An Investigation and Analysis of the 2021 Surfside Condo Collapse

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An Investigation and Analysis of the 2021 Surfside Condo Collapse

A Thesis

Submitted to the Graduate Faculty of the
University of New Orleans
in partial fulfillment of the
requirements for the degree of

Master of Science
in
Engineering
Civil and Environmental Engineering

by

Ryan Blanchard

B.S. Loyola University New Orleans, 2016

May, 2022
DEDICATION

Dedicated to those who lost their lives.

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Tzvi Ainsworth
Michael Altman
Richard Augustine
Luis Fernando Barth
Valeria Barth
Deborah Berezdivin
Luis Bermúdez
Elena Blaser
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Julio Velasquez
Theresa Velasquez
Leidy Vanessa Luna
Villalba
Benny Weisz
Lisa “Malky” Weisz
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Thank you for your guidance through this process.
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Thank you for your fountain of encouragement and wisdom.

My love,

You are my foundation.
You keep me grounded and help me reach heights I still struggle to comprehend.

Mom and Steve,

Thank you for all of your sacrifices to get me to this moment.
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A central mission of civil engineering is to create safe structures to prevent the loss of life and property. The most effective tool in this effort is the development and standardization of codes to govern best practices, material specifications, and safe minimums and assumptions. While much of this is done through thorough testing and research, many lessons are learned the hard way. Historically, major disasters and failures have brought about new standards and changes to ensure that history is not repeated. On June 24th, 2021, 98 people lost their lives when Champlain Towers South, a 12-story condominium in Surfside, Florida, collapsed suddenly in the night. There are lessons to be learned in this tragedy. This investigation and analysis seeks to find them.
CHAPTER 1 INTRODUCTION

1.1 Introduction

On June 24th, 2021, Champlain Towers South (CTS) suffered a partial collapse at approximately 1:22 a.m., killing 98 people. Completed in 1981, the 12-story condominium stood on Surfside Beach for nearly 40 years. The building had a history of maintenance issues mostly stemming from its proximity to the Atlantic Ocean.

On June 25th, NIST announced that it was launching a full technical investigation of the collapse under the authority of the National Construction Safety Team (NCST) Act. NIST’s role under the NCST Act is not to determine whether there was a criminal act or a violation, nor to determine any associated culpability. Instead, the ultimate goal of the NIST investigation is to determine the technical cause of the collapse and, if indicated, to recommend changes to building codes, standards and practices, or other appropriate actions to improve the structural safety of buildings. There are millions of high-rise condominium units in Florida alone, many of them aging structures near the ocean. While a NIST investigation is intended to identify the cause of the Champlain Towers South collapse, it could also uncover potential issues in other similar buildings nearby and throughout the nation.

This is the fifth investigation that NIST has conducted under the NCST Act. Previous NCST reports include the World Trade Center final reports published in 2005 (WTC 1 and 2) and 2008 (WTC 7); the 2003 Station nightclub fire final report published in 2005; and the 2011 Joplin tornado investigation report published in 2014. The Hurricane Maria investigation is ongoing (with an interim report recently published).

1.2 Limitations

The major limitation of this investigation was access to information and evidence. All information that was worked with is that which is publicly available. I have no access to materials recovered from the site, nor documents that are private, nor interview details from eyewitnesses.

For instances, in the bidding process for the work necessary for the 40-year certification, potential contractors would have walked through the site taking their own pictures and videos documenting the condition of the building to form their bid estimates. Access to that information would no doubt be invaluable in determining the state of the building immediately before collapse and provide a large sample of evidence from which to draw evidence and conclusions. Additionally, eyewitness testimony was only from those who gave interviews to the press, only covered the broad sequence of events, and confirmed information that was already established.
Another hindrance was limited access to modeling software. With the resources available, I was only able to model single members at a time and my calculations mostly had to be simple enough to perform by hand. This limited both the number and complexity of analysis.

Finally, time was the ultimate limiting factor. Because of the time limitations of this investigation, evidence had to be prioritized. Evidence that was more obvious in importance was examined thoroughly while evidence that didn’t have obvious importance received a cursory examination or none at all. Because of time constraints, this investigation also suffers from tunnel vision and only one hypothesis was analyzed. This investigation neither explores alternative hypotheses nor seeks to disprove this central hypothesis.
2.1 Current NIST Investigation

On June 25th, NIST announced they would be launching an investigation into the cause of the collapse. “This is an unspeakable tragedy, and like all NIST investigations, we will conduct a fact-finding study to prevent tragedies like this in the future,” said James Olthoff, who is currently performing the duties of the undersecretary of commerce for standards and technology and NIST director. “We intend to undertake a thorough technical investigation into what caused the collapse, to ultimately make recommendations that would make our buildings safer and keep something like this from happening again. This effort will take time, but we will work on this as long as necessary.”

There are millions of high-rise condominiums in Florida alone, many of them near the ocean and/or aging. The findings of this investigation could uncover potential issues for other similar buildings nearby and throughout the nation.

Leading up to this decision, a preliminary NIST team spent several days in Surfside, Florida, to determine if the event met the criteria for a full investigation under the NCST Act. The team determined it was a

“major building failure at significantly less than its design basis, during construction, or while in active use… a fact-finding investigation of the building performance and emergency response and evacuation procedures will likely result in significant and new knowledge or building code revision recommendations needed to reduce or mitigate public risk and economic losses from future building failures.”

The Champlain Towers South investigation is the fifth investigation NIST has conducted using authorities granted by the 2002 National Construction Safety Team (NCST) Act. The act gives NIST and its teams primary authority to investigate the site of a building disaster, access key pieces of evidence; and collect and preserve evidence from the site of a failure or disaster. It also calls for NIST to issue reports and make recommendations to improve building codes and standards.

The team is led by Judith Mitrani-Reiser, associate chief of the Materials and Structural Systems Division in NIST’s Engineering Laboratory. In that role, Mitrani-Reiser leads the development and coordination of statutory processes for making buildings safer. She manages and provides oversight on building failure investigations and coordinates work with other federal agencies to reduce losses in the United States from disasters and failures of our built environment.

