walter p moore

305.704.9200 main

June 4, 2026

Cornelius Benton
Director of Restoration & Repair
Edge Construction, LLC
525 NE 13th Street
Fort Lauderdale FL 33304

Re: Broward Health Atrium: Foundation Analysis and Vertical Expansion Feasibility Study
1600 S Andrews Ave, Fort Lauderdale FL 33316
Walter P Moore Project No. D26.26017.00

Mr. Benton:

Walter P Moore (WPM) was engaged by Edge Construction, LLC ('the Client') to verify the capacity of the existing foundations and to perform a feasibility study for a vertical expansion applied to the Broward Health Medical Center located at 1600 S. Andrews Avenue, Fort Lauderdale, Florida, 33316. Services have been provided in accordance with the Letter of intent issued by the Client on February 26, 2026.

Our scope included reviewing construction documents in addition to the 2025 reports submitted by GMG Structural Engineers, Master Consulting Engineers, Inc., TLC Engineering Solutions, and TRC Worldwide Engineering related to perceived deficiencies in the pile foundations. We also completed an evaluation of the building, both in its existing condition and with a vertical expansion over a portion of the structure. Our evaluation included an analysis of a numerical model representing the above-grade superstructure and below-grade pile foundations.

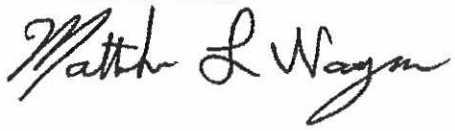
Our findings indicated that the building in its existing condition does not require strengthening of the foundation system. This conclusion was based on a comparison between allowable strengths of piles specified in the geotechnical report by Nutting Engineers of Florida, Inc. submitted prior to initial construction and estimated demands required by the current building code including wind loads specified for structures located in Broward County.

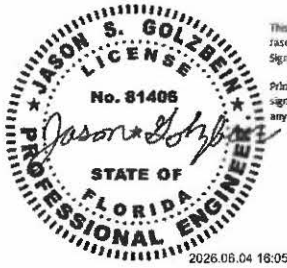
Our analysis of the building with a vertical expansion consisting of three additional stories and considering 100% wind load as required by the building code, indicated that 11 piles located beneath one of the shear wall cores had insufficient compression and 9 piles had insufficient tension capacities. Based on a 50% wind pressure reduction in prescriptive code-based pressures (maximum allowed by wind tunnel testing), a single pile was observed to be overstressed in compression only and its demand-to-capacity ratio was 1.01. This reduction in demand suggests the feasibility of minimizing the extent of strengthening applied to the foundation system if wind loads are reduced by performing a wind study.

All piles located supporting steel gravity columns had sufficient capacities. Reinforced concrete shear walls should have sufficient compression, tension, and shear capacities to resist the corresponding demands for both the existing building and building with vertical expansion.

We very much appreciate this opportunity to provide these services to you. Please do not hesitate to contact us if we can further assist you.

Sincerely,
WALTER P. MOORE AND ASSOCIATES, INC.





This item has been electronically signed and sealed by Jason Golzbein, P.E. on the date indicated using a Digital Signature.

Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

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Matt Wagner, S.E. (IL)
Principal/Managing Director

Jason Golzbein, P.E.
Senior Associate

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Introduction

Scope of Work

Based on the signed engagement with the Client, Walter P Moore's scope included completing two main tasks as outlined below:

Task 1 – Foundation Analysis and Conceptual Repair Strategies

Should the foundation concern remain unresolved by others, Walter P Moore will:

1. Review available construction documents, specifications, third-party reports/analyses, and other related materials.
2. Conduct a comprehensive peer review of foundation analysis carried out by others addressing the reported issues identified in the 2025 reports.
3. Perform a structural analysis to assess:
    a. Foundation behavior focused on addressing the reported issues.
    b. Suitability of the current existing building carrying loads.
4. Create conceptual foundation retrofit strategies, if necessary.
5. Prepare a presentation style report of our findings and recommendations.

The scope is confined to the analysis of the foundations for gravity and wind loads related to the current structure.

1. We will identify any apparent deficiencies within the existing structure found through our analysis; however, it should be noted that this feasibility study cannot guarantee the identification of all potential deficiencies.
2. The model developed is used for the general behavior of the structure and focuses on the behavior of the foundations. The analysis is not exhaustive of all members.
3. The results of this study are conceptual and not intended for final design or construction purposes, and a more detailed analysis will be required if the project proceeds.

Task 2 – Three Story Addition Feasibility Analysis

This scenario presumes that any reported foundation deficiencies cited in the 2025 reports have been addressed. Walter P Moore will:

1. Review available construction documents, specifications, third-party reports/analyses, and other related materials.
2. Perform a structural analysis to assess the feasibility of adding 3 additional floors to the structure.
3. Create conceptual structural designs and identify conceptual retrofits for the vertical expansion.
4. Prepare a presentation style report of our findings and recommendations.

Assumptions: At present, we are undertaking a feasibility-level structural analysis solely, independent of architectural, mechanical, electrical, plumbing, and other design disciplines.

Coordination with these disciplines will be necessary in subsequent project phases if the concept advances, as their requirements could impact structural feasibility.

The scope is confined to conceptual design of the structural system and general member sizing based on gravity and wind loads related to the proposed addition of 3 floors.

1. For the purposes of this proposal, it is assumed that the additional 3 floors are limited to the current 5-story tower (Figure 1).
2. Walter P Moore will complete one iteration of analysis and conceptual design for a 3-story addition to the tower. Any revisions or additional analyses requested by the client will be considered additional services subject to separate agreement and compensation.
3. We will identify any apparent deficiencies within the existing structure found through our analysis; however, it should be noted that this feasibility study cannot guarantee the identification of all potential deficiencies.
4. The model developed is used for the general behavior of the structure and focuses conceptual design of the additional floors to the tower. The analysis is not exhaustive of all members.
5. The results of this study are conceptual and not intended for final design or construction purposes, and a more detailed analysis will be required if the project proceeds.

Project Background

The Broward Health Medical Center is a multi-story hospital building located in Fort Lauderdale, Florida constructed in 2005. Our scope included evaluating the Broward Health Atrium located in the northeast portion of the complex, which included a 5-story tower designed for future vertical expansion by up to 3 additional levels (Figure 1). The primary structure consists of composite steel/lightweight concrete deck, steel framing, and reinforced concrete shear walls. Steel columns and reinforced concrete shear walls are supported by auger cast piles.

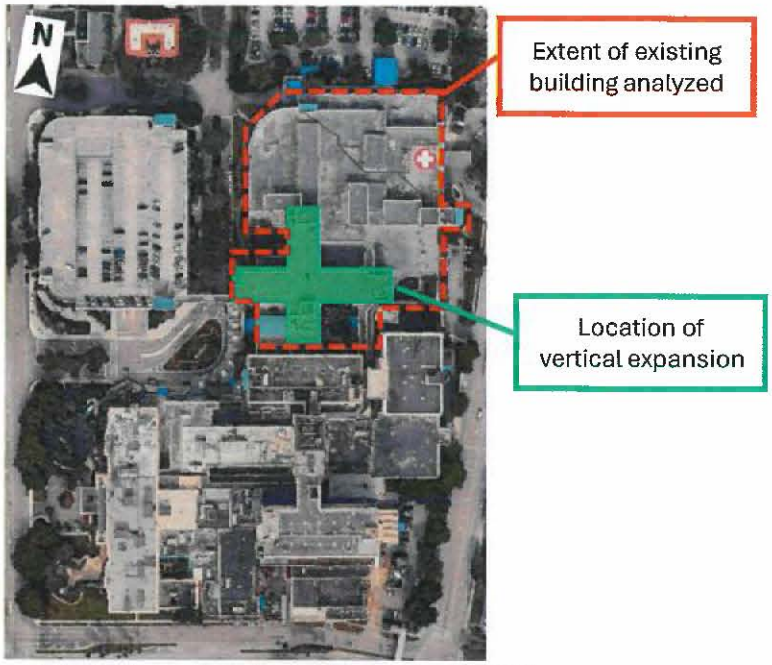


Figure 1: Site Plan (taken from Google Maps on May 20, 2026)

