Fort Lauderdale Police Headquarters

WJE Peer Review, Phase 1 - North Elevation Evaluation

1300 West Broward Boulevard
Fort Lauderdale, Florida 33312

December 31, 2024
WJE No. 2024.4855.0

PREPARED FOR:
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Fort Lauderdale, Florida 33312

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Letter: WJE Peer Review – Interim Findings Regarding Foundations

File Named *Pictures.pdf* containing Photographs of roof rectifications

Penhall Technologies Structural Scanning Report

INTRODUCTION

At the request of the City of Fort Lauderdale (City), Wiss, Janney, Elstner Associates, Inc. (WJE) has completed our Phase 1 – North Elevation Evaluation for the new Fort Lauderdale Police Headquarters building, located at 1300 West Broward Boulevard, Fort Lauderdale, Florida, 33312. This report summarizes the scope and findings of our review and contains project background, document review, and observations from our site visit. It also summarizes the findings from our structural design review of selected portions of the existing structure, reviews the rectification measures already implemented and planned to be implemented, discusses our findings, and provides additional comments and recommendations for consideration.

SUMMARY OF FINDINGS

The findings of our structural design review of the structural framing at and north of gridline 2 are as follows. More extensive information, discussion, and as-appropriate, conceptual rectification measures are found within the body of the report.

Roof Beams

- We concur with the findings by others that the roof cantilever beams were not code-compliant in flexural strength as-designed.
- The cantilever beams were not code-compliant for flexural strength even after the column enlargement rectifications. The cantilever beams are code-compliant for flexural strength in their current configuration with the added upturned beams.
- The cantilever beams gridlines on H and J are not code-compliant for shear and combined shear and torsion strength. We note that no rectifications to increase the shear strength of the beams have yet been proposed or implemented. The cantilever beams on gridlines G and K are code-compliant for shear and combined shear and torsion strength.
- We found that the spandrel beams north of gridline 2 have code-compliant strength.

Third Floor Beams

- We found that the third floor cantilever beams and spandrel beams north of gridline 2 have code-compliant strength.

Beam Deflections

- A discussion of the beam deflections and survey elevation data is contained in the report, but in short, the immediate and long-term relative deflections between the third floor beams and roof beams need to be carefully evaluated by Thornton Tomasetti prior to the final design of the curtain wall at the north elevation of the third story.

Third Story Columns

- We determined that there are design issues at all four third story columns at gridline 2 and the rectifications implemented to date have not fully resolved the design errors.
- These columns are not code-compliant in shear strength.

- These columns are not code-compliant in flexure at some locations or under certain analysis assumptions.

First Story Columns

- We concur with the findings by others that rectification measures are necessary to provide code-compliant strength and we find the proposed column jackets are a reasonable rectification measure.
- We recommend that the ties of the column jacket proposed by the SEOR be closed ties as discussed herein and we recommend that the surface of the existing column be roughened prior to the installation of the column jacket.

Foundations

- We concur with the findings by others that additional rectification measures beyond the enlarged footings are necessary and we find that the proposed micropiles and pile caps are a reasonable rectification measure.
- Comments on the micropile and pile cap design were provided in letter dated November 6, 2024. In addition to our comments in that letter, we recommend that additional dowels be added between the existing footing and the new pile cap.

PROJECT BACKGROUND

Description of the Structure

The Fort Lauderdale Police Headquarters is a 191,000-square-foot, three-story structure currently under construction. The building will include workspace for over 700 personnel, including training rooms, public meeting areas, and a community space. The design of the building was led by AECOM (the prime consultant), with Thornton Tomasetti (TT) as the Structural Engineer-of-Record (SEOR). The general contractor for the Project is Moss and Associates (Moss). Collectively, these entities are referred to as the Project Team. TT also served as the threshold inspector for the building.

The main portion of the building is rectangular in plan, with the length of the building oriented on a north-south axis. The north elevation of the building steps northward at the third floor and roof, forming a cantilevered projection at these floors. An isometric view of the building is shown in Figure 1. The facade of the building is clad with a combination of precast concrete fascia and full-height glass fenestration systems.

The gravity framing of the structure consists of one-way reinforced concrete slabs spanning between east-west oriented precast joists. These joists span to and are supported by north-south oriented reinforced concrete soffit beams, which bring the load to the cast-in-place concrete columns.

The lateral structural system for the building is comprised of cast-in-place concrete shear walls and/or concrete moment frames.

The foundations for the columns are spread footings bearing on soil in which vibro-compaction was used to improve the bearing strength.

Understanding of Issues to Date

WJE understands that after the precast concrete fascia panels were installed at the cantilevered roof overhang on the north side of the building, cracking and excessive deflection of the concrete slab and beams were reported. These deflections were significant enough that the glazing units spanning between the third floor and the roof no longer fit in the allotted space between the floors.

We also understand that the SEOR has acknowledged to the City that some of the roof structural design members at the cantilever at the north elevation of the building were under-designed and did not fully account for the weight of the precast concrete fascia panels nor the facade panels. After realizing this issue, the SEOR developed rectification measures that enlarged the cross-section and added a diagonal strut (corbel) to the two interior third story columns on the north elevation, reportedly reducing the span of the cantilever beam by 3 feet. The SEOR also developed an additional rectification to further strengthen and address the incremental long term dead load deflection and immediate live load deflection of the roof beams at the cantilever on the north elevation. This additional rectification, which was applied to four beams, added to the height of the beams by creating "upturned" sections that project above the roof slab. We understand that these measures have already been implemented. We understand that the roof beams were not jacked back to an undeflected position prior to implementing the rectifications. As such, while the rectifications serve to reduce long term deflections of the cantilever beams, they do little-to-nothing to reduce the deflections that were already locked in place prior to their implementation.

The footings for the interior columns at the north elevation—the same columns that were enlarged at the third story—were also enlarged. However, it was later decided that additional modifications to the first story columns at the north elevation (all four columns) and the foundations for these columns were necessary to address design deficiencies.

