the collapse was caused by a localized failure of a singular structural

121 element. Specifically, the failure of a slab due to punching shear would spread to the center of the  
122 building first, and then to the eastern wing a few seconds later (Lu et al. 2021).

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

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

## 135 **THE APPLIED ELEMENT METHOD**

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

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

144

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

146 In the AEM approach to the analysis of RC structures, the mechanical behavior of the concrete  
147 material is represented by a series of springs distributed along the interface between the two elements  
148 (Fig. 1, c). The contribution of steel rebars embedded in the material can be explicitly accounted for  
149 by coupling the mechanical contribution of additional springs representing the steel reinforcement.  
150 The steel springs are placed in their actual position in the cross-section of the considered structural  
151 element (Fig. 1, d). As the springs consider the axial stiffness  $k_n$ , and the shear stiffnesses  $k_{s,1}$  and  $k_{s,2}$ ,  
152 the contribution of both longitudinal and transversal reinforcing bars, for the given constitutive laws,  
153 is automatically accounted for in the numerical analysis. In this study, the Maekawa and Okamura  
154 (1985) model is considered for representing the axial behavior of concrete (Fig. 1, e), while a linear  
155 relationship up to failure is assumed for the behavior of concrete subject to combined shear and  
156 compressive loads (Fig. 1, f). Finally, the Menegotto and Pinto (1973) model is adopted for  
157 representing the nonlinear behavior of steel reinforcement (Fig. 1, g).

158

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

161

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

## 164 **THE AEM NUMERICAL MODEL OF THE CHAMPLAIN TOWERS CONDO**

### 165 **Structure description and material properties**

166 The Champlain Towers South structure consists of RC flat slabs supported by RC columns.  
167 The thickness of the slab is 23 cm (9") on the basement floor, 24 cm (9 ½") at the Lobby level, and  
168 20cm (8") for typical floors. Different concrete compressive strengths were considered in the design



169 of the building: columns and shear walls were designed with strength varying from 41 MPa (6000  
170 psi) to 28 MPa (4000 psi), while the slabs were designed with compressive strength varying from 28  
171 MPa (4000 psi) to 21 MPa (3000 psi), (Fig. 2, a). The longitudinal reinforcement of columns is  
172 varying from size Ø36 mm (#11) at the lower floors to Ø25 mm (#8) at the upper floors. The  
173 reinforcement of the two shear walls includes two columns at each edge and a reinforcement Ø13  
174 mm (#4) mesh, spaced at 30 cm (12"). Ø13 mm (#4) stirrups were used for Ø36 mm (#11) longitudinal  
175 reinforcement while Ø10 mm (#3) stirrups were used for the rest of the bar sizes, (Fig. 2, b).

176 Fig. 2. Color map of concrete strength in columns, shear walls, and slabs (a) [MPa (*ksi*)],  
177 and diameter of reinforcement bars implemented in the numerical model (b) [mm]  
178

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

180 Table 1. Concrete material properties introduced in the AEM numerical model [Stresses in  
181 MPa (*ksi*), Elastic Modulus in GPa (*Mpsi*)]  
182

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

190 Fig. 3. AEM numerical model view of the punching shear reinforcement in the lobby slab,  
191 basement, 2nd floor, and typical floor.  
192

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

196 (36"x 42") transfer girders are supporting 30,5x61cm (12"x24") columns elevating from the 2<sup>nd</sup> floor  
197 to the roof.

198 As specified in the as-built drawings notes (William M. Friedman & Associates Architects  
199 1979), reinforcing bars meet ASTM A-615 Grade 60 criteria, with yield strength equal to 414 MPa  
200 (60 ksi), (Table 2).

201 Table 2. Steel material properties introduced in the AEM numerical model [Stresses in MPa  
202 (ksi), Elastic Modulus in GPa (Mpsi)]  
203

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

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

### 213 **Loads**

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

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

228 Table 3. Loads [ $\text{kN/m}^2$  ( $\text{lb/ft}^2$ )]  
229

230 For the Typical Floor, the following loads were assumed based on the typical weights of the  
231 building materials (Breyer et al. 2020): dead load in addition to slab self-weight,  $1\text{kN/m}^2$  ( $20\text{lb/ft}^2$ ),  
232 accounting for the floor finishes, ceilings, façade elements, windows, doors, railings, MEP systems,  
233 and any additional load not explicitly introduced in the model; dead load of walls & partitions,  $0.5$   
234  $\text{kN/m}^2$  ( $10\text{lb/ft}^2$ ); live load,  $0.5\text{kN/m}^2$  ( $10\text{lb/ft}^2$ ); the total considered distributed load results  $2\text{kN/m}^2$   
235 ( $40\text{lb/ft}^2$ ).