Review of construction documents

To verify the capacity of the foundation system and determine whether it required strengthening for the existing condition of the structure or with vertical expansion, we reviewed construction documents including the following items:

- *Pre-construction building drawings*
  - Design criteria, including
    - Applicable building codes
    - Gravity loads
    - Wind loads
  - Design material properties
  - Dimensions

- *Standards and Building Codes*
  - Florida Building Code, 2001, Second Edition
  - Florida Building Code, 2023, Eighth Edition
  - ASCE 7-1998, Minimum Design Loads for Buildings and Other Structures
  - ASCE 7-2022, Minimum Design Loads and Associated Criteria for Buildings and Other Structures

There was a major change in the estimation of wind loads as specified in the Florida Building Code (FBC) and ASCE 7 that occurred between the time of initial design and current practice. ASCE 7-1998 used smaller "service-level" wind loads multiplied by an importance factor of 1.15 for essential facilities like hospitals and a load factor of 1.6. ASCE 7-2022 uses larger "ultimate" wind loads which separately defined based risk categories. Despite the change in wind load classification for wind loads, the difference in magnitudes of wind pressures to be applied to the building estimating using the two codes is smaller than 10%, and wind loads for high-velocity hurricane zones based on Section 1620 of the 2023 FBC, with references to ASCE 7-2022 are used throughout this report.

Review of 2025 reports

A review of the reports written by the four structural engineering firms submitted prior to our engagement with the Client was completed. A summary of each report is given below including our opinion.

1. The report submitted by GMG Structural Engineers included results of destructive field testing of the pile head in one location where they documented steel reinforcement. They claimed to observe #3 bars embedded in the pile (instead of #4 bars as specified in the drawings) and concluded that shear wall foundations had insufficient tensile capacity for current wind demands.
   a. *We believe their evaluation is overly conservative as it was based on a single exploratory probe and a bid-set detail. Based on reports received from Nutting Engineers of Florida, Inc., the piles developed their intended capacity, thus we do not agree with GMG's conclusion.*

2. The report submitted by TRC Worldwide Engineering included results of finite element modeling of the superstructure and foundation systems. They concluded that the structure had insufficient capacity to resist wind demand based on excessive lateral drift and insufficient tension capacity of piles. They assumed piles had steel reinforcement consisting of #3 bars as was documented in the report submitted by GMG Structural Engineers.
   a. *We believe that TRC's assumption that all piles contained #3 bars is overly conservative given that Nutting Engineers reported that piles developed their intended capacity.*

3. The report submitted by Master Consulting Engineers, Inc. included non-destructive evaluation of a single exposed pile head and a numerical analysis of the superstructure and foundation systems. The results of their evaluation showed that #4 bars were embedded in the tested pile (different than the pile probed by GMG). They

concluded that the piles had sufficient capacity to resist wind loads considered in original design.

   a. *We agree with these conclusions. MCE did not report conclusions on vertical expansion of the existing structure.*

4. The report submitted by TLC Engineering Solutions included review of original geotechnical and pile installation monitoring reports (both submitted by Nutting Engineers of Florida, Inc.) and a linear elastic finite element analysis of the superstructure and foundation systems. TLC concluded that the installation monitoring report confirmed that piles reached their design capacities as specified in the drawings. Based on the results of their linear analysis, they concluded that some of the piles located beneath shear wall cores had insufficient capacity to resist combined tension and bending demands caused by wind loads applied to the existing building (without vertical expansion) using specifications given in ASCE 7-1998 and original drawings.
   a. *We agree that the piles in tension have reduced shear capacities but TLC did not indicate that piles in compression have higher shear capacity. It would be more appropriate to look at the total shear on the group.*

Analysis by WPM

To provide the client with 1) an additional opinion on whether the foundations are sufficient to resist demands of the existing building and 2) a feasibility study for vertical expansion, we performed a numerical analysis of the above grade superstructure and below grade pile foundations. Our analysis assumptions and model description are explained below.

Analysis assumptions

In general, we based our analysis on the design criteria and material properties given in sheet "S0.1 - General Notes" of the drawings titled "Phase II – Expansion to Broward General Medical Center" submitted by KSI Structural Engineers with the following caveats:

1. Wind loads were estimated using the provisions given in the 2023 Florida Building Code (FBC)
   a. Section 1604.5 of 2023 FBC
      i. Based on descriptions included in Table 1604.5 - Risk Category of Buildings and Other Structures, the building is considered an "Essential Facility" with Risk Category IV
   b. Section 1620.2 of 2023 FBC
      i. Based on buildings with Risk Category IV located in Broward County, Wind velocity (3-second gust) used in structural calculations shall be 185 mph
   c. Section 1620.3 of 2023 FBC
      i. Based on the provision given in Section 1620.3 and definitions given in Section 26.7 of ASCE 7-2022, the building is considered to be Exposure Category C.

2. For estimating wind pressures applied along the building height, the main structure up to Level 4 ("Fourth Floor" as indicated on drawings) was considered as a single low-rise building with a total height (including roof) less than 60 feet. Two sections (outlined in red in Figure 2) had building elevations exceeding 60 feet and were considered as separate from the main structure with wind pressures estimated from Level 5 ("Fifth Floor) to their corresponding roofs.

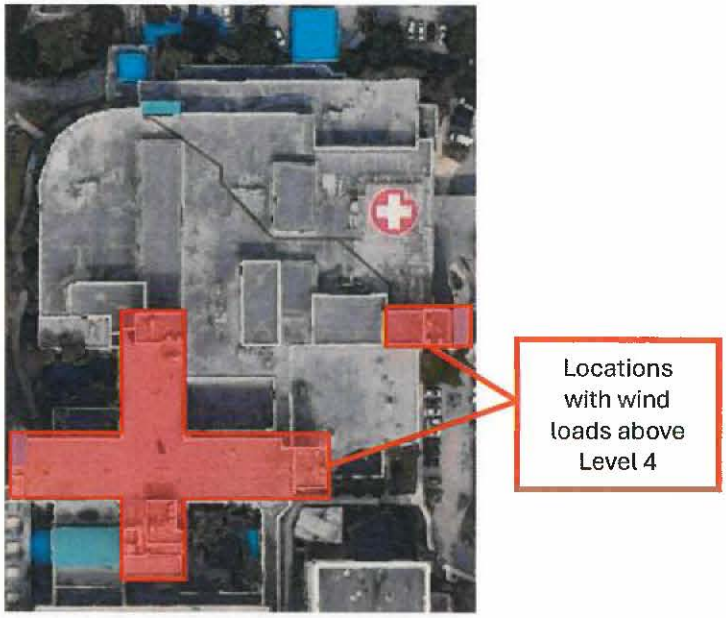


Figure 2: Locations with building elevations exceeding 60 feet

3. Load combinations were based on load and resistance factor design (LRFD) and allowable stress design (ASD) combinations as specified in ASCE 7-2022.
    a. Pile capacities were specified as allowable strengths and were evaluated using service-level load combinations (ASD).
    b. For the expansion model, our evaluation included evaluating the compression, tension, and shear at bases of core shear walls using ultimate strength load combinations (LRFD).
4. Reductions in live load were considered to maximize the resultant uplift in pile foundations using provisions given in ASCE 7-2022.
    a. For live loads equal to 100 pounds per square foot or less, live loads applied to floor slabs were multiplied by 0.4 and live loads applied to roof slabs were multiplied by 0.6 for areas tributary to steel columns and to reinforced concrete (RC) shear walls.
    b. For live loads exceeding 100 pounds per square foot, live loads were multiplied by 0.8.
5. We assumed that the allowable capacities of piles were as specified on the drawings by the geotechnical engineer which included a vertical compression capacity of 250 kip, a vertical tension capacity of 84 kip, and a lateral capacity of 24 kip.