The SEOR then developed rectifications for the first-story columns and column foundations on the north elevation of the building to provide additional load-carrying capacity, but installation of these measures has not been completed. The first-story columns are to be enlarged to the west, south, and east, effectively forming a "C"-shaped column jacket. Micropiles and pile caps are being added to support the columns instead of the existing spread footings. We understand that the City and Project Team are moving forward to implement the micropile/pile cap solution and jacket the first story columns, but this work has not been completed to date. WJE reviewed the design of the micropiles and pile cap and previously issued a letter (dated November 6, 2024) with comments for consideration by the SEOR and pile contractor (Keller). A copy of that letter is attached to this report.

No rectification measures have been proposed by the SEOR for the second-floor columns or beams, nor for any other structural elements beyond the north cantilever.

DOCUMENT REVIEW

To understand the design and construction of the north elevation of the building we reviewed pertinent portions of the following documents:

- Structural Design Drawings
  - Revisions through November 15, 2023
  - 68 sheets

- Structural Design Drawings
  - Revisions through May 24, 2024
  - 70 sheets
- Structural Design North Cantilever Supporting Calculations
  - By Thornton Tomasetti
  - File name is *2024-04-25_CALC_HQ Roof Cantilever Field Fix at North Elevation.pdf*
  - Dated April 25, 2024
  - 175 sheets
- Precast joist and soffit beam shop drawings and calculations for part of roof
  - By Tekton Construction Group (Tekton)
  - Moss Submittal #326.1
  - File name is *2023.12.11_034100_326.1_ROOF AB PRECAST JOIST SOFFIT BEAMS CALCS_TTREVIEWED.pdf*
  - Issue Date November 27, 2023
  - 134 sheets
- Precast joist and soffit beam shoring shop drawings and calculations for roof
  - By Tekton
  - Moss Submittal #322.0 – 033000.10
  - File name is *2023.10.19_033000.10-322.0_TEMP-SHORING-FOR-ROOF-LEVEL-(ABCD)-CALCS_TTREVIEWED.pdf*
  - Issue Date October 7, 2023
  - 36 sheets
- Reinforcement shop drawings and calculations for roof
  - By Tekton
  - Moss Submittal #264.0 – 032000.10
  - File name is *2023.10.26_032000.10-264.0_HQ-ROOF-LEVEL-SLAB-BEAM-CMU_TTREVIEWED.pdf*
  - Issue Date October 13, 2023
  - 9 sheets
- Reinforcement shop drawings and calculations for roof with upturned beams
  - By Tekton
  - Moss Submittal #267.0
  - File name is *2024.08.23_032000.10-267.0---ROOF-REBAR-SHOP-DRAWINGS-W-ADDED-BEAMS---SUB_TTREVIEWED.pdf*
  - Issue Date August 23, 2023
  - 9 sheets
- Precast joist and soffit beam shop drawings and calculations for part of third floor
  - By Tekton
  - Moss Submittal #322.0 - 034100
  - File name is *2023.09.08_034100-322.0_L3-AB-PRECAST-JOIST-SOFFIT-BEAMS-CACLS_TTREVIEWED.pdf*

    - Issue Date August 25, 2023
    - 211 sheets
- Precast joist and soffit beam shoring shop drawings and calculations for third floor
    - By Tekton
    - Moss Submittal #318.0 – 033000.10
    - File name is *2023.09.19_033000.10-319.0_Temp-Shoring-for-L3-(ABC-D)-Calcs_TTREVIEWED.pdf*
    - Issue Date September 5, 2023
    - 33 sheets
- Reinforcement shop drawings and calculations for third floor
    - By Tekton
    - Moss Submittal #260.0 – 032000.10
    - File name is *2023.08.25_032000.10-260.0_HQ-3rd-FL-Rebar_TTREVIEWED.pdf*
    - Issue Date August 11, 2023
    - 11 sheets
- Shoring shop drawings for cast-in-place concrete
    - By Tekton
    - Moss Submittal #311.0 – 033000.10
    - File name is *2023.08.31_033000.10-311.0_-Temp-Shoring-Re-shoring_TTREVIEWED.pdf*
    - Issue Date August 21, 2023
    - 78 sheets
- Shoring shop drawings for roof spandrel beams
    - By Tekton
    - Moss Submittal #031000.10-1.0
    - File name is *24.06.17---HQ-North-Area-Shoring-Shop-Drawings---Sub.pdf*
    - Issue Date June 10, 2024, sign and seal date is June 14, 2024
    - 6 sheets
- File named *Approved%20Set-2024-03-28-BLD-CMIS-24030669 Corrective Permit Rev.pdf*
    - 10 pages
    - Includes drawings for enlargement of column spread footings, enlargement of third story interior columns, notes regarding beam deflections, and RFI #435
- Threshold Inspection Reports by TT
    - 11 files
    - Reports 001 through 143
    - Reports 189 through 207
- Survey data from the following dates (all 2024)
    - 3/27, 3/28, 3/29, 4/1
        - We understand that this data was developed by the shell contractor
    - 4/5 through 7/30 (74 dates total)
        - Data developed by Keith and Associates, Inc. (Keith)

- Daily reports from Moss for the following dates (all 2024)
  - 8/26 through 8/31
  - 9/3 through 9/7
  - 9/9 through 9/13
  - 9/17, 10/11, 10/14, 10/15
- Other files and subcontractor daily reports from Lotspeich for the following dates (all 2024)
  - 8/29 through 8/31
  - 9/3 through 9/6
  - 10/11
  - File named *Lotspeich FTL Police Epoxy Injection Quantities.pdf*
- Concrete material test report from Nutting Engineers of Florida, Inc.
  - One file containing one sheet, file name is *CTR_24-17625-C01.pdf*
  - Location of concrete is indicated as roof upturned beams
  - 28 day compressive strength is greater than 11,000 pounds per square inch (psi)
- Folder name "*Pictures*"
  - Contains 5 files includes a photograph of the reinforcement and partial formwork for the enlargement of a third story column
- RFI #480
  - Related to cracks at roof level
- RFI #537
  - Related to exposed reinforcement at roof level
- Architectural Precast Shop Drawings
  - Submittal 343, Revision 1
- Folder named "*Approved Submittals*" related to rectifications at north elevation
  - Contains 6 files
    - *24.08.12 - SIKADUR-35 HI-MOD LV - APP.pdf*
    - *24.08.14 - MAPEI_PLANITOP_XS - APP.pdf*
    - *24.08.14 - SIKADUR 31 HI-MOD GEL - APP.pdf*
    - *24.08.21 - HIT-RE 500 V3 INJECTION MORTAR - APP.pdf*
    - *24.08.26 - Roof Upturn Beam Concrete Mix Design - APP.pdf*
    - *24.08.30 - Roof Rebar SDs w Added Beams - AAN.pdf*
- File named "*Rebar Installation Sequence Color Sheet.pdf*"
- File named "*TTE Acceptance Letter 20241014 CRack Repair Ltr_ss.pdf*"
  - Letter from Thornton Tomasetti regarding inspection of epoxy crack injection
  - Contains 1 page
- File named "*Pictures.pdf*"
  - Contains 51 pages of photographs
  - Photographs are of north elevation rectifications at roof slab and third story columns
  - Some photographs of roofing installation after rectifications are completed are included in file