Glenn Bell, co-director of the safety organization Collaborative Reporting for Safer Structures and co-founder of the American Society of Civil Engineers Technical Council on Forensic Engineering, serves as associate lead. Bell has more than 45 years of experience
evaluating existing structures, investigating structural failures and taking the lessons learned from them to ensure those failures are not repeated.
“This team has an incredible amount of experience in forensic engineering, having studied many building failures,” said Mitran-Reiser. “We are going into this with an open mind and will examine all hypotheses that might explain what caused this collapse. Having a team with experience across a variety of disciplines, including structural and geotechnical engineering, materials, evidence collection, modeling and more, will ensure a thorough investigation.”

The goals of the NIST investigation are:

1. Establish the likely technical cause or causes of the building failure.
2. Recommend, as necessary, specific improvements to building standards, codes and practices.
3. Recommend any research and other appropriate actions needed to improve the structural safety of buildings.

The members of the investigation are split into the following groups:

- Building and Code History: Jim Harris and Jonathan Weigand
- Evidence Preservation: David Goodwin and Christopher Segura
- Materials Science: Ken Hover and Scott Jones
- Geotechnical Engineering: Youssef Hashash and Sissy Nikolaou
- Structural Engineering: Jack Moehle and Fahim Sadek
- Social Science: Emel Ganapati

2.2 Relevant differences between ACI 318-77 (‘77) and ACI 318-14 (‘14)

In general, ‘77 is organized mainly by type of loading conditions (i.e., Axial and Flexure) with separate sections for the design of more complex members (i.e., 2-way slabs). This leads to flipping back and forth between sections often to find applicable codes for the specific loading situation of each member. Additionally, ‘77 often uses inconsistent language in and between sections. For instance, for flexural design it defines the minimum reinforcement ratio (rho min) while for shear design it defines the minimum area of steel (Av min).

‘14 does much better on both issues. ‘14 is organized by mainly member type with separate sections for more general requirements. This leads to a much clearer reading experience with most of the information needed for a member in one location. Additionally, ‘14 uses very consistent language with it often represented in multiple forms (i.e required area of steel and minimum reinforcement ratio).

In general, ‘14 gives more detailed equations that offer accuracy across a variety of materials that could be used in the design of reinforced concrete. Often, ‘14 gives a more elaborate governing equation for a particular situation alongside a simpler minimum requirement. These
minimum requirements often line up closely with the equations given in ‘77. However, some equations in ‘77 overestimate values significantly compared to corresponding equations in ‘14, sometimes by as much as 2 or 3 times.

Additionally, ‘14 goes through both the determination of forces on a member and the design of said member to support those forces in a clear, thorough, and easy to follow manner. That cannot be said of ‘77.

2.2.1 Loads

‘77 offers far fewer factored loading combinations than ‘14. Additionally, ‘77 does not elaborate on what falls under what type of loading. In general, ‘77 uses larger, more conservative factors for calculating loading combinations. However, ‘14 offers more options for load combinations that focus on particular types of loading that are of most concern.

\[
U = 1.4D + 1.7L \quad \text{(9-1 ACI 318-77 p.29)}
\]
\[
U = 1.2D + 1.6L + .5(Lr \text{ or } S \text{ or } R) \quad \text{(5.1.3b ACI-14 p.58)}
\]

2.2.2 Beams

The governing equations for minimum flexural reinforcement in beams for both codes are as follows:

\[
Rho = \frac{200}{fy} \quad \text{(10-3 ACI 318-77 p.34)}
\]
\[
\text{As min} = \text{lesser of: } 3*\sqrt{f'c}/fy * \text{bw*d}
\]
\[
\text{And } \frac{200}{fy} * \text{bw*d} \quad \text{(ACI 318-14 p.137)}
\]

‘14 provides the exact same equation (in a different form) as ‘77 along with a more versatile equation that considers the wide variability of reinforced concrete. This is typical of the differences between the two.

For shear strength on flexural members, ‘77 well overestimates ‘14.

\[
Vc = 2*\sqrt{f'c}*\text{bw*d} \quad \text{(11-3 ACI 318-77 p.41)}
\]
\[
\text{Av min/s} = .75*\sqrt{f'c} \text{bw/fy} \quad \text{(Table 9.6.3.3 ACI 318-14 p.139)}
\]

Again, these are the same equations in different forms except for the factor in front. ‘77 overestimates ‘14 by 2.67 in this case. ‘14 also provides details of how shear reinforcement should be varied over the span of a beam while ‘77 does not.
2.2.3 Columns

The details for column design vary significantly between the two. Here the difference in organization is very apparent with ‘14 providing a separate chapter on column design which goes in depth about design considerations and governing equations specifically on the interaction of axial and moment forces. ‘77 simply mentions that moment forces should be considered in the design of axial members and offers an equation to increase the design moment based on the properties of the member. Furthermore, ‘14 offers and explains column interaction diagrams along with equations for generating the various points used to generate said diagram. The commentary for ’77 does show a simplified diagram but does not give any equations of points on it for someone to generate said diagram.

2.2.4 2-way Slabs

This section was the most similar between the two codes. They offered the same general information as well as the same detailed methods of design and analysis. ‘14 tended to offer the same information in a clearer way with simple charts versus the complicated equations offered in ‘77. The biggest differences are, ‘14 does not offer bent reinforcement in 2-way slabs, nor does ‘14 prescribe continuous or overlapped bottom reinforcement. ‘14 also, puts hard caps on many minimums and maximums. For example, maximum spacing of reinforcement:

\[ s = 2h \]  \hspace{1cm} (13.4.2 ACI 318-77)

\[ s = \text{lesser of: } 2h \text{ or } 18" \]  \hspace{1cm} (8.7.2.2 ACI 318-14 p.106)

2.2.5 Temperature Steel

Here both versions have similar requirements with ‘14 requiring .002 times the gross area of concrete for steel with a yield strength of 60000 psi or less, while ‘77 requires .002 times the gross area of concrete for steel of 40000 or 50000 psi and .0018 times the gross area of concrete for steel of 60000 psi.
CHAPTER 3 GOALS AND OBJECTIVES

The main objective of this investigation is to understand the underlying mechanisms of the CTS collapse.