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

## 239 **NON-LINEAR DYNAMIC ANALYSES AND COLLAPSE SCENARIOS**

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

245           **Column removal scenarios**

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

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

263           Fig. 4. Inner (top) and perimeter (bottom) column removal scenario, Vertical deflection.  
264

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

267           **Localized degradation scenario**

268           Several media discussed evidence of extensive degradation of the pool deck slab in the  
269 immediate aftermath of the collapse. Indeed, the area was partially covered by ornamental plants,  
270 which, on top of the additional weight, might also have caused corrosion of the slab steel

271 reinforcement due to watering and lack of proper impermeabilization. Static analyses were performed  
272 considering only the vertical dead load, as per load assumptions, without accounting for any  
273 degradation. These analyses showed that the area of the pool deck was substantially weaker and  
274 subjected to higher deflections and stresses than the area within the twelve-story building footprint  
275 (Fig. 5, a). For example, the deflection in the pool deck area reaches 2cm, while it resulted in only  
276 fractions of a centimeter in the area pertaining to the twelve-story building itself. The larger spans,  
277 together with the limited reinforcement, led to stresses in the punching shear reinforcement  
278 substantially higher than in the rest of the structure. The mean value of normal stresses in the punching  
279 shear reinforcing bars on the ground floor of the twelve-story building was found to be below 100MPa  
280 ( $\approx 15$ ksi). In the pool deck area, the normal stresses in the punching shear reinforcing bars reached  
281 200MPa ( $\approx 30$ ksi), which corresponds to half of the yield stress of the steel, 414MPa (60ksi),  
282 according to the original design specification (William M. Friedman & Associates Architects 1979).  
283 In particular, the area where the initial collapse occurred, showed the highest stresses, specifically at  
284 the top side rebars of the pool deck slab (Fig. 5, b).

285 Fig. 5. Static analysis, Vertical displacement at basement level [cm (*in*)] (a), and normal  
286 stresses in top punching shear reinforcement [MPa (*Ksi*)] (b)  
287

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

290 Fig. 6. Building plan at basement level with the indication of the twelve-story building's  
291 footprint and the degraded area of the pool deck level  
292

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

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

300 Fig. 7. Lobby level, comparison between cracks distribution in case of foundation  
301 settlement scenario (left) and pool deck degradation scenario (right), Principal strains in Dir.1 [-]  
302

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

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

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

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

320 The three previously mentioned “slab drops” were originally designed to be 59cm (23”), and  
321 then reduced to 46cm (18”) in a second design revision (William M. Friedman & Associates  
322 Architects 1979). The degradation analysis of the pool deck slab shows how the depth of the three

323 girders, resulting from the slab drops, could have played a significant role in propagating the collapse  
324 of the slab to the rest of the building. Because the building was particularly sensitive to the loss of  
325 perimeter columns, when the slab and connecting beams fail, a concentrated bending moment is  
326 transferred to the three perimeter columns, leading to column overload and consequent collapse of  
327 the building (Fig. 8).

328 Fig. 8. Effect of deep beams in causing the instability of the perimeter columns at 2.0s,  
329 Principal strains in Dir.1, Scale color red equal to 0.1 Strain [-], and deformed shape scaled by a  
330 factor of 2 (left)  
331

### 332 **Arch and catenary actions: from the failure of the pool deck slab to the** 333 **disproportionate collapse of the building**

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

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

345 Fig. 9. Column 11.1-L, Compressive stresses in the concrete [MPa (*Ksi*)] (top), normal  
346 stresses in beams' and columns' reinforcement [MPa (*Ksi*)], normal forces [kN (*kips*)] in the  
347 column and bending moments in the beams [kN\*m (*kips\*inch*)] (bottom), at different stages of the  
348 collapse.  
349

350 When degradation is introduced, steel rebars start yielding. The slab starts deflecting  
351 downwards and the concrete in the perimeter columns reaches its maximum compressive strength at

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

364 Fig. 10. Distribution of Principal Strains [-] after column' failure (left), and punching shear  
365 failure at pool deck slab (right)  
366

### 367 **Side-by-side comparison of the collapse**

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

375 Fig. 11. Side-by-side comparison between the footage of the collapse and the numerical  
376 analysis in nine different timestamps; Image courtesy of Slater (2021).  
377