- Report from Lakdas/Yohalem Engineering, Inc. (Lakdas) named "*Evaluation Report of Cracked Beams and Slabs Off Gridline 2 at Roof & Floor Slab City of Ft. Lauderdale Police Headquarters*"
  - Contains 51 pages
  - Dated May 31, 2024
- Report from Penhall Technologies named "*Ft. Lauderdale Police HQ Structural Reinforcement Field Report for Lakdas/Yohalem Engineering*"
  - Contains 18 pages
  - Dated 5/22/2024
- Report from ScanTek GPR named "*FTl Police Station*"
  - Contains 11 pages
  - Dated 5/3/2024
- Files related to building foundations
  - Submittals 4, 345, 346, 347, 348
  - File named "*23.07.31 -Vibro Compaction Shop Drawings & Calcs. - Rev.1 SUBMITTED.pdf*"
  - "*Report of Limited Geotechnical Exploration*" by Nutting Engineers of Florida Inc, dated January 2021
    - 50 pages
  - Drawing Sheet HQ-S6-1-04, Revision 2, dated September 12, 2024
  - Drawing Sheet HQ-S6-1-05, Revision 1, dated September 12, 2024
  - File named "*2024-08-30_CALC_GRID 2 Foundations.pdf*" containing foundation calculations
- Filed named "*Police HQ Views*"
  - 9 pages schematically showing affected area and rectifications at north elevation

Structural Framing

The structural framing plan for the portion of the roof that is the primary focus of this report is shown in Figure 2 and Figure 3. The analogous portion of the structural framing plan for the third floor is shown in Figure 4 and Figure 5. The area of affected framing is generally at and north of gridline 2. Figure 6 shows a schematic section of the building north of gridline 3 at gridline H indicating in red the area of actual and proposed rectification measures to the structural framing. These measures include:

- Enlargement of the third story column (applies to columns at gridlines H and J)
- Addition of upturned beam at the roof (applies to gridlines G, H, J, and K)
- Jacketing of the first story column (applies to gridlines G, H, J, and K)
- Addition of new pile cap and micropiles (applies to gridlines G, H, J, and K)

Beams sizes, reinforcement, and other pertinent information about the original design of the roof and third floor framing north of grid line 3 are shown in Figure 7 through Figure 9 (structural design drawings) and Figure 10 (precast soffit beam shop drawings). See Figure 2/Figure 3 and Figure 4/Figure 5 for beam designations at roof and third floor, respectively. The roof and the third floor beams in question include the following structural drawing beam designations:

- Third Floor: 3SB-87, 3SB-90, 3SB-93, 3SB-96, 3B-101, 3B-102, 3B-103
- Roof: RSB-73, RSB-76, RSB-79, RSB-82, RB-97, RB-98, RB-99

Column sizes and reinforcement for columns at and north of gridline 3 are shown in Figure 11. At gridline 2, the first and third story columns have a 16 inch by 16 inch square cross-section and are reinforced with four No. 10 vertical bars. The second story columns are circular in cross-section, have a diameter of 24 inches, and are reinforced with eight No. 8 vertical bars.

The general notes (Sheet HQ-S0-1-02) specify a design 28-day concrete compressive strength of 4,000 pounds per square inch (psi) for column footings, 5,000 psi for formed slabs and beams, and 5,000 psi for columns. The steel reinforcement is noted on the same sheet as ASTM A615, Grade 60 for deformed bars, ASTM A706 for weldable deformed bars, and ASTM A1064 for welded wire reinforcement.

Contained as an appendix within the file "*2024-04-25_CALC_HQ Roof Cantilever Field Fix at North Elevation.pdf*" are concrete cylinder strength test reports from construction by Nutting. These reports indicate that the average measured 28-day compressive strength for the columns at gridline 2 were as shown in Table 1.

Table 1. Average 28-day compressive strength for concrete placed in columns at gridline 2

Gridlines	Story	Average 28-day compressive strength (psi)
G-2	1	5,850
L-2	1	6,520
H-2, J-2	1	7,140
J-2, H-2, G-2	2	6,020
G-2, H-2, J-2, K-2	3	5,530

**The column strengths for the third story columns are taken from the test report that identifies the general location as "ROOF DECK POUR 1" as it is presumed that the columns and slab were placed at the same time.*

The rectification measure designed by the SEOR for third story columns is shown in Figure 12 and Figure 13. It consists of a supplemental structural element with a corbel-like section sistered onto and integrated with the existing column, functionally enlarging it. The new element is reinforced with four No. 9 vertical reinforcing bars and No. 4 closed ties spaced at 10 inches on center. The "corbel" is reinforced with four inclined No. 9 longitudinal bars enclosed with No. 4 ties at 6 inches on center. The ends of the longitudinal bars are not hooked and do not extend into the existing column. Two bent No. 7 bars are present at the top of the new "corbel". These No. 7 bars do not extend into the existing column. The supplemental element is integrated with the existing column using No. 5 "U"-bars placed at 12 inches on center. The legs of these "U"-bars are embedded 8 inches into the existing column and anchored using epoxy adhesive. Later, after the supplemental element was constructed, approximately the bottom 2 inches of the new column were cut at the interface between the column and the existing third floor. We understand through verbal correspondence with various parties that this modification was made at the suggestion of Lakdas and that the SEOR accepted this change. We do not, however, have written documentation of how this modification came to pass.