This objective was met by accomplishing the following specific goals:

1) Decipher the available information and pinpoint critical pieces of evidence.
2) Evaluate the strength and demands of key members and their influence on the whole structure.
CHAPTER 4 FACTUAL INFORMATION

4.1 Condo Collapse

4.1.1 History and Location

CTS was designed by William M. Friedman and Sergio Breiterman who were known for cutting corners. Both later had their licenses suspended for ‘gross incompetence’ unrelated to CTS.

CTS was the first project of a group of Canadian businessmen with the vision of bringing luxury high-rise developments to the relatively untapped beachfront town of Surfside. The development was plagued with issues. All within less than a year of construction, the first general contractor resigned, the second was fired, a building moratorium delayed the breaking of ground until the developers paid the city $200,000 to update the town's inadequate sewer, and a mid-project cease-and-desist when developers submitted a last-minute addition of a penthouse.

Completed in 1981, the 12-story condominium stood on Surfside Beach for nearly 40 years. The building had a history of maintenance issues exacerbated by its proximity to the Atlantic Ocean.

4.1.2 Timeline of Events

1981: CTS construction completed
2016: Neighboring 87 Terrace construction completed
2018 October: engineering report warns of major structural damage and exponential expansion of deterioration
2020 October: remedial work approved by condo association
June 24, 2021 ~1:15 a.m.: Tourists nearby hear what sounds like a car accident. Find rubble in the garage. Several residents hear loud construction noises.
June 24~1:22 a.m.: The central portion of the building collapses cascading to the rest of the building. Search and rescue efforts begin before dawn. The only four survivors are found.
June 25: state of emergency declared in Florida.
July 1: NIST announces beginning of full investigation.
July 4: the standing portion of the building is demolished.
July 7: operations shift from search and rescue to recovery.
July 9: the last survivor, Binx the cat is rescued.
July 26: the final missing person is identified, bringing the death toll to 98.

4.2 Design Drawings

The original design drawings make up a 336-page pdf accessed from the Surfside public records page. Within this document, there are at least three versions of the design plans of CTS: the 1979 plans, the 1979 penthouse addition plans, and the 1980 plans with revisions. These versions are more or less identical with only minor changes (and a penthouse) between them. There are possibly 4-5 versions, but these are the only labeled ones. It was extremely difficult to even
identify which papers belonged to which version as they were out of order and almost all were unlabeled, undated, and/or damaged. Most of the pages are of little note for this analysis as they detail things like the water and sewage system, electrical systems, and layouts of individual condo units.

![Figure 4.2.1: Example of drawing of unknown date and version (Friedman p. 1)](image)

4.2.1 First Floor Framing Plan

First is the ‘79 and ‘80 versions of the first-floor framing plan. In the ‘79 version, there is an elevation change that acts as a supporting beam along the K and 12.1 column lines. This beam is not present in the ‘80 version. As will be shown later, this can provide a clear indication of which version of the plans were used in construction. The area near this elevation change has several heavy planters directly on the slab along these lines in both versions of the plans (Friedman p. 40), as well as seen in photos of the pool deck area (“Champlain Towers South.” Miami Condos). Several columns supporting the pool deck are turned 90 degrees between the two versions; this change is not marked as a revision on the ‘80 plans. Another area of interest on the first-floor framing plan is the area between column lines K and M, and column line 10 (supporting the building proper), and 11 (supporting the pool deck area.) This area features several elevation-change beams and transfer beams with planters above. This area is of note because it is close to the location of the rubble seen on video taken a few minutes before the building collapse. (NewsNation). Lastly, on the first-floor framing plan are the reinforcement details for the beam type A, elevation changes, and within the slab. These are used in the analysis to calculate the strength of select members.
Figure 4.2.2: 1979 first floor framing plan (Friedman p. 162)

Figure 4.2.3: 1980 revisions first floor framing plan (Friedman p. 31)
Figure 4.2.4: Reinforcement details for common beam types (Friedman p. 162)

Figure 4.2.5: Close up of planter area (Friedman p. 162)
Figure 4.2.6: 1979 first floor framing plan with elevation change along column line K present (Friedman p. 162)

Figure 4.2.7: 1980 first floor framing plan with elevation along column line K missing (Friedman p. 31)
4.2.2 Column Schedule

Next in the plans is the column schedule (Friedman p. 35) which mentions the reinforcement details of the columns, with column types A, C, and N relevant to this analysis. The type A columns were 24 inches square and supported the part of the building that withstood the collapse. The type C columns were 16 inches square and supported the perimeter of the portion of the building that collapsed. The type N columns were 8 inches by 16 inches and supported the pool deck.

![Column Schedule](image)

Figure 4.2.8: Column Schedule (Friedman p. 35)

4.2.3 Garage Floor Plan

Further into the document, the garage level floor plan (Friedman p. 59) shows the layout and numbering of parking spaces in the garage and will be helpful when reviewing videos to align what is seen with the design documents.
4.2.4 Flat Plate Sections

The flat plate sections (Friedman p. 38), shows the arrangement of reinforcement in the slabs which is useful for calculating their strength. This section of the document shows the reinforcement at slab openings used to resist punching shear failure.

4.2.5 Eastern Elevation

Next, the eastern elevation (Friedman p. 46), shows the height of the southern privacy wall to be 4 feet and eastern privacy wall to be 3 feet, which will be used in later calculations. Additionally, it gives the elevation of each floor, useful for the column analysis.

Figure 4.2.9: Reinforcement details for column types A, C, and N (Friedman p. 35)
Figure 4.2.10: Pile cap plan showing the location of each column type. Type A is marked in blue; Type C is marked in red; Type N is marked in Yellow (Friedman p. 159).

4.2.6 Site Plan

The last page of note is the site plan (Friedman p. 96) which gives the arrangement of planters and landscaping. This page clearly calls for the deck and parking areas to be sloped.
Figure 4.2.11: Garage level floor plan (Friedman p. 59)

Figure 4.2.12: Flat plate sections (Friedman p. 38)
Figure 4.2.13: East Elevation (Friedman p. 46)

Figure 4.2.14: Site Plan (Friedman p. 96)
4.3 2018 Field Survey Report

In 2018, the condo association for CTS contracted Morabito Consultants to perform an inspection in anticipation of the 40-year recertification process. In the report from this inspection, there is documentation of widespread water damage to the building. While most of this is cosmetic, the last three pages note significant structural damage.