Model description of existing building

We generated a numerical model of the northeast portion of the Broward Health Medical Center (Figure 3) based on the specifications and dimensions submitted in the structural drawings received by the Client (Figure 4). The model included steel framing and reinforced concrete shear walls (Figure 5) supported on pile foundations (Figure 6). A description of each component is as follows.

Primary gravity system

The main gravity supporting system comprised concrete slab on composite steel deck, steel beams, and steel columns. Floor slabs consisted of 18-gage steel deck with 4.25-inch-thick lightweight concrete. Diaphragms were modeled as membranes with in-plane stiffness. Diaphragms on Levels 2-3 ("Second Floor" and "Third Floor") were modeled as rigid membranes as there were virtually no floor openings (Figure 7) and preliminary model results showed no appreciable changes in the distribution. This simplification helps to reduce the complexity of the model. There were multiple floor openings on Level 4 and above so each was modeled as a semi-rigid membrane (Figure 8).

Steel beams framing into steel columns were modeled as shear connections. Secondary beams (not framing directly into steel columns) were not modeled as distinct elements as their influence on overall response was assumed to be negligible, but their weight was included based on conservative estimates of floor loads of 10 pounds per square foot.

Gravity loads as specified in the design criteria of the received structural drawings were modeled as floor loads and applied to appropriate sections of floor slabs based on their use as shown in the drawings. An example of the loads assigned to different sections of floor slabs on Level 2 is shown in Figure 9.

In general, spans between steel columns varied between 20-30 feet. Columns were modeled as continuous elements with pinned connections at their bases representing base plates bolted to tops of reinforced concrete pile cap foundations with thicknesses between 3-5 feet supported by 48-foot long, 16-inch diameter auger cast piles. Piles caps located beneath steel columns varied in plan area and number of supporting piles.

Primary lateral system

The main lateral force resisting system comprised 12-inch-thick reinforced concrete shear core walls with widths varying between 10-13 feet and lengths varying between 20-26 feet. Each core wall was assumed to have the same degree of modest cracking and the resultant stiffness reduction factor for in-plane forces was set at 70% of gross, elastic properties. Openings used for doors, elevators, and windows in shear walls were ignored as the pile was the greater concern. A full analysis of the cores will need to performed in the next phase.

Shear walls were supported by 4-foot-thick pile cap foundations on multiple 48-foot long, 16-inch diameter auger cast piles. Based on the geotechnical report submitted by Nutting Engineers, we assumed that the longitudinal reinforcement in piles located beneath shear walls consisted of four #4 bars extending 24 feet from top of pile in addition to one #9 bar extending the full length of pile. Transverse reinforcement consisted of #4 ties spaced at four inches on center. Piles were generally located along centerlines of shear walls, with some piles located between edges of shear walls and edges of pile cap foundations (Figure 6).

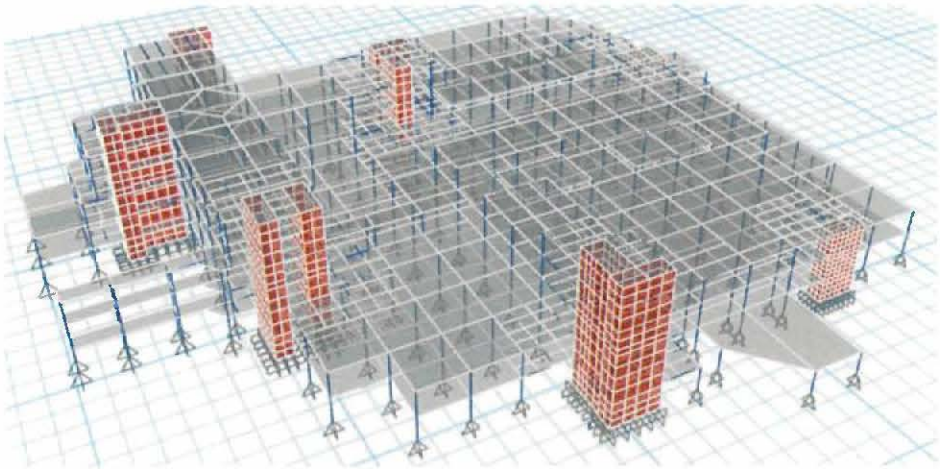


Figure 3: Isometric of numerical model of existing building

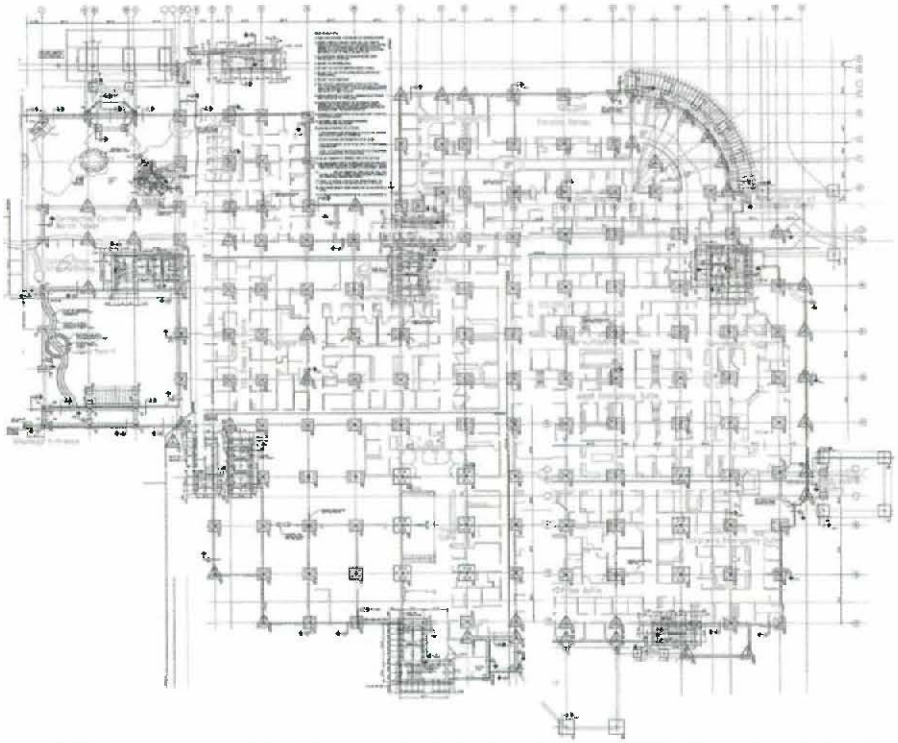


Figure 4: Plan view of ground floor from received structural drawings ("First Floor Foundation Plan")