At some point after the column enlargement was installed, upturned reinforced concrete projections above the existing cantilevered soffit beams were added above the roof deck. We understand that these upturned beam projections were added to strengthen the cantilever beams and reduce the expected long-term deflection of the cantilever roof beams. The reinforcement for these beams is shown in reinforcing shop drawings contained in Submittal 267.0. The upturned beam design is also contained within the document "*Rebar Installation Sequence Color Sheet.pdf*" and is shown in Figure 14.

The upturned beam projections extend from the tip of the cantilever to 14 feet to the south of gridline 2. The height of each is 12 inches. The width of the beams is shown in Submittal 267.0, Sheet HQ-R57 as 24 inches. Two layers of longitudinal bars are shown in Figure 14 and the number of bars and bar size is noted on Sheet HQ-R57 as eight No. 9 bars, with four bars in each layer. Inverted No 4 bar "U"-stirrups are doweled into the top of the roof beam. The stirrup spacing is shown as 4 inches on center north of gridline 2 up to the transverse beam, 6 inches on center for 6 feet 6 inches south of gridline 2, and 12 inches on center from 6 feet 6 inches to 14 feet south of gridline 2. Two No. 5 "L"-bars doweled into the existing beam are provided at each end of the beam. Concrete material cylinders tests (from *CTR_24-17625-C01.pdf*) indicate that the 28-day compressive strength for these supplemental measures is greater than 11,000 psi.

The design of the column jackets in the first story at gridline 2, the added micropile, and the associated pile cap is found on Sheets HQ-S6-1-04 and HQ-S6-1-05, both dated September 12, 2024. The installation sequence for these elements was set forth diagrammatically in six steps on Sheet HQ-S6-1-04, as shown in Figure 15 (steps 1 through 3) and Figure 16 (steps 4 through 6). The details of the supplemental elements, including structural member sizes and reinforcement, are found on Sheet HQ-S6-1-05 and are shown in Figure 17 and Figure 18.

The column jacket is a 6 inch thick three-sided element that builds out the east, south, and west faces of the existing columns. The column jacket is reinforced with ten No. 9 vertical bars and has No. 4 ties at 12 inches on center. The ties for most of the column height are not continuous ties, but are made up of three components, which are two truncated "U"-shaped bars and one full "U"-shaped bar. One leg on the truncated "U"-shaped bars and two legs on the full "U"-shaped bar are doweled into the existing column with epoxy adhesive. The top two column ties have a different configuration (see lower detail in Figure 18).

The new pile caps are shown in plan in Figure 19 and Figure 20. Pile cap reinforcing information is shown in Figure 20 through Figure 24. The micropiles, four per column, are shown as being designed by a delegated design engineer. Micropiles are spaced at 4 feet from the center of the existing column in the east-west direction and 2 feet on center from the existing column in the north-south direction. The piles bypass the original footing and the new pile cap is largely positioned on top of the existing footing, extending over and engaging with the new micropiles. Twelve No. 7 and six No. 6 inverted "U"-shaped dowels are embedded using epoxy adhesive into the top of the existing footing to tie the new pile cap and existing concrete footings together (Figure 19 and Figure 20). Twenty-four No. 6 dowels are embedded into the sides of the existing column at the base (Figure 20, Figure 23, and Figure 24). Longitudinal reinforcement spans between the east and west micropiles on the north and south sides of the column. This reinforcement consists (per side) of nine No. 8 straight bars on top, nine No. 8 straight bars on bottom, and nine No. 8 bars with hooks at each end at the bottom. The bottom reinforcement is to be in two layers. No. 5 stirrups with four legs are placed at 4 inches on center (Figure 22). As of the date of this report, the micropiles and pile cap have not been installed.

Timeline of Issues and Rectifications

The reviewed documentation indicates the following timeline for notable events to date related to the roof framing and remedial work:

- Roof beams and roof slab were completed on November 13, 2023
  - Threshold Inspection Report No. 057
- Precast panels erected in early March 2024
  - Threshold Inspection Report No. 125
    - Date on report says November 27, 2023, but report is in sequence and date on prior No. 124 is March 6, 2024, and date on subsequent No. 126 is March 12, 2024
    - Page 6 of SEOR calculations for north cantilever (dated April 25, 2024) notes panel erection date as March 11, 2024
  - "Slab deflected by approximately ½" to ¾" according to the contractor"
    - The location of the measured deflection and how the deflection was measured is not noted. We are assuming that the measurement is indicative of the true deflection at the tip of the cantilever beams and was relative to the top of the columns at gridline 2.
  - Cracks observed in roof slab and in Beam RSB-79
- Threshold Inspection Report No. 126
  - Dated March 12, 2024
  - Cracks observed "on the north of gridline 2 on the Roof Level Area A. Contractor stated that the slab has deflected ½" to ¾" on the area above and to the east of gridline J."
- First survey of framing elevations was performed on March 27, 2024
  - We do not know the company name for the initial surveys
- Third story column enlargement placed on April 2, 2024
  - Threshold Inspection Report No. 140
  - Threshold Inspection Report No. 141 (April 3, 2024) shows column enlargement after forms were stripped
- Footing enlargement placed on April 4, 2024
  - Threshold Inspection Report No. 142
- First survey by Keith on April 5, 2024
- First survey by Keith with columns on May 3, 2024
- Shoring for the roof spandrel beams was put in place
  - We do not know the exact date, but shoring shop drawings have sign and seal date of June 14, 2024
- Last survey by Keith on July 30, 2024
- Roof slab cracks injected with epoxy on September 3, 2024, and September 4, 2024
  - Threshold Inspection Report No. 202 and No. 203
- Roof upturned beams placed on September 9, 2024
  - Threshold Inspection Report No. 207

- Shoring of spandrel beams removed
  - Date of removal is unknown but was prior to October 18, 2024

Documentation of Observed Cracking and Rectifications

In the set of files we received, documentation of the cracking observed at the roof was primarily recorded in the threshold inspection reports and in a file named "*Pictures.pdf.*" As noted in the timeline above, cracks in the roof slab were observed by the contractor immediately following the installation of the precast facade panels at the roof level. These observations are noted in Thornton Tomasetti's Threshold Inspection Report No. 125 along with photographs (Figure 25 through Figure 28).