4.3.1 Waterproofing

The report covers the waterproofing on the pool deck, drive area, and planters (Moribito, 2018, p. 7). Specifically, it says the waterproofing is beyond its useful life and “Failure to replace the waterproofing in the near future will cause the extent of the concrete deterioration to expand exponentially.” Additionally, it cites that these areas are not sloped, causing water to sit on the waterproofing layer until it evaporates.

4.3.2 Cracking and Spalling

The last two pages note significant spalling and cracking on the garage level (Moribito, 2018, pp.8-9). The report also notes that many of the cracks had been repaired before, with telltale signs of poor craftsmanship and new cracks spreading from the repaired spots.

Figure 4.3.1: Typical cracking and spalling (Moribito, 2018, pp.8-9)
4.4 2020 Condo Association Meeting Minutes

In 2020, with the recertification process beginning, the CTS condo association once again contracted with Moribito to inspect the integrity of the structure and to begin the bidding process for the work that needed to be done. The findings of this inspection are detailed in the minutes from the October 2020 condo association meeting.

4.4.1 Exploratory Demolition

The testing included some exploratory demolition, performed by a contractor only referred to as CPR, to gauge the condition of the structural slab. Five 3 ft. by 3 ft. cutouts and six cores were taken at various locations which are detailed on a site map (minutes p. 43). Test probe ‘A’ and core ‘A’ were performed near the planter area along column line 11. Additionally, note 8 of the investigation summary states, “The contractor shall restore all demolished areas back to their original conditions.” This includes the waterproofing, paver system, stamped concrete, and planters. This page is dated July 13th, 2020.

Figure 4.4.1: Examples of Cores and Test Probes taken (minutes pp. 38-39)
Figure 4.4.2: (a) (Left) Locations of samples taken, (b) (right) close up of test probe A location, (c) (below) Note 8 specifying repair procedures (minutes p. 43)

8. MORABITO CONSULTANTS SHALL BE NOTIFIED TO INSPECT AND DOCUMENT ALL TEST PITS UPON CONTRACTORS COMPLETED DEMOLITION. ONCE THE INSPECTION AND DOCUMENTATION IS COMPLETED, THE CONTRACTOR SHALL RESTORE ALL DEMOLISHED AREAS BACK TO THEIR ORIGINAL CONDITIONS. REPLACE ANY DAMAGED WATERPROOFING PER WATERPROOFING MANUFACTURERS REQUIREMENTS, RE-INSTALL THE PAVER SYSTEM AT THE POOL DECK, REPLACE/REPAIR THE STAMPED CONCRETE AT THE PARKING AREA, AND RESTORE THE PLANTER TO THE ORIGINAL CONDITION.

Figure 4.5.1: image of pool deck facing East with extra planters in the background
4.5 Pre-Collapse Images

4.5.1 Miami Condo Investments Photos

Several photos of CTS were retrieved from a listing on Miami Condo Investments, a realty agency based in Miami. These photos generally show the exterior and public areas of CTS. Three photos of the pool deck area, retrieved from a condo investment website, were of importance to this analysis. The first picture shows the pool deck facing east towards the beach. Here there are several square planters visible that are not noted in any of the design drawings (“Champlain Towers South.” Miami Condos). The second picture shows the entire building facing north-west (“Champlain Towers South.” Miami Condos), where the planters along column line K are shown clearly as well as the extra planters noted earlier.

The last picture shows the pool deck looking down from the roof again facing east towards the beach (“Champlain Towers South.” Miami Condos). A hole is seen near the large planters between column lines K and M. Based on the inspection survey in the condo association meeting minutes (minutes p. 43), this approximately lines up with the location of test probe ‘A’. Using the test probe hole as a reference we find that the shadow of the southern wall leans approximately 3 ft. north, and the shadow on the eastern wall leans approximately 1.5 feet to the west. Using this information in conjunction with the longitude and latitude of CTS (25.8728 -80.1212), this picture was taken in mid-October sometime between 10:30 a.m. and 11:00 a.m. If this picture was taken on or near July 13th, the shadow from the southern wall would be less than six inches long. This hole straight down to the structural slab was open to the elements for at least three months allowing water to bypass all waterproofing protections during this period.

Figure 4.5.2: (a) (above) The pool deck from above showing the extra planters and a cutout. (b) (left) A close up of the cutout.
4.5.2 July 17th, 2020 Garage Walkthrough

On July 17th, 2020, a prospective condo owner went to CTS to see a unit for sale and filmed their visit. During this visit they walked through the parking garage to see where their parking spot would be. This video gives significant insight into the state of CTS immediately before the collapse as we get a fairly comprehensive view of the parking garage and the damage within and can place the damage based on the markings seen.

At 0:24, there are extensive stalactites from calcium carbonate leaching hanging from the ceiling and many of these are rust colored. These are signs of severe water intrusion from above. Not only are minerals being leached from the concrete, but the rebar within the slab is also being deteriorated. The stalactites are centered around a previous repair, showing its ineffectiveness.

At 0:51, in the center of the frame is column I12.1, behind and to the left is column K11.1. We can confirm this as these two columns are not in line which matches the garage level floor plan (Friedman p. 59). From this vantage point, if it were present, the one-foot elevation change should be evident, meaning construction followed the 1980 revisions and not earlier plans.

At 1:02, the central drive is directly in front of the camera. This is between column lines L and M. On the ceiling of the garage the elevation-change near the planters next to the building edge is visible. There is a stark contrast between what is underneath the pool deck and what is underneath the building. Under the pool deck, there is bubbled and peeling paint, and the beam type A in frame is chalky white with almost no paint left, clear signs of water intrusion. The area under the building is pristine, painted concrete.
Figure 4.5.4: screenshot from 0:24 of garage walkthrough showing rust and stalactites near a previously repaired crack. (Terenzi, F.)

Figure 4.5.5: screenshot from 0:55 of the garage walkthrough showing the beam type A extending left from column K11.1 and no beam extending right (Terenzi, F.)
Figure 4.5.6: screenshot from 1:02 of garage walkthrough video showing contrast between what is under the pool deck and building (Terenzi, F.)