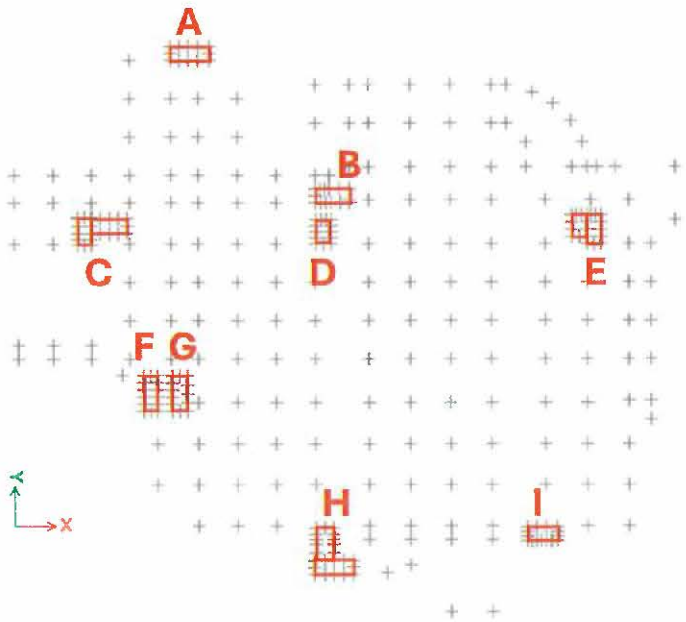


Figure 5: Layout of steel columns and reinforced concrete shear walls

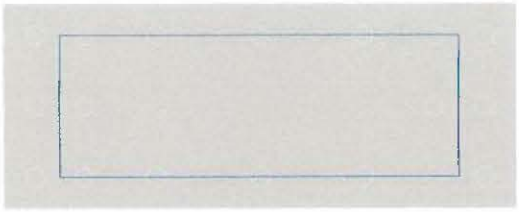


Figure 6: Plan view of centerline of Core A shear wall (purple rectangle) supported by pile cap foundation (gray rectangle) on piles (white circles)

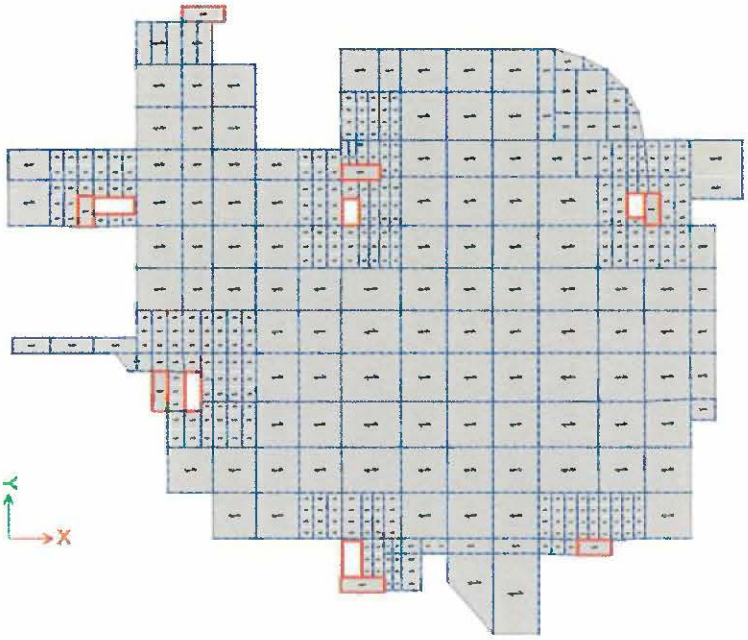


Figure 7: Plan view of Level 2 ("Second Floor" from structural drawings)

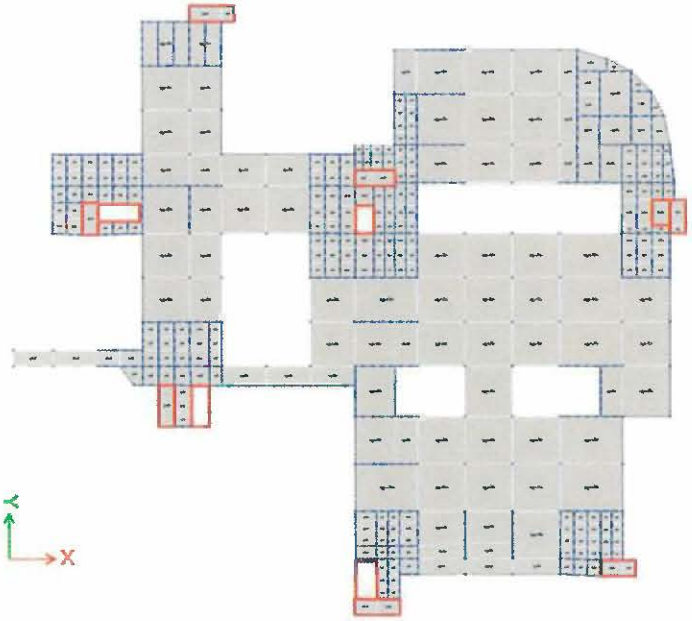


Figure 8: Plan view of Level 4 ("Fourth Floor" from structural drawings)

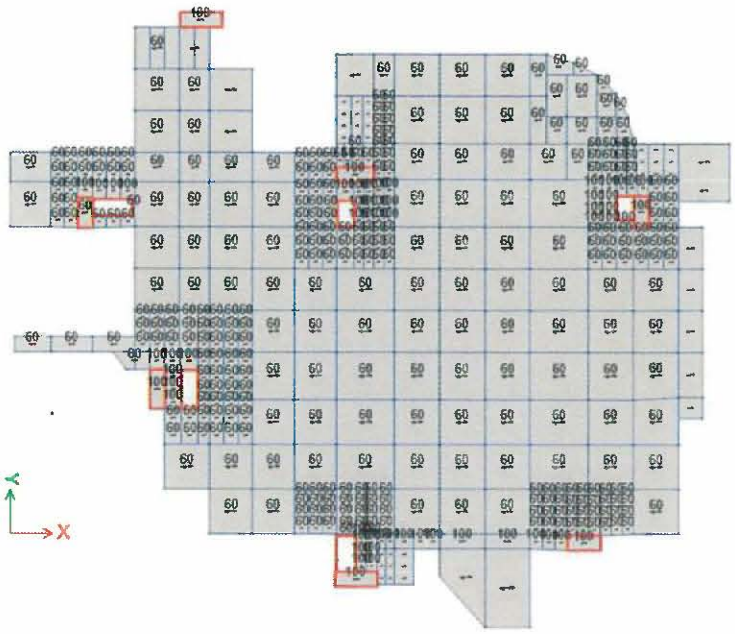


Figure 9: Load key plan for Level 2

Model description of building with vertical expansion

The same assumptions and modeling techniques used for the existing building were applied to the building with vertical expansion. The vertical expansion consisted of adding three stories above the cruciform section of the existing building (Figure 1). The additional stories were identical to each other (Figure 10) and use the same elements and dimensions as those of Level 5 ("Fifth Floor" of received structural drawings) of the cruciform section. Wind loads were calculated for the expansion model considering a new roof height at an elevation of 112 feet above the ground floor and were applied to areas affected by the vertical expansion.

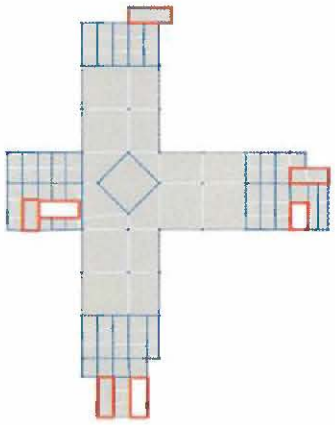


Figure 10: Plan view of additional stories added to cruciform section of existing building

Results

The results of the numerical analyses conducted on models representing the existing building and building with vertical expansion are summarized below.

Existing Building

The focus for modeling the existing building was to verify if the pile foundations located beneath shear wall cores had sufficient tension capacity to resist uplift demands caused by lateral wind loads. In addition to these load checks, the compression capacities of pile foundations beneath shear wall cores and steel columns were verified.