Additional documentation and observations were summarized in Threshold Inspection Report No. 127, dated March 13, 2024. Water was sprayed onto the top of the roof slab (Figure 29 and Figure 30) and it was observed that some water infiltrated through the cracks in the slab and was evident on the underside of the slab (Figure 31 through Figure 34), indicating that the cracks extended through the entire thickness of the slab.

Remedial work to columns H-2 and J-2 (column enlargement) was first documented in Threshold Inspection Report No. 137, dated March 28, 2024. In this report, photographs are provided (e.g., Figure 35) showing holes drilled into the north face of the column to receive epoxy adhesive installed dowels as part of the rectification of the column and roof structure. Documentation of that work continues in Threshold Inspection Report No. 138 (dated March 29, 2024) with installation of the rebar dowels into the north face of columns H-2 and J-2 (Figure 36 through Figure 38) and in Threshold Inspection Report No. 139 (dated April 1, 2024) with installation of the vertical reinforcement and ties (Figure 39). Documentation of the beginning of footing enlargement at the base of columns H-2 and J-2 is also recorded in this report (Figure 40) and documentation of the work on the third story column enlargement (Figure 41 and Figure 42) and footing enlargement (Figure 43 and Figure 44) continued in Threshold Inspection Report No. 140, dated April 2, 2024. In report No. 140, it was noted that the form on the column enlargement was moved 2 inches to the north to provide adequate cover on the reinforcement and this change was approved by the SEOR. Photographs of the completed third story column enlargement (Figure 45 and Figure 46) were provided in Threshold Inspection Report No. 141 (dated April 3, 2024), and continued work on the footing enlargements was documented (Figure 47 and Figure 48). Threshold Inspection Report No. 142 (dated April 4, 2024) stated that the column footing enlargements were completed (Figure 49 and Figure 50).

Epoxy injection of cracks in the roof slab at the north end of the building and work on installing the upturned beam projections were first documented in Threshold Inspection Report No. 200, dated August 30, 2024. In this report it was noted that holes were drilled for the inverted "U"-shaped stirrups (Figure 51), roof cracks were cleaned, and injection ports were installed in preparation for epoxy-injection (Figure 52). This work continued in Threshold Inspection Report No. 201, dated August 31, 2024. Some stirrup installation (Figure 53) and epoxy injection (Figure 54) occurred on September 3. 2024 (Threshold Inspection Report No. 202). Documentation of this work continued in Threshold Inspection Report No. 203, dated September 4, 2024 (Figure 55 and Figure 56). Similar work continued in Threshold Inspection Report No. 204, dated September 5, 2024, and No. 205, dated September 6, 2024. Threshold Inspection

Report No. 206, dated September 9, 2024, documented completed reinforcement placement (Figure 57 and Figure 58) and noted that concrete was placed for the upturned beam projections.

Additional photographic documentation of the roof crack epoxy injection, upturned beam projections, and cutting of the base of the column enlargement is provided in the file named "*Pictures.pdf*." This file contains photographs dating from August 29, 2024, to October 15, 2024, and is attached to this report.

Nondestructive Evaluation Reports

Two nondestructive evaluation reports related to the roof slab and roof structural framing were provided to WJE. The companies that generated the reports used ground-penetrating radar (GPR) to scan structural members to identify reinforcing locations.

GPR is an electromagnetic nondestructive test method in which short-pulse radar waves are emitted from an antenna placed in contact with the surface of the concrete structural member. The behavior of those radar waves is governed by electromagnetic wave propagation theory. When a wave leaves the antenna, it is attenuated (i.e., weakened in strength) as it propagates into the concrete member. When the wave encounters an interface of differing electromagnetic properties (e.g., reinforcement), a portion of the energy from the wave is reflected to the antenna (in proportion to the difference in dielectric constant at the interface). These reflections can be used to identify and reconstruct reinforcement locations in concrete structural members. Bar sizes (diameter) are not determined with this nondestructive technique. Chipping to expose and measure the bar is necessary to determine the bar size, if desired.

WJE reviewed the report provided by ScanTek GPR dated May 3, 2024. The report was prepared by Zack Anderson. The report indicates that it is a summary of work carried out at the building from April 29 to May 2. The area of the work included both the roof and third floor. The report notes that the reinforcement was marked in the field using black to represent the top layer of reinforcement and blue to represent the bottom layer of reinforcement. Four "spots" were scanned. Spot 1 was noted as being at column G2, i.e., at gridlines G-2, or beams RSB-72 and RSB-72 (Figure 59). ScanTek GPR noted that they marked out all the rebar within the "first two inches of the concrete as the top layer." Note that ScanTek did not report how they calibrated a dielectric constant (and in term wave velocity) in order to accurately estimate cover depth. ScanTek GPR states that "Both sets of rebar are spaced roughly eight inches on center with one another." It appears that ScanTek GPR is referring to both the longitudinal and transverse steel in the beam with this statement.

Spot 2 was the top of the spandrel beam at the north end of the slab between gridlines G and H (RB-97), see Figure 60. The north-south bars in the beam, which would be the either the slab reinforcement or transverse reinforcement in the beams (stirrups), were noted as being spaced at roughly 12 inches on center, and the east-west bars, which would be the longitudinal steel in the beam, were noted as being spaced "roughly every six to eight inches"..."for both layers of rebar."

Spot 3 was noted as being at column H2, i.e., at gridlines H-2, or beams RSB-75 and RSB-76 (Figure 61). The reinforcement was noted as being "two to four inches into the slab", presumably meaning that ScanTek believed that the cover depth was two to four inches. The beam reinforcement was noted as generally being 8 inches on center. The north-south bars, which would be the longitudinal reinforcement, were noted as being spaced at approximately 6 inches on center, and the east-west bars, which would be the transverse reinforcement, were noted as being spaced approximately 12 inches on center.

Spot 4 is noted as being at a "Level 3 beam" but appears to be the precast facade panel at the roof, based on the photograph provided (Figure 62). The reinforcement was noted as being "approximately two to four inches in to the concrete", presumably meaning that ScanTek believed that the cover depth was two to four inches. The bar spacing was noted as approximately 8 inches on center, but it was not clear if this spacing referred to the vertical bars, horizontal bars, or both.