Figure 4.5.7: Google Earth image from 12/2004 showing palm trees in the extra planters.
4.5.3 Google Earth Historical Maps

Reviewing Google Earth maps, large palm trees can be seen in these planters. Palm trees generally weigh about 150 pounds per vertical foot, a significant load added to the slab. The earliest map showing the palm trees clearly is dated 12/2004 but it is likely they were planted long before that. Again, from Google Earth maps, the palm trees were removed sometime mid-to-late 2017 but the planters remained, which would have required heavy machinery to remove.

4.6 Collapse Images

4.6.1 Cellphone Video from Neighboring Hotel

On the morning of the collapse (approximately five minutes before), tourists in the pool area for a neighboring hotel began filming when they heard a noise that they assumed at first was a car accident. What they found was spilling water and a pile of rubble in the garage of CTS. In the video, column line M is visible. The first two columns from the end of the ramp, M8 and M10 are clearly visible. Beyond that is a pile of rubble either obscuring or including column M11.1. Faintly visible behind the rubble pile is the fourth column M13.1. It is unclear the exact extent of the rubble or whether column M11.1 was part of it.
4.6.2 CCTV Footage from Neighboring Condo

Video surfaced quickly after the collapse of the neighboring condominium’s CCTV footage. The angle of the camera unfortunately does not provide a view of the pool deck, and the quality is low as it is a cellphone video of a computer monitor. That being said, it does show the sequence of the collapse well and helps narrow down the possible locations of the initiation point. The video shows the central portion of the building collapsed first. It appears one or two columns along the building edge collapsed first with neighboring columns following shortly after. The central portion of the building collapsed in three sections with each column line pulling down the next one behind it. This portion collapsed in about four seconds from start to finish. The eastern portion (columns O1 through P10) stayed standing for approximately six seconds leaning westwards towards the new hole, before finally succumbing to the instability and pancaking on top of the rubble pile. The collapse of the building took less than fifteen seconds leaving only the western wing of the building standing. The sequence is summarized in the following figure.
4.7 Postcollapse Image

After the collapse, all eyes were on CTS providing an immense amount of coverage. Unfortunately, most of the key evidence was buried under the rubble. One picture from the day after the collapse did hold useful information regarding this analysis. The picture shows the debris pile from the property edge. On the left is the standing portion of the building and the parking area that has sheared off the columns down onto the cars below. It is unclear whether this shear failure preceded the building collapse or vice versa. Though, it is clear that much of the reinforcement to prevent punching shear was ineffective. Seen at the base of the column marked 72 (column I14) is part of the reinforcement meant to prevent punching shear (Friedman p. 38). Additionally, all the rebar in the photo is clean of concrete. This indicates poor bondage between the rebar and concrete, likely from deterioration due to water intrusion in the slab.

4.8 Cleared Site Image

Once the site had been cleared and it had been deemed safe, select parties were allowed to visit the site to take pictures and video of what remained. In one of these videos, the southern retaining wall is visible with a clear difference between what collapsed and what was demolished. On the left side of the retaining wall on the property edge, the rebar connecting the slab to the wall
broke and dragged down the wall. This shows clear rebar deterioration due to water intrusion. On the right, the concrete zipped under the portion of the building that was demolished, as it was more intact and had good bondage with the concrete. For this to occur, the rebar would have needed to lose significant cross-sectional area to deterioration.

Figure 4.7.1: (a) (left) photo of rubble pile showing failed punching shear reinforcement on the left, (b) (right) close up of failed punching reinforcement

Figure 4.8.1: Photo south retaining wall showing the difference between what collapsed (left) and what was demolished (right) (Ostroff, J.)
CHAPTER 5 ANALYSIS

5.1 Analysis of Shadows

Revisiting Figure 4.6.2a, we can use the shadows visible and the orientation of the building to calculate an approximate time and date the picture was taken. Using the cutout as a reference, the shadow from the south wall was measured to be slightly less than 3 ft and the east wall shadow was measured to be approximately 1.5 ft.

Figure 5.1.1: Shadow of south wall measured with reference to the cutout

Figure 5.1.2: Shadow of east wall measured with reference to the cutout

Figure 5.1.3: Expected shadow profile for October 13th at 10:30 a.m.
Using this information in combination with the heights of the walls (4 and 3 feet, respectively) and the coordinates of CTS, shadow profiles were generated. Based on these profiles the photo showing the cutout unrepaired was taken in early-October between 10:30 a.m. and 11 a.m., meaning the cutout was still unrepaired three months after it was made at least.

5.2 Analysis of Rubble in Garage

If we do some image processing to Figure 4.8.1, we get the following:

Figure 5.1.4: Expected shadow profile for July 13th at 10:30 a.m.

Figure 5.2.1: Screenshot from video of initial collapse with heavy image processing.
Here, column line M is visible. The first two columns from the end of the ramp, M8 and M10 are clearly visible. Beyond that is a pile of rubble either obscuring or including column M11.1 seemingly to contain the large square planter sat atop column M11.1. Faintly visible behind the water stream is the fourth column M13.1. Unfortunately, the quality of the video is poor, and it is unclear the exact extent of the rubble or whether column M11.1 was part of it.

5.3 Planter Area

The planter area was selected for further investigation due to its proximity to both the initial pool deck collapse and the building collapse. Additionally, the area is more heavily loaded than surrounding areas. The loading pattern from the slab and planters was simplified. Span lengths were taken as the clear distance between columns. The slab was treated as a one-way action supported by the building and the elevation change beams. All concrete was treated as 3-ksi normal weight concrete reinforced with grade 60 rebar. From the first-floor framing plan and first floor plan (Friedman pp. 31 and 53 respectively), loading diagrams were drawn up and shear and moment diagrams were calculated from there. All connections between members were taken as fixed ends. The results for the moment and shear at support ends and the maximum positive moment are summarized in Table 1.