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

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

387 Fig. 12. Comparison between actual debris distribution (a) and analysis results (b); Image  
388 (a) reconstructed by the Authors based on available media pictures.  
389

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

## 395 **PROGRESSIVE COLLAPSE ANALYSIS UNDER ALTERNATIVE DESIGN SCENARIOS**

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

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

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

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

412 Fig. 13. Original design (a), and alternative scenario (b), assuming the absence of beams  
413 connecting perpendicularly the pool deck to the perimeter column of the structure, Analysis results,  
414 Vertical displacement (c) [cm (*in*)], and Principal strains at slab failure (d) [-]  
415

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

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

423 The reason why, after the collapse of the central portion of the building, the collapse  
424 propagated to the eastern wing only, is related to the different torsional stiffness resulting from the  
425 structural layout. While the West core of the building consisted of RC shear walls oriented both in  
426 the North-South and East-West directions, the East RC core of the structure was composed of only  
427 one shear wall oriented in the North-South direction. Therefore, the torsional capacity of the western  
428 core was significantly larger than the one of the eastern core. When the central block of the building  
429 starts failing, it was observed that slabs were detaching from the western and eastern wings at the

430 interface of the shear wall. Afterward, the eastern wing fails, due to the lack of torsional capacity. In  
431 light of that, two additional shear walls were introduced in the East-West direction (Fig. 14, c). Results  
432 show that the change in the design is effective in avoiding the collapse of the eastern wing (Fig. 14,  
433 d, f). The difference in terms of rotation observed both in the original model and in the new design  
434 configuration can be evaluated from, Fig. 14, e, f.

435 Fig. 14. Original design (left), and alternative scenario (right), assuming the presence of two  
436 additional shear walls  
437

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

## 441 **CONCLUSIONS**

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

454 the simulated collapse through nonlinear dynamic analysis and the actual footage of the event showed  
455 a reasonable match.

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

#### 466 **DATA AVAILABILITY STATEMENT**

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

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478 Champlain Tower South Condo (Slater 2021).

## 479 NOTATIONS

480 *The following symbols are used in this paper:*

481  $E$  = elastic modulus (GPa);

482  $f_c$  = compressive strength of concrete (MPa);

483  $f_t$  = tensile strength of concrete (MPa);

484  $G$  = shear modulus (GPa);

485  $G_i$  = element's centroid;

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

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

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

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

490 RTF = reinforcing bars;

491  $u_i$  = element's degree of freedom;

492  $\epsilon_u$  = ultimate strain of steel;

493  $\mu$  = friction coefficient;

494  $\sigma_y$  = yield stress of steel (MPa);

495  $\sigma_u$  = ultimate stress of steel (MPa);

496  $\tau_s$  = shear strength of concrete (MPa);

497

498 **SUPPLEMENTAL DATA STATEMENT**

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

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

Concrete	$f_c$	$f_t^{(a)}$	$\tau_s^{(b)}$	$\mu$ [-]	$E^{(c)}$	$G^{(d)}$
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: <sup>a</sup>  $f_t=f_c/10$ ; <sup>b</sup>  $\tau_s=3.8 f_c 0.33$ ; <sup>c</sup>  $E_c = 5000*(f_c k^{1/2})$ , [Mpa]; <sup>d</sup>  $G= E/(2(1+\nu))$ ,  $\nu=0.2$

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

Steel	$\sigma_y$	$\sigma_u$	$\epsilon_u$	E	G
Grade60	414 (60)	579 (84)	1	200 (29)	80 (12)

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Table 3. Loads [kN/m<sup>2</sup> (*lb/ft<sup>2</sup>*)]

Floor	Dead load in addition to slab self-weight	Dead Load of walls & partitions	Live Load	Total per floor
Typical	$\approx 1.0$ (20)	$\approx 0.5$ (10)	$\approx 0.5$ (10) <sup>(a)</sup>	$\approx 2.0$ (40)
Lobby	$\approx 2.0$ (40)	$\approx 0.5$ (10)	$\approx 1.0$ (20)	$\approx 3.0$ (60)
Basement	-	-	$\approx 1.0$ (20) <sup>(b)</sup>	$\approx 1.0$ (20)

Note: <sup>a</sup> 1/4 of 2.0kN/m<sup>2</sup> ( $\approx 40lb/ft^2$ ) design load for residential buildings; <sup>b</sup> 1/2.5 of 2.5kN/m<sup>2</sup> ( $\approx 50lb/ft^2$ ) design load for garages.

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