The second nondestructive evaluation report that was provided to us was developed by Penhall Technologies (Penhall) on behalf of Lakdas. The report is dated May 22, 2024, and indicates that the purpose of the work was "To locate number of rebars and their depths." The equipment used was noted as "Mini Xt with LineTrac." A dielectric constant of 7.68 was used for the GPR scans. No information was provided on how the dielectric constant was calibrated. Twelve locations were scanned including the third floor slab and roof slab at gridlines G-2, H-2, J-2, and K-2. The sides of the roof beams at gridlines G-2, H-2, J-2, and K-2 were also scanned. The results of the scans were marked on the structural members. A copy of the Penhall report is attached to this report. For many of the beams scanned, the top reinforcement was noted as being located around 2 ½ to 5 inches below the top of the slab. For reference, the precast shop drawings for the subject soffit beams (Sheet SC-3A, dated October 25, 2023) indicate either a clear cover of 1 ½ inches or a dimension to center of longitudinal bars of 1 ½ inches. If the clear cover is 1 ½ inches, then the distance to the top of the longitudinal bars would be 2 inches (1 ½ inch clear cover plus ½ inch diameter stirrup (No. 4 bar)).

Survey Data and Observed Deflections

It was noted by the Thornton Tomasetti (who is serving as the Threshold Inspector in addition to being the SEOR) in Threshold Inspection Report No. 125 that, after attaching the precast facade panel to the roof beams, there was 1/2" to 3/4" of immediate deflection at the end of the cantilever roof beams according to the contractor. The location of the measured deflection and how the deflection was measured are not noted. We are assuming that the deflection was measured at the tip of the cantilever beams and was relative to the top of the columns at gridline 2. After issues with the roof beams were observed, surveys of the elevation of the top of the roof beams were conducted to understand the current elevation of the beams and how this elevation changed over time.

WJE has data from four surveys in March and early April of 2024. These surveys are not stamped or signed, nor is the company that performed these surveys identified. However, we understand through communication with the other surveyor (Keith) that these initial surveys are believed to have been performed by the shell contractor. We understand that the reference elevation for the surveys was taken as the building elevation of 0 feet 0 inches, which corresponds to an unknown location on the top of the finished ground floor slab. We are not sure how a reference elevation of 0 feet 0 inches was established relative to any datum. Only the roof beams at the north elevation at the roof (RB-97, RB-98, and RB-99) were surveyed. The top of the columns at the roof at gridlines G-2, H-2, J-2 and K-2 were not surveyed.

Other surveys were performed by Keith starting on April 5 through July 30, 2024. A total of 74 surveys were performed by Keith. Initially the surveyed locations were the same or approximately the same as the previous surveyor; however, from May 3 onward, the elevation at the top of the columns at the roof at gridlines G-2, H-2, J-2 and K-2 was surveyed. The reference elevation for the surveys by Keith is the North

American Vertical Datum of 1988 (NAVD 88). The elevations are reported by Keith in feet to three decimal places. No statement on the accuracy of the survey is provided.

For evaluating the structure later in this report, we only use the surveys from Keith because to understand the relevance of the reported deflection to the structural design and performance of the roof framing, it is necessary to know the relative difference in elevation between the end of the cantilever roof beams and the columns. Table 2 below summarizes the survey data from Keith. The location of the survey points in plan is shown in Figure 63. The table does not include data for all days, but rather key days (approximately monthly from date of concrete placement or from date of installation of precast facade panels) between the start and finish days of data collection.

Table 2. Survey elevation data

	Date						
Survey Point	4/5/2024	4/12/2024	5/3/2024	5/13/2024	6/13/2024	7/13/2024	7/30/2024
A	55.966′	55.963′	55.955′	55.955′	55.951′	55.955′	55.959′
B	55.955′	55.948′	55.941′	55.938′	55.935′	55.942′	55.944′
C	55.907′	55.897′	55.890′	55.888′	55.887′	55.898′	55.898′
D	55.878′	55.870′	55.862′	55.859′	55.855′	55.867′	55.868′
E	55.864′	55.859′	55.847′	55.849′	55.844′	55.856′	55.858′
F	55.885′	55.880′	55.872′	55.870′	55.865′	55.875′	55.876′
G	55.860′	55.856′	55.846′	55.844′	55.838′	55.849′	55.849′
H	55.851′	55.848′	55.838′	55.837′	55.829′	55.840′	55.840′
I	55.861′	55.858′	55.847′	55.846′	55.837′	55.848′	55.849′
J	55.861′	55.857′	55.847′	55.846′	55.840′	55.849′	55.849′
K	55.865′	55.860′	55.852′	55.850′	55.842′	55.852′	55.852′
L	55.835′	55.830′	55.821′	55.819′	55.813′	55.823′	55.822′
M	55.843′	55.837′	55.828′	55.825′	55.818′	55.828′	55.829′
N	55.862′	55.855′	55.846′	55.843′	55.838′	55.846′	55.847′
O	55.910′	55.906′	55.899′	55.897′	55.892′	55.899′	55.899′
P	55.958′	55.956′	55.949′	55.949′	55.944′	55.948′	55.950′
A1	--	--	56.046′	56.043′	56.042′	56.042′	56.042′
F1	--	--	56.026′	56.024′	56.025′	56.025′	56.025′
K1	--	--	56.024′	56.025′	56.026′	56.026′	56.026′
P1	--	--	56.029′	56.030′	56.027′	56.027′	56.027′

Notes: See Figure 63 for location of survey points on roof plan. Elevation is in units of feet.

Table 3. Survey elevation data for top of columns

Gridline	Survey Point	Data Collected on May 3, 2024	Data Collected on July 30, 2024	Difference
G-2	A1	56.046′	56.042′	0.004′
H-2	F1	56.026′	56.025′	0.001′
J-2	K1	56.024′	56.026′	-0.002′
K-2	P1	56.029′	56.027′	0.002

Notes: See Figure 63 for location of survey points on roof plan. Elevation is in units of feet.