Figure 5.3.1: Model of Planter Area

The results are summarized in Tables 1 and 2. More detailed information for each beam is available in the appendix. I believe these results to be significantly lower than what would have been present in-situ. The soil would likely be saturated with water most of the time and the unit weight may be underestimated by as much as 50 percent. Additionally, the loads used were unfactored and neglected live loads and the slab surrounding the planter area. Members 3, 4, and 5 change depth where an elevation change intersects the member. It is unclear in the design drawings how this affects the reinforcement design, and this change was neglected. Most
importantly, the effects of deterioration were neglected and would have significantly affected the available strength in these members. Torsion was also neglected and would have a noticeable effect on conditions.

Several issues stand out from this analysis. Firstly, Member 3 is loaded to 82 percent of its moment capacity in this model. It was likely overloaded given it is a transfer beam for both of the elevation-changes. All of the beams are carrying a significant amount of their available shear strength, about 60 percent at the lowest. Again, beam 3 is loaded to 83 percent of its capacity.

A significant weakness in the design of the planter area is the deflection. All the beams exceed allowable deflections with the transfer beams all doubling or tripling their allowable maximum. Most of this difference was hidden with the changing elevation and the planters in the area so it was likely not apparent to the residents of CTS.

Table 1: Results from beam analysis on planters

<table>
<thead>
<tr>
<th>Label</th>
<th>Type</th>
<th>Span</th>
<th>M1 (k-ft)</th>
<th>V1 (Kips)</th>
<th>M2 (K-ft)</th>
<th>V2 (Kips)</th>
<th>Mmid (K-ft)</th>
<th>Δ (in)</th>
<th>L/180</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11” elv chg</td>
<td>20’ 6”</td>
<td>81.8</td>
<td>24.1</td>
<td>78.4</td>
<td>21.9</td>
<td>40.8</td>
<td>.171</td>
<td>.114</td>
</tr>
<tr>
<td>2</td>
<td>11” elv chg</td>
<td>21’ 4”</td>
<td>86.9</td>
<td>24.1</td>
<td>86.9</td>
<td>24.1</td>
<td>43.5</td>
<td>.161</td>
<td>.119</td>
</tr>
<tr>
<td>3</td>
<td>Type A</td>
<td>17’ 8”</td>
<td>104</td>
<td>25.7</td>
<td>99.5</td>
<td>23.9</td>
<td>65.6</td>
<td>.315</td>
<td>.098</td>
</tr>
<tr>
<td>4</td>
<td>Type A</td>
<td>17’ 8”</td>
<td>56.2</td>
<td>14.2</td>
<td>75.6</td>
<td>20.2</td>
<td>53.6</td>
<td>.282</td>
<td>.098</td>
</tr>
<tr>
<td>5</td>
<td>Type A</td>
<td>17’ 10”</td>
<td>84.3</td>
<td>24.1</td>
<td>55.9</td>
<td>12.9</td>
<td>54.7</td>
<td>.205</td>
<td>.099</td>
</tr>
</tbody>
</table>

$M_1$ and $V_1$ are the western end for the elevation change beams and the northern end for the Type A beams. Based on the cross-sections from the first-floor framing plan, the shear and flexural strengths were calculated and are summarized in table 2.
Table 2: Strength of Beams

<table>
<thead>
<tr>
<th>Label</th>
<th>$A_g$ (in$^2$)</th>
<th>$M_n$ (K-ft)</th>
<th>$\phi M_n$ (K-ft)</th>
<th>$V_c$ (psi)</th>
<th>$V_{\text{max}}$ (psi)</th>
<th>$M_{\text{max}}/\phi M_n$</th>
<th>$V_{\text{max}}/V_c$</th>
<th>$\Delta L/180$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>246</td>
<td>221.9</td>
<td>199.7</td>
<td>161.0</td>
<td>98.0</td>
<td>.410</td>
<td>.609</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>246</td>
<td>221.9</td>
<td>199.7</td>
<td>157.3</td>
<td>98.0</td>
<td>.435</td>
<td>.623</td>
<td>1.35</td>
</tr>
<tr>
<td>3</td>
<td>180</td>
<td>141.2</td>
<td>127.1</td>
<td>172.1</td>
<td>142.8</td>
<td>.818</td>
<td>.830</td>
<td>3.21</td>
</tr>
<tr>
<td>4</td>
<td>180</td>
<td>141.2</td>
<td>127.1</td>
<td>190.5</td>
<td>112.2</td>
<td>.595</td>
<td>.589</td>
<td>2.88</td>
</tr>
<tr>
<td>5</td>
<td>180</td>
<td>141.2</td>
<td>127.1</td>
<td>175.9</td>
<td>133.9</td>
<td>.663</td>
<td>.761</td>
<td>2.07</td>
</tr>
</tbody>
</table>

5.4 Interior and Roof 2-way Slabs

A 2-way slab analysis was performed on the interior slabs to aide in the later column analysis and give good estimates of the loads imparted on columns near the initial collapse. To perform a two-way slab analysis, a simple model was created to make calculations of forces practical using the direct design method (DDM). The model consisted of a 6” slab on 16” square columns with a center-to-center distance of 20 ft. This model succeeds in all requirements except for the ratio of short to long spans. The actual situation that is being modeled fails all, except the gravity loads requirement, but it is close to the model, especially near the area of interest: columns K-M9.1. The actual spans around these columns range between 19 to 23 feet and are all approximately square. The design of CTS is full of offset columns and short spans, so the assumptions made for these calculations are not the most accurate. This analysis was concerned with how the loading of the interior slabs affected the exterior columns. The load carrying capacity of the slabs on the interior of the building was of lesser concern as they did not appear to be of a structural concern based on the evidence looked at and were reinforced beyond the minimums set by the analysis.

To simplify calculations, loads were generally non-conservative as they did not consider live or super-imposed dead loads. The slabs on all floors had overhangs or balconies on the edge of the building that were not included in the model. Values are summarized in table 3 and calculations are included in the appendix.