Piles supporting steel columns

To verify that pile caps located below steel columns had sufficient capacity, the vertical compression demands at bases of columns were compared with the total compression capacity of the corresponding pile cap calculated based on the number of supporting piles. It is likely that the original design used the full live load (instead of using reduced live loads as allowed by the building code), so the pile foundations were observed to have a large amount of reserve capacity, where the largest demands on the piles did not exceed 60% of their capacities (Figure 11).

Piles supporting concrete shear walls

Vertical reactions

Reactions at bases of shear walls were distributed into pile caps and pile foundations to verify whether the piles had sufficient capacity. Piles were observed to have sufficient compression capacity relative to peak compression demands considering all ASD load combinations with reduced live loads. Compression demands were maximized in piles located at corners of pile cap foundations, and peak compression demands were smaller than approximately 80% of the capacity for a corner pile below the Core C pile cap foundation (Figure 12). For vertical tension loads, all piles had sufficient tension capacities with peak demands not exceeding 70% of their capacities at the same corner pile located below the Core C pile cap foundation (Figure 13).

Shear

To verify whether pile foundations had sufficient capacity to resist shear demands, total shear demands at bases of shear walls were compared with total shear capacities of pile caps. Based on ASD load combinations with reduced live loads, all pile caps were observed to have sufficient shear capacity, where the Core C pile cap foundation was observed to have the largest demands for shear at approximately 70% of the capacity (Table 1).

Concrete shear walls

Compression, tension, and shear demands were compared with their corresponding capacities for shear walls. The ultimate design compression strength was calculated to be approximately 2000 pounds per square inch of cross section of wall. Based on LRFD load

combinations with reduced live loads, no walls were observed to have peak compression demands exceeding 1000 psi (Figure 14) indicating sufficient compression capacity.

The ultimate design tensile strength at corners of shear walls (where demands were observed to be maximized) was calculated to be approximately 110 kip per foot of wall based on the eight #9 bars shown in the drawings (Figure 15). The maximum tensile demand was observed to occur at the corner of Core C shear wall and was approximately 60 kip per foot of wall indicating shear walls had sufficient tensile capacity (Figure 16).

The ultimate design shear strength was calculated to be approximately 40 kip per foot of wall based on contributions of concrete and horizontal steel distributed along faces of walls. Based on LRFD load combinations with reduced live loads, no walls were observed to have peak shear demands exceeding 20 kip per foot of wall (Figure 17) indicating walls have sufficient shear capacity.

Table 1: Demand-to-capacity ratios for shear in pile caps for existing building

Pile cap foundation	Number of piles	Total shear capacity of pile cap (kip)	Total shear demand on pile cap (kip)	DCR
A	18	432	253	0.59
B	18	432	208	0.48
C	22	528	367	0.69
D	16	384	147	0.38
E	34	816	300	0.37
F	18	432	213	0.49
G	18	432	210	0.49
H	36	864	469	0.54
I	18	432	179	0.42

Note: "DCR" is demand-to-capacity ratio

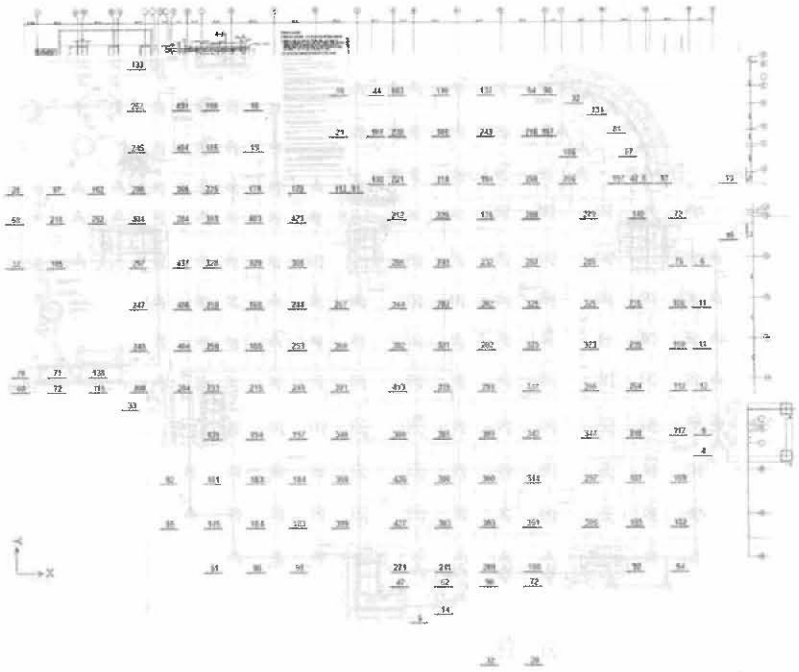


Figure 11: Peak compression loads at bases of steel columns for existing building

Note: Reactions were estimated considering all ASD load combinations ***with reduced live loads***

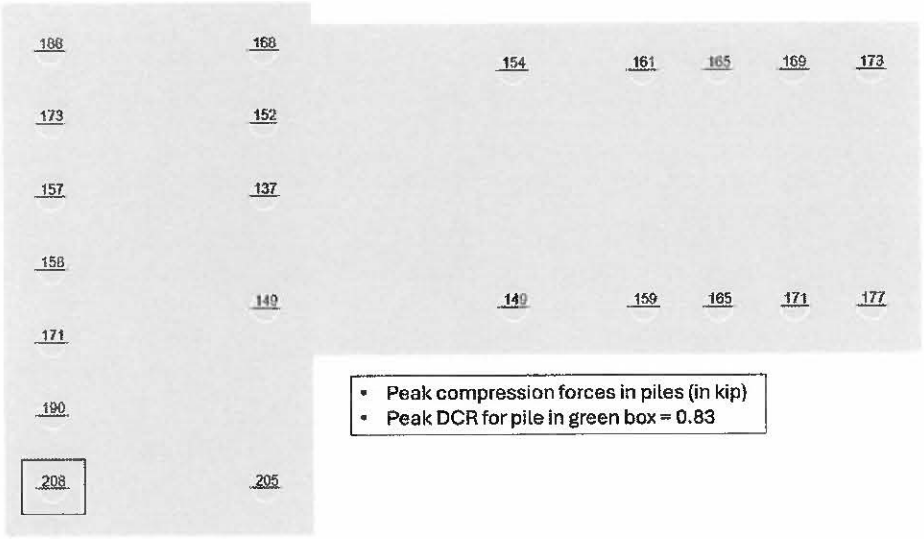


Figure 12: Peak compression demands for piles under Core C pile cap foundation for existing building

Note: Reactions were estimated considering all ASD load combinations ***with reduced live loads***

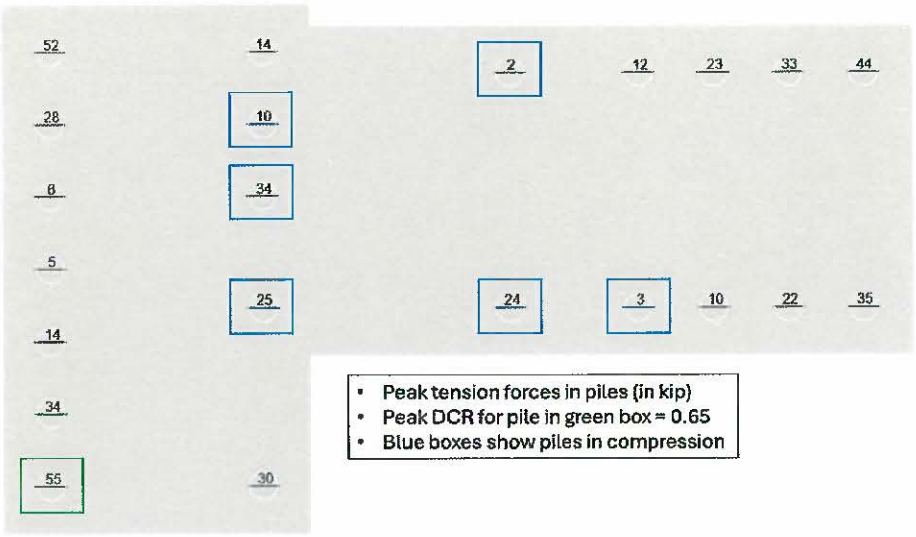


Figure 13: Peak tension demands for piles under Core C pile cap foundation for existing building

Note: Reactions were estimated considering all ASD load combinations ***with reduced live loads***

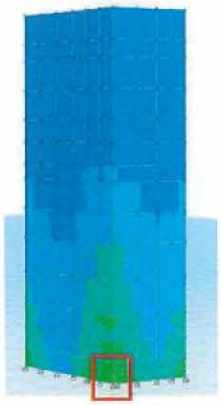


Figure 14: Peak compressive stress at corner of Core C shear wall for existing building

Note: Reactions were estimated considering all LRFD load cases ***with reduced live loads.*** Region with red box shows peak compressive demand equal to approximately 1000 psi.