Relative to column top locations, the elevation difference between the first day when column elevation data was collected (May 3, 2024) and the final day of survey data collection (July 30, 2024) is summarized in Table 3 for survey points A1, F1, K1, and P1 which are located approximately at the top of the columns at gridlines G-2, H-2, J-2, and K-2, respectively. Possible sources that might contribute to a difference between these two dates include column shortening due to creep and shrinkage, settlement of the foundations and underlying soil, and survey accuracy. The difference in elevation between the initial survey date and the final survey date (79 days) is less than 1/16 inch for one column and less than 1/32 inch for the other three columns. These differences are likely within the accuracy limit for the equipment used by the surveyor.

For comparison, survey points A, F, K and P are located approximately at the end of the cantilever roof beams (beams RSB-73, RSB-76, RSB-79, and RSB-82, respectively), meaning that the elevation difference between these two sets of data can be interpreted to approximate the deflection at the tip of the cantilever beams. These calculations of elevation difference approximating the deflections are shown in Table 4.

Table 4. Calculations of beam elevation differences in inches based on survey data, by date

Beam|Date	April 5, 2024†	May 13, 2024	June 13, 2024	July 12, 2024	July 30, 2024
RSB-73*	1.0	1.1	1.1	1.0	1.0
RSB-76*	1.7	1.8	1.9	1.8	1.8
RSB-79*	1.9	2.1	2.2	2.1	2.1
RSB-82*	0.9	1.0	1.0	0.9	0.9
RB-97 to G-2**	1.7	1.9	1.9	1.7	1.7
RB-97 to H-2**	1.8	2.0	2.0	1.9	1.9
RB-98 to H-2**	2.1	2.2	2.4	2.2	2.2
RB-98 to J-2**	2.0	2.1	2.3	2.1	2.1
RB-99 to J-2**	2.2	2.4	2.5	2.4	2.4
RB-99 to K-2**	2.0	2.2	2.3	2.2	2.2

*Notes: *Cantilever tip elevation difference relative to top of column, **Mid-span elevation difference relative to column, †Uses column data from May 3, 2024, for calculations*

A mid-span (or approximately mid-span) deflection relative to the closest column can be approximated by taking the difference between the near mid-span point elevation and the nearest column elevation. For example, for RB-97 two values are reported in Table 4. The first value is the difference in elevation between points C and A1. The second value is the difference in elevation between points D and F1.

The data from Table 2 and Table 4 can also be used to approximate relative deflections between the spandrel beam midspan and the spandrel beam supports, if so desired. We do not use these relative deflections as part of our comparison between the structural analysis model results (see later sections of this report) and the survey data, so we are not reporting them here.

Evaluation Report by Lakdas/Yohalem Engineering, Inc.

Lakdas/Yohalem Engineering, Inc. (Lakdas) was engaged by the city of Fort Lauderdale to evaluate the design of the building in the area north of gridline 2. This evaluation included reviewing the original design, as-built conditions, the in-place remedial work, and providing preliminary designs for any additional remedial work that they recommended. Lakdas summarized their work and findings in a report dated May 31, 2024. At the time of Lakdas' evaluation, the column enlargements at the third story were in place at the columns at gridlines H-2 and J-2.

Lakdas evaluated the cantilever beams north of gridline 2 at the third floor and the roof for strength. They considered two design cases, one with the as-specified cover over the bars of 1 ½ inches (i.e., 1 ½ inches of clear cover plus ½ inch diameter stirrup (typical) for a total cover over the longitudinal bars of 2 inches), and one case with the cover over the top bars of 4 inches. The 4-inch cover assumption is based on the findings of the GPR scanning by Penhall. For the roof beams along gridlines H and J, they evaluated the strength of the beams at 3 feet north of the face of the existing column at H-2 and J-2 in light of the then-present column enlargement.

Lakdas also evaluated the corbel design at the top of the column H-2 and J-2 retrofits, the long-term deflection of the roof cantilever beams supported by these new corbels, and the foundations at the base of the H-2 and J-2 columns.

Lakdas indicated that they evaluated the beams for flexure (bending), shear, torsion, and deflection. They refer to three tables in their report, but we were not able to find those tables in our copy of the report.

Lakdas noted the following in regard to the cantilever beam designs before the column enlargement at H-2 and J-2 was completed:

- *"Original strength of the beam per the permitted documents (beam schedule) for the roof cantilever beams on grids H and J was determined to be provided only 12% of the required strength"*
- *"Original strength of the beam per the reinforcement shop drawing documents. The cantilever beams on grids H and J were determined to be approximately 48% of the required value."*
- *"The original strength of the beam per rebar placement at the site was determined as the average cover to the rebar was 4 inches. We found that the beam's strength is approximately 10% less than the design strength with a 1.5-inch cover"*

Lakdas then notes the following related to the cantilever beam designs after the column enlargement at H-2 and J-2 was completed:

- *"Two cantilever beams at H and J are 80% of the required value 3'- 0 away from the face of the column on grid 2 (considering the location of the rebar at 4" cover to the top rebar per onsite GPR survey)."*
- *"Considering the effect of cracked beam condition on the cantilever beam top rebar with a 4" cover to the rebars, the repaired beams were determined to have only 70% of the required strength."*
- *"Considering the effect of the soffit beam in the cantilever beam (with a 1" gap between and the soffit piece), the repaired cantilever beam was determined to provide 60% of the required strength. It is this office's considered opinion, supported by the attached letter from the soffit beam supplier PCI Engineers, that the four cantilever beams north of gridline two at gridlines G, H, J, and K are, on average, 35% less than the required strength. Therefore, it is crucial that further assessment and potential reinforcement are undertaken without delay."*

It is not clear from Lakdas' report if the strength deficiencies they report are flexural or shear deficiencies. Further, based on the claimed percent of deficiency it is unclear if the deficiencies noted are indeed strength deficiencies or are stiffness deficiencies. If the existing (or designed) condition truly had only 12 percent of the required strength, the structural member would likely have already failed. As the in-place members have not failed and Lakdas's report is vague, a more reasonable inference of their unclear report is that the beam only has 12 percent of the required stiffness.