The slabs within the building seem to be well designed, in most circumstances temperature steel governed. The interior slabs imparted an axial load of 15.4 kips and 34.9 kip-feet onto the exterior columns. These values are used in the later column analysis.
Table 3: 2-way slab forces

<table>
<thead>
<tr>
<th>One-way shear (kips)</th>
<th>Two-way shear (kips)</th>
<th>axial load on columns (kips)</th>
<th>moment load on columns (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_u = 1.38$ kips</td>
<td>$V_u = 61.1$ kips</td>
<td>15.4</td>
<td>34.9</td>
</tr>
<tr>
<td>$V_c = 6.09$ kips</td>
<td>$V_c = 83.7$ kips</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4: 2-way slab reinforcement requirements

<table>
<thead>
<tr>
<th>Required steel to resist flexural loads</th>
<th>Required steel to resist temperature change</th>
<th>Area of steel provided</th>
<th>Maximum spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>$1.46$ in$^2$ per linear foot</td>
<td>$1.296$ in$^2$ per linear foot</td>
<td>$2.48$ in$^2$ per linear foot</td>
<td>16”</td>
</tr>
</tbody>
</table>

5.5 Columns

The columns near the planter area are of concern as this is both where the initial collapse was approximately, and this is where the building collapse began.

5.5.1 Type C

Using the axial and moment results from the slab analysis, an approximate load can be calculated for the first column line. The following values were found.

<table>
<thead>
<tr>
<th>K9.1</th>
<th>L9.1</th>
<th>M9.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_u = 558.9$ kips</td>
<td>$P_u = 560.5$ kips</td>
<td>$P_u = 549.0$ kips</td>
</tr>
<tr>
<td>$M_u = 131.3$ k-ft</td>
<td>$M_u = 111.6$ k-ft</td>
<td>$M_u = 159.4$ k-ft</td>
</tr>
<tr>
<td>$e = 2.82”$</td>
<td>$e = 2.39”$</td>
<td>$e = 3.48”$</td>
</tr>
</tbody>
</table>
Here the columns fall close to, but within the allowable design line. These results are likely very low. The calculations do not consider things like the weight of the exterior wall, the cantilever overhangs, and changes in slab thickness. The loads on the columns were likely above the allowable line especially when issues of poor maintenance, pooling water, and deterioration are considered. However, the column was likely to still be within the nominal capacity line. If the type A beam from the planter areas collapsed during the pool deck collapse, moment demands would have increased by about 20 - 30%, not accounting for a reduction in strength from the collapse.
### 5.5.2 Type N

Using the results of the planter area analysis, the following values were used in the column type N analysis.

<table>
<thead>
<tr>
<th>Column</th>
<th>$P_u$ (kips)</th>
<th>$M_u$ (kip-ft)</th>
<th>$e$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K11.1</td>
<td>23.9</td>
<td>99.5</td>
<td>49.96</td>
</tr>
<tr>
<td>L11.1</td>
<td>20.0</td>
<td>75.6</td>
<td>44.9</td>
</tr>
<tr>
<td>M11.1</td>
<td>12.9</td>
<td>55.9</td>
<td>52.0</td>
</tr>
</tbody>
</table>

**Figure 5.5.2: Interaction diagram for columns K11.1 and L11.1**
Two out of the three type N columns from the planter area were severely overloaded. This is extremely concerning as these columns would both be under higher loads and would be weakened by long-term deterioration. Given this, it is surprising that CTS (or at least the pool deck) stood as long as it did under these conditions. In addition, column M11.1 had an extra planter on top that was offset and would impart moment-loading perpendicular to the loading analyzed above. While the analysis of the garage video was inconclusive, given this information, it seems unlikely that any disturbance to the delicate balance holding this column upright would result in its remaining standing.

5.6 Pool Deck

5.6.1 Temperature Steel Reinforcement

The most surprising finding in this analysis was that many areas of the pool deck were either under reinforced or barely reinforced. Specifically, the entire southern wall between the pool and Collins Ave. has about 101% of the required minimum temperature steel. Additionally, much of the center of the pool deck near where the debris can be seen in the video ranges from 89 to 105% of the required minimum. Figure 5.6.1 notes the findings.
Figure 5.6.1: floor plan highlighting areas susceptible to temperature changes in yellow
CHAPTER 6 SUMMARY AND CONCLUSIONS

6.1 Findings

1. CTS was built from 1980 drawings.
2. Water intrusion was widespread and significantly impacted the strength of the structure.
3. Signs of distress were plain and obvious in the garage area.
4. Columns near the initial collapse were likely heavily loaded beyond allowable stresses.
5. Several areas of the pool deck were inadequately reinforced against temperature change effects exacerbating the effects of water intrusion.
6. The pool deck was not sloped exacerbating the effects of water intrusion.
7. At least one location of exploratory demolition was not repaired in a timely fashion exacerbating the effects of water intrusion.
8. Several extra planters were installed on the pool deck and contained palm trees for over a decade, increasing the demands on the slab.
9. One or multiple of the columns along column line 9.1 between K and N failed first.
10. A portion of the pool deck collapsed before the building.
11. Evidence of column M11.1 collapsing is inconclusive.
12. Reinforcement against punching shear in the pool deck was bypassed.
13. Interior slabs were likely adequately designed.

6.2 Probable Cause

In terms of the mechanisms of the collapse, the obvious cause was the effects of long-term water intrusion. More specifically the following sequence of events is hypothesized:

1. The slab near column M11.1 gives way from the increased strain from the steep temperature drop and long-term deterioration.
2. This destabilizes M11.1, the heavy, waterlogged planter boxes, and the planter on top. All three collapsed into the garage.
3. One or more of the building columns are damaged as the connection beams are partially or completely ripped out.
4. The columns stand for several minutes before finally succumbing to their injuries and collapsing.

CTS sat for 40 years in a delicate balance of insufficient members supporting each other. Over time this balance became harder and harder to maintain as these members lost their strength to deterioration. Ultimately, Champlain Towers South was designed negligently, it was constructed negligently, and it was maintained negligently. Tragedy was almost inevitable.
6.3 Recommendations

Local building codes require all assumptions and calculations used to design a structure be submitted with the design drawings. This will help with the inspection and investigation processes giving insight to anyone trying to gauge the integrity of a structure long after it is built.

Building codes should incorporate standardized language for describing the condition of a structure. The dangers of CTS’s condition were known at least three years before its collapse. This information had been communicated but there was a disconnect about the significance of the damage and the importance of rapid remediation.