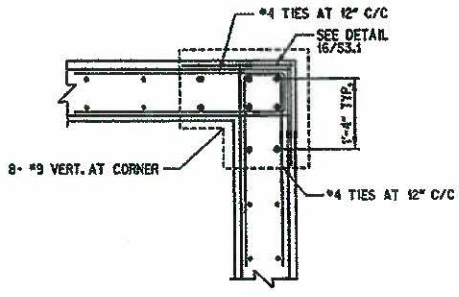


Figure 15: Longitudinal reinforcement detail at corners of shear walls



Figure 16: Peak tensile demands at corner of Core C shear wall for existing building

Note: Reactions were considering all LRFD load cases ***with reduced live loads.*** Region with red box shows peak tensile demand equal to approximately 60 kip per foot of wall.



Figure 17: Peak shear demands for Core H shear wall for existing building

Note: Reactions were estimated considering all LRFD load cases ***with reduced live loads.*** Region with blue box shows peak shear demand equal to approximately 20 kip per foot of wall.

Building with Vertical Expansion

The vertical expansion was applied to the existing building and consisted of 3 stories added to the cruciform section of the building (Figure 18). The focus for modeling the building with vertical expansion was to verify if the pile foundations have sufficient compression and tension capacities. In addition to piles, we verified if shear walls had sufficient compression, tension, and shear capacities.

Two wind cases were applied to the building. One case assumes the code required wind loading for a Wind Exposure Category C building in Broward County (100% Wind Load). The other case assumes a 50% reduction in the code required wind loads (50% Wind Load) reflecting the best-case results of a wind study conducted on the building and its surrounding topographic conditions. If permitted by the Broward Building Department, designing the expansion using the results of the wind study would reduce the amount of retrofitting. The cost

of such a study would be a small fraction of the cost associated with extensive strengthening of the foundation system.

Piles supporting steel columns

To verify that pile caps located below steel columns had sufficient capacity, the vertical reactions at bases of columns were compared with the total compression capacity of the corresponding pile cap calculated based on the number of supporting piles. Because steel columns were not part of the lateral force resisting system, gravity demands were the same for both Case 1 and Case 2. The added floors did increase the column demand but since the original design did not reduce the live loads, there is a large amount of available foundation capacity . The largest demands were observed to be smaller than approximately 70% of their capacity (Figure 19).

Piles supporting concrete shear walls

Case 1: 100% Wind Load

Reactions at bases of shear walls were distributed into pile caps and pile foundations to verify whether the piles had sufficient capacity. For 100% wind loads, only piles located beneath Core C were observed to have demands which exceeded their capacities. 11 piles were observed to have insufficient compression strength (Figure 20). Compression demands were between 5-40% larger than the capacities, and 5 piles had demands at least 10% larger than their compression capacity. For tension loads, there were 11 piles that had insufficient strength (Figure 21). The largest tension demand was observed to be twice as large as its capacity, and 4 piles had tension demands at least 50% larger than their capacity of 84 kip.

To verify whether pile foundations had sufficient capacity to resist shear demands, total shear demands at bases of shear walls were compared with total shear capacities of pile caps. All piles caps had sufficient shear capacities except for the pile cap under Core C shear wall which was overstressed producing demands nearly 20% larger than the capacity (Table 2). Here, shear capacities are taken as the specified lateral strengths given by the geotechnical engineer, or 24 kip per pile. Concrete piles resisting axial tension have reduced shear capacity but piles in compression have increased shear capacity, and the latter will resist most of the total base shear acting on the pile group. Given this, the lateral capacities of the piles may be sufficient to resist the total base shear for 100% wind loads.

Case 2: 50% Wind Load

For 50% wind loads, a single pile located beneath Core C were observed to have insufficient compression strength (Figure 22) and only by 2 kip or 1% of the total compression capacity. All piles were observed to have sufficient tension capacity.

All pile caps were observed to have sufficient shear capacity. The pile cap under the Core C shear wall was observed to have the largest shear demands approaching 70% of the capacity (Table 3).

Concrete shear walls

Case 1: 100% Wind Load

Compression, tension, and shear demands were compared with their corresponding capacities for shear walls. The ultimate design compression strength was calculated to be approximately 2000 pounds per square inch of cross section of wall. Based on LRFD load combinations with reduced live loads, no walls were observed to have peak compression demands exceeding 1700 psi (Figure 23) indicating sufficient compression capacity.

One corner of the Core C shear wall showed tensile stresses approaching 125 kip per foot of wall which exceeded their tensile capacity (Figure 24). Because there is additional vertical reinforcement distributed along wall faces (consisting of #8 at 12 inches on center) that will also resist tensile demands, the ultimate design tensile strength may be increased to approximately 600 kip over a distance of 5 feet, compared with tensile demands approaching 525 kip over the same distance, indicating that the wall has sufficient tensile capacity.

The ultimate design shear strength was calculated to be approximately 40 kip per foot of wall based on contributions of concrete and horizontal steel distributed along faces of walls. Based on LRFD load combinations with reduced live loads, no walls were observed to have peak shear demands exceeding 25 kip per foot of wall (Figure 25) indicating walls have sufficient shear capacity.

Case 2: 50% Wind Load

No walls were observed to have peak compression demands exceeding 1000 psi (Figure 26) indicating sufficient compression capacity. The peak tensile demand at one corner of Core C shear wall showed tensile stresses approaching 50 kip per foot of wall indicating that shear walls had sufficient tensile capacities (Figure 27). No walls were observed to have peak shear demands exceeding 12 kip per foot of wall (Figure 28) indicating walls have sufficient shear capacity.

Table 2: Demand-to-capacity ratios for shear in pile caps for Case 1 (100% Wind Load)

Pile cap foundation	Number of piles	Total shear capacity of pile cap (kip)	Total shear demand on pile cap (kip)	DCR
A	18	432	389	0.90
B	18	432	401	0.93
C	22	528	623	1.18
D	16	384	335	0.87
E	34	816	308	0.38
F	18	432	236	0.55
G	18	432	245	0.57
H	36	864	450	0.52
I	18	432	259	0.60

Note: Total shear demand was estimated considering all ASD load combinations ***with reduced live loads***

Table 3: Demand-to-capacity ratios for shear in pile caps for Case 2 (50% Wind Load)

Pile cap foundation	Number of piles	Total shear capacity of pile cap (kip)	Total shear demand on pile cap (kip)	DCR
A	18	432	224	0.52
B	18	432	217	0.50
C	22	528	332	0.63
D	16	384	197	0.51
E	34	816	166	0.20
F	18	432	131	0.30
G	18	432	126	0.29
H	36	864	229	0.27
I	18	432	139	0.32

Note: Total shear demand was estimated considering all ASD load combinations ***with reduced live loads***

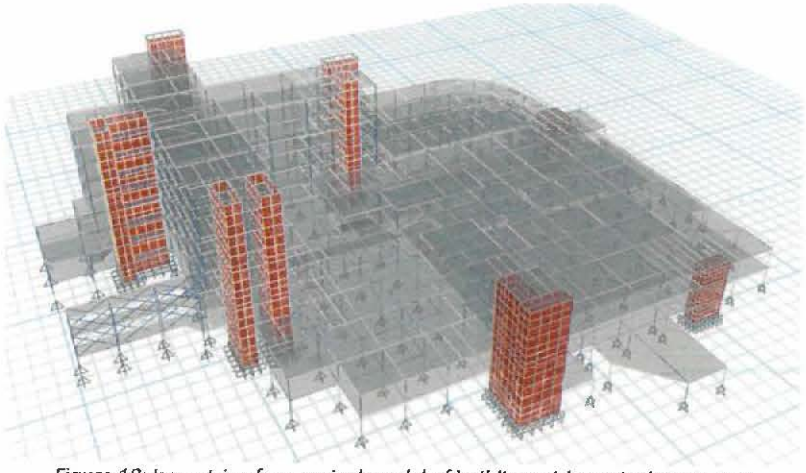


Figure 18: Isometric of numerical model of building with vertical expansion

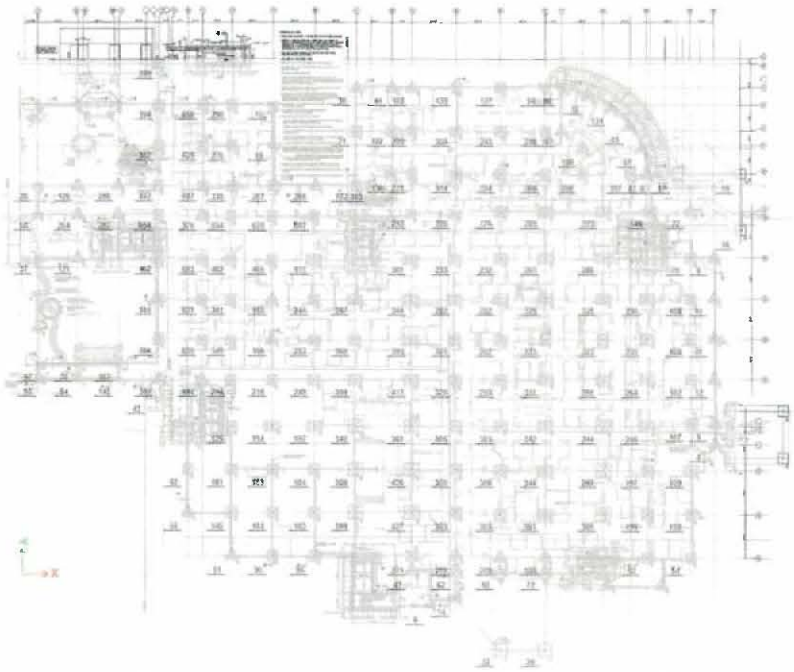


Figure 19: Peak compression loads at bases of steel columns for Cases 1 and 2

Note: Reactions were estimated considering all ASD load combinations ***with reduced live loads***

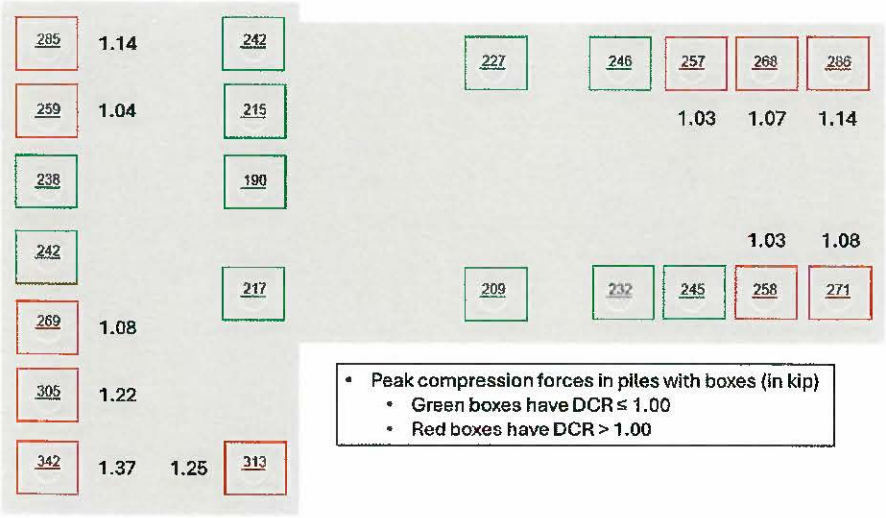


Figure 20: Peak compression demands for piles under Core C pile cap foundation for Case 1 (100% Wind Load)

Note: Reactions were estimated considering all ASD load combinations ***with reduced live loads***

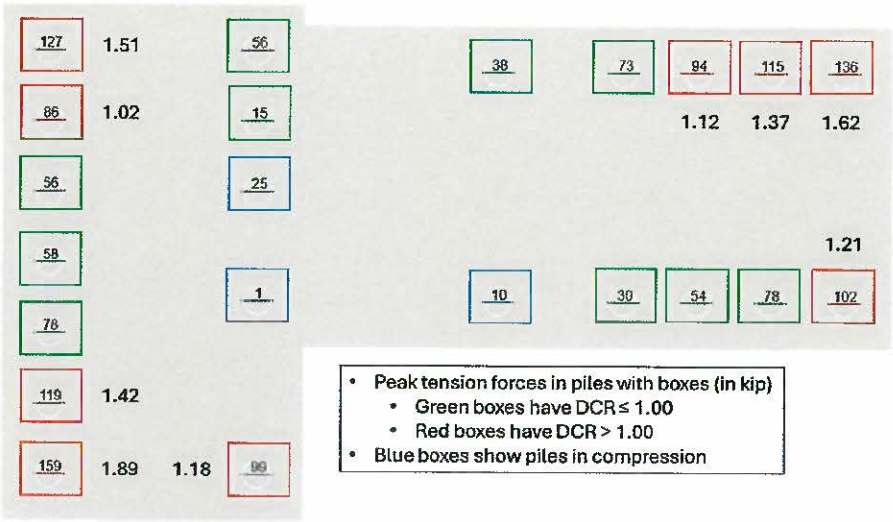


Figure 21: Peak tension demands for piles under Core C pile cap foundation for Case 1 (100% Wind Load)

Note: Reactions were estimated considering all ASD load combinations ***with reduced live loads***



Figure 22: Peak compression demands at piles under Core C pile cap foundation for Case 2 (50% Wind Load)

Note: Reactions were estimated considering all ASD load combinations ***with reduced live loads***



Figure 23: Peak compressive stress at corner of Core C shear wall for Case 1

Note: Reactions were estimated considering all LRFD load cases ***with reduced live loads.*** Region with red box shows peak compressive demand equal to approximately 1700 psi.

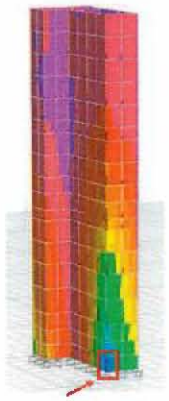


Figure 24: Peak tensile demands at corner of Core C shear wall for Case 1

Note: Reactions were estimated considering all LRFD load cases ***with reduced live loads.*** Region with red box shows peak tensile demand equal to approximately 125 kip per foot of wall.