6.4 Future Research

Without access to much of the necessary evidence, future research is difficult. However, there are three areas further research might be fruitful. First, combing through the minutiae of 40 years of permit applications and email correspondences might yield some key details not apparent in other documents. Records of local newspapers might lead to a better understanding of the construction process and the development of CTS over time. Additionally, sources like Facebook, Twitter, YouTube, Instagram, and other social media platforms might hold useful information from vacation photos and similar media that could possibly provide details on the structure. Second, more complex and in-depth modeling can highlight other points of interest in the structure not found in this investigation. Third, this investigation focused on one hypothesis. It neither seeks alternatives nor seeks to disprove it. Future investigations should do both of these to find the hypothesis that stands up to the most rigorous standards possible.
CHAPTER 7 REFERENCES

American Concrete Institute. (1979). *Building code requirements for reinforced concrete: ACI 318-77*.

American Concrete Institute. (1979). *Commentary on building code requirements for reinforced concrete: ACI 318R-77*.

American Concrete Institute. (2014). *Building code requirements for structural concrete: (ACI 318-14) \( \text{and commentary (ACI 318R-14)} \).


CHAPTER 8 APPENDIX:

8.1 Results from Beam Analysis

8.1.1 Beam 1

Loading for beam 1
Beam 1 shear, moment, and deflection diagrams

- Shear diagram
- Moment diagram
- Deflection diagram
8.1.2 Beam 2

Loading for beam 2
Beam 2 shear, moment, and deflection diagrams

Shear diagram:
- Green line represents shear force.
- Axes: Shear (lb) vs. Distance from Left of Beam (ft).

Moment diagram:
- Red line represents bending moment.
- Axes: Moment (lb-ft) vs. Distance from Left of Beam (ft).

Deflection diagram:
- Blue line represents deflection.
- Axes: Deflection (in) vs. Distance from Left of Beam (ft).

Reactions:
- Blue dots indicate reaction forces.
- Labels: 63,200 lb-ft and 23,400 lb.

Axial Load:
- Orange arrows indicating axial forces.

Vertical Load:
- Red arrows indicating vertical forces.

Distance from Left of Beam (ft):
- X-axis in increments of 5 ft from 0 to 20 ft.
8.1.3 Beam 3

Loading for beam 3
Beam 3 shear, moment, and deflection diagrams

Shear diagram:
- Load Case: D
- Envelope

Moment diagram:
- Load Case: D
- Envelope

Deflection diagram:
- Short-Term LC: D
- Envelope

Reactions:
- Axial Load
- Vertical Load

Distances and loads:
- 21900 lb
- 24100 lb
- 23900 lb
- 25700 lb
8.1.4 Beam 4

Loading for beam 4
Beam 4 shear, moment, and deflection diagrams

Shear Diagram:
- Envelope
- Load Case D

Moment Diagram:
- Envelope
- Load Case D

Deflection Diagram:
- Envelope
- Short-Term LC D

Force Diagram:
- Reactions
- Axial Load
- Vertical Load

24100 lb
66200 lb/ft
14200 lb
7560 lbs
8.1.5 Beam 5

Loading for beam 5
Beam 5 shear, moment, and deflection diagrams

[Shear diagram]

[Moment diagram]

[Deflection diagram]

[Load diagram]
8.2 Calculations for 2-way Slab Analysis

\[ d_l = t_{slab} - c_{clear} - d_b - db/2 \]
\[ = 6'' - .75'' - .625'' - .313'' = 4.31'' \]

\[ d_l = t_{slab} - c_{clear} - db/2 \]
\[ = 6'' - .75'' - .313'' = 4.94'' \]

\[ d_{avg} = 4.63'' \]

\[ \omega = 1.2 \times \frac{6''}{12''} \times 150 \text{ pcf} + 1.6 \times .04 \text{ psf} = .154 \text{ ksf} \]

\[ A_{T1} = 20''/2 - (16''/2*12'') - 4.63''/12'' = 8.948 \text{ ft}^2 \]
Shear tributary (1w)

\[ V = A_{T1} = .154 \times 8.948 = .138 \text{ kips} \]
\[ V_c = 2f'c_b w d = 2 \times 1 \times 3000 \times 12'' \times 4.63'' = 6.09 \text{ kips} \]
\[ V_c > V \]

\[ A_{T2} = 20' \times 20' - ((16'' + 4.63'')/12'')^2 = 397.04 \text{ ft}^2 \]
Shear tributary (2w)

\[ V_u = A_{T2} = .154 \times 397.04 = 61.14 \text{ kips} \]
\[ V_c = 4f'c_b w d \]
\[ = 4 \times 1 \times 3000 \times (4 \times 20.63'') \times 4.63 = 83.7 \text{ kips} \]
\[ V_c > V \]

\[ A_t = 20' \times 20' /2 = 200 \text{ ft}^2 \]
\[ P_u = * A_t = .154 \times 200 = 30.8 \text{ kips} \]
\[ M_o = \frac{l_2 h^2}{8} = .154 \times 20 \times 18.67^2 / 8 = 134.2 \text{ K-ft} \]

Exterior Negative factor = .26
\[ M_{\text{ext}} = 34.9 \text{ K-ft} \]

\[ b = 20 \times 12 / 2 = 120 \text{ in} \]
\[ A_s = 8 \times .31 = 2.48 \text{ in}^2 \]

\[ a = A_s f_y / .85 f_c; b = 2.48 \times 60 / .85 \times 3 \times 120 = .486'' \]
\[ c = a / .85 = .572'' \]

\[ t = .003 d / c - .003 = .003 \times 4.94 / .572 - .003 = .022 \]

Therefore, tension controlled.

\[ A_s = M_{\text{ext}} / f_y (d_l - a/2) = 34.9 / 60 (4.94 - .148) = 1.46 \text{ in}^2 \]

Minimum for temperature steel:
\[ .0018 \times 120 \times 6 = 1.296 \text{ in}^2 \]

\[ s_{\text{max}} = 2h = 16'' < 18'' \]
\[ 120'' / 8 = 15'' < 16'' \]
CHAPTER 9 VITA

The Author was born in Grand Rapids, Michigan. He obtained his bachelor’s degree in physics and computer science from Loyola University New Orleans to pursue a Master’s in Civil Engineering.