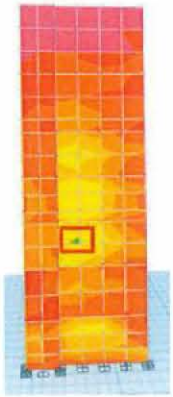


Figure 25: Peak shear demands for Core C shear wall for Case 1

Note: Reactions were estimated considering all LRFD load cases ***with reduced live loads.*** Region with red box shows peak shear demand equal to approximately 25 kip per foot of wall.

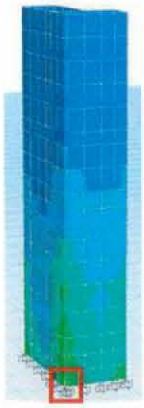


Figure 26: Peak compressive stress at corner of Core C shear wall for Case 2

Note: Reactions were estimated considering all LRFD load cases ***with reduced live loads.*** Region with red box shows peak compressive demand equal to approximately 1000 psi.



Figure 27: Peak tensile demands at corner of Core C shear wall for Case 2

Note: Reactions were estimated considering all LRFD load cases ***with reduced live loads.*** Region with blue box shows peak tensile demand equal to approximately 50 kip per foot of wall.

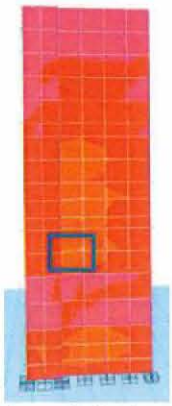


Figure 28: Peak shear demands for Core C shear wall for Case 2

Note: Reactions were estimated considering all LRFD load cases ***with reduced live loads.*** Region with blue box shows peak shear demand equal to approximately 12 kip per foot of wall.

Summary of Main Findings

Existing Building

Steel columns

1. Piles supporting steel columns have sufficient compression capacities to resist gravity demands.

Concrete shear walls

1. All pile foundations below shear walls have sufficient compression and tension capacities.
2. Shear walls have sufficient compression, tension, and shear capacities to resist the corresponding demands.

Vertical Expansion

Case 1: 100% Wind Load

Steel columns

1. Piles supporting steel columns have sufficient compression capacities to resist gravity demands.

Concrete shear walls

1. 11 piles have insufficient compression capacity, and the largest compression demand is 37% larger than the compression capacity of the piles.
2. 9 piles have insufficient tensile capacity, and the largest tension demand is approximately twice as large as the tension capacity of the piles.
3. Shear walls have sufficient compression, tension, and shear capacities to resist the corresponding demands.

Case 2: 50% Wind Load

Steel columns

1. Piles supporting steel columns have sufficient compression capacities to resist gravity demands.

Concrete shear walls

1. 1 pile has insufficient compression capacity and is within 1% of the compression capacity highlighting the advantage of performing a wind tunnel test to reduce wind loads based on local topographic conditions surrounding the building.
2. All piles have sufficient tensile capacities.
3. Shear walls have sufficient compression, tension, and shear capacities to resist the corresponding demands.

Conclusions

Existing condition

No strengthening is required for the building in its current as-built condition and use.

Building with Vertical Expansion

The feasibility study has shown that vertical expansion is possible. Limited strengthening of the foundation may be required. The extent of these modifications will be determined based on the results of the wind tunnel study. This feasibility study is not exhaustive and will require a complete design check of all structural members to confirm the assumptions of this study.

Conceptual Design and Recommendations

Because demands on some piles supporting the Core C shear wall have the potential to exceed the their capacities with the addition of additional floors, a general conceptual design would involve strengthening in this region. To reduce disruption to building operations during construction, it may be advantageous to utilize the courtyard located plan south of the core walls (Figure 29). The areas behind the elevator pits are emergency electrical rooms, and depending on the amount of existing equipment in these spaces, their use may also reduce disruption during construction (Figure 30).

A general strengthening design would include adding deep piles to the south of Core C and cast a new pile cap extending to the existing cap. Two new shear walls that connect corners of the walls enclosing stairs with the walls enclosing elevators may be required to help distribute the forces(Figure 31). The purpose of the new shear walls is to reduce peak stresses at the corners of the existing walls, in addition to increasing the overall stiffness and strength of the core. The corners of the new walls may also be tied into the pile system supporting the steel column and the column itself to provide further strengthening. New core walls may need only to span along the height of the first story depending on required wind loads.

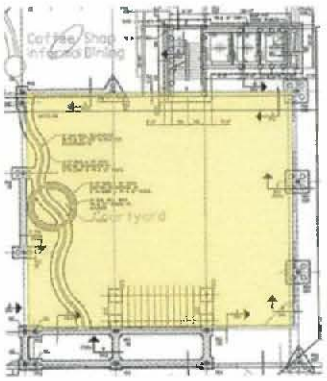


Figure 29: Courtyard located plan south of the Core C shear wall

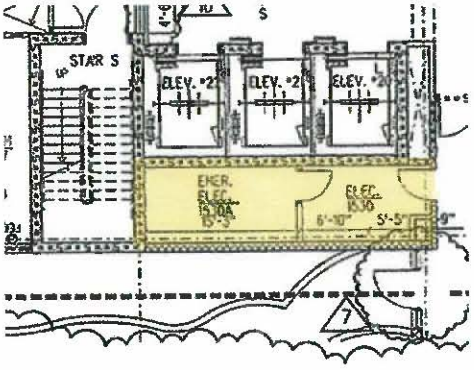


Figure 30: Emergency electrical rooms located behind elevator pits within Core C shear wall

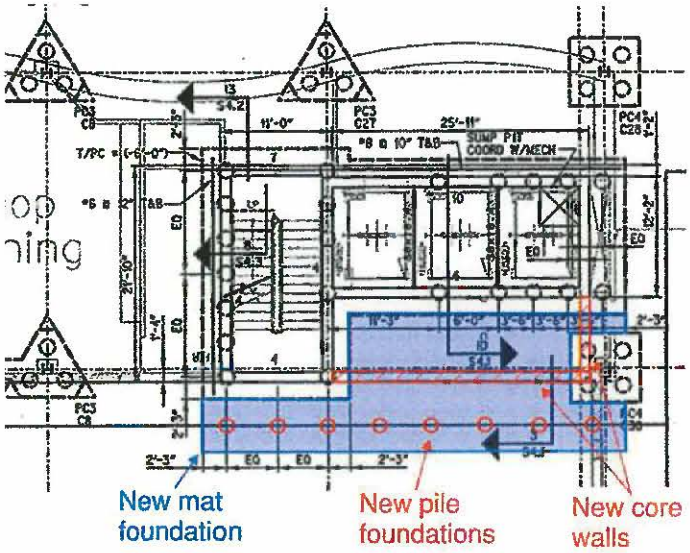


Figure 31: Conceptual design for strengthening of the Core C shear wall

As part of the design phase, we recommend a geotechnical engineer be consulted to provide geotechnical guidance related to settlement, shear capacities of the existing piles, and possible deep foundation stiffening if necessary.

Limitations

This report has been prepared for Edge Construction and the Owner to summarize our findings and opinions related to verifying foundation capacities and performing a feasibility study for vertical expansion. Please note that if additional discovery information is obtained, it is possible that we may form additional opinions or alter opinions expressed in this report.

Walter P Moore has no direct knowledge of, and offers no warranty regarding the condition of concealed construction or subsurface conditions beyond what was revealed in our review. Any comments regarding concealed construction or subsurface conditions are our professional opinion, based on engineering experience and judgment, and derived in accordance with current standard of care and professional practice.

We have made every effort to accurately and completely present all the pertinent results of this study. If there are perceived omissions or misstatements in this report regarding any aspect of the conditions associated with this study, we ask that they be brought to our attention as soon as possible so that we have the opportunity to fully address them in a timely manner.