Lakdas ultimately ended up recommending that the cantilever beams at the roof be strengthened using either carbon fiber strips or upturned beams. They also recommended that the cantilever beams at the third floor at gridlines H-2 and J-2 be strengthened. Lakdas recommended sealing all the cracks in the roof deck using epoxy and a penetrating sealer prior to strengthening structural members.

Lakdas evaluated the spandrel beams at the third floor and roof and noted that these beams have sufficient capacity for strength but are deficient for long-term deflection. Lakdas also noted that the spandrel beam deflection "could significantly affect the integrity of the [curtain wall] panels, posing a potential risk."

Lakdas reviewed the haunch (corbel) of the column enlargement and concluded that the haunch did not meet the design requirements of ACI 318 and recommended a carbon fiber wrap encompassing the new haunch and existing column.

Lakdas reviewed the footing enlargement for the columns at gridlines H-2 and J-2 and concluded that the rectification design proposed by the SEOR was not sufficient. Lakdas was skeptical of the increase in bearing capacity from 7,000 pounds per square foot (psf) to 9,000 psf claimed by the SEOR and geotechnical engineer (Nutting) and noted that increasing the bearing pressure could lead to significant differential settlement. Lakdas also noted that the footing enlargement proposed by the SEOR does not consider eccentric load on the footing. In this report, Lakdas proposed a footing with greater plan area and thickness.

FIELD OBSERVATIONS

Brent Chancellor, PhD, PE, of WJE visited the building on November 27, 2024, to observe the rectification measures completed to date and to generally observe existing conditions including readily visible distress. Observations were made from the top-of-slab elevation of each story. We did not observe or review the top of the roof slab as this area was already covered by roofing materials. Similar to the photographs of distress reviewed as part of the document review, we observed cracks on the underside of the roof slab that had been injected with epoxy (Figure 64 through Figure 67) as well as cracking of the cantilever beams that had not been injected (Figure 68 through Figure 71). The third story column rectifications described earlier in this report were observed to be in place (Figure 65 and Figure 66), and the bottom of the supplemental element had been cut approximately 2 inches above the finished floor and removed (Figure 72). The observations we made on site were consistent with the photographs reviewed during the document review.

STRUCTURAL DESIGN REVIEW

Original Design

Structural Analysis Model

To evaluate the original design of the north elevation of the building, we developed a structural analysis model in the software program, ETABS, by Computers and Structures, Inc. (CSI). ETABS is generally used for the design and analysis of buildings. Because the focus of this study was on the cantilever portion at the north edge of the building, the ETABS model that we constructed for the various analyses we performed was limited to the area roughly bound by gridlines 1 to 5, and E to L. Isometric and section views of the structural analysis model are shown in Figure 73 through Figure 75. The sizes of the structural members in the model were taken from the structural design drawings by the SEOR ("PERMIT SET", dated June 10, 2022) and included slabs, joists, beams, columns, and walls. We also included the precast concrete facade elements in the model with attachment points to the structure at the locations of the precast panel connections based on the Architectural Precast Shop Drawings included in Submittal #034500-353.1. Only three bays of the building were modeled as it is not necessary to model the full building to obtain the gravity response of structural elements (beams, columns, etc.) on the north elevation of the building. The columns were modeled as pinned at the base. We did not explicitly model the footings or pile caps. Effective cracked stiffnesses of slab and beam elements for the purpose of deflection estimations were in most cases automatically calculated in ETABS based on material properties specified in the structural drawings, as well as the demands computed by the program, but we also assessed the sensitivity of the model to a variety of assumptions regarding stiffness. To obtain member forces for strength calculations, column flexural stiffness modifiers were applied based on the guidelines in ACI 318-14[1]. A column flexural stiffness modifier of 0.7 was applied to the first and second story columns. A column flexural stiffness modifier of 0.5 was applied to the third story columns due to the relatively low axial force demands on the columns in this story. A torsional stiffness modifier of 0.35 was used as an approximation of the post-cracked torsional stiffness of the spandrel beams and cantilever soffit beams.

The analytical models incorporated 'nonlinear staged construction' techniques to emulate the construction timeline, as well as installation of the remedial measures including the enlargements of the Level 3 columns, the rooftop upturned beams, and the Level 1 column jackets. This analysis technique is intended to provide a better accounting of how forces are distributed throughout the structure compared to a more simplified analysis in which the structure is assumed to be fully constructed before any gravity loads

[1] ACI 318-14: Building Code Requirement for Structural Concrete (ACI 318-14)

are applied. In a staged-construction analysis, member properties and/or loads are "turned on" or inserted in a specific sequence or at a specific analysis stage that better emulates the incremental construction and incremental loading that a structure experiences in "real life."

Design Loads

The self-weights of reinforced concrete structural elements were automatically calculated within ETABS based on member size and concrete density. The superimposed dead loads and live loads (the latter of which was reduced where applicable in accordance with the Building Code) were applied to structural elements and selected from sheets HQ-S0-2-01 and -02 ("Floor Loading Diagrams") of the structural design drawings. The load from the curtain wall between the third floor and the roof was taken as 135[2] pounds per linear foot (plf) and was applied to the spandrel beam at the third level.

The structural design drawings specify two loading conditions for Level 2: one in which the floor is assumed to be open and subject to roof live loads, and one in which the floor is assumed to be enclosed and subject to occupancy live loads. The latter condition governs for the purposes of this study, so occupancy live loads (i.e., 80 psf) were applied to the Level 2 slab. We did not evaluate vertical wind load demands on the beams as gravity load effects generally govern. These loads were factored and combined using the load combinations from the 2020 edition of the Florida Building Code, Building (2020 FBC), the building code governing the design of the building. Effects of lateral load cases (e.g., wind) were not considered. Occupancy and roof live loads were reduced.

The structural analysis model with the loads described above was used to estimate the factored design demands (axial force, shear force, bending moment, and torsion) in the roof beams and columns. We compared the demands from our model with those shown in the rectification calculations developed by the SEOR (after they included the demand due to the precast facade panels) and found our member demands were similar. It is not surprising that the member demands calculated by WJE would be similar but somewhat different from those calculated by the SEOR given that different engineers preparing different models of the same building using different software usually get different but similar results.

We note that the bending moment demands in the columns are sensitive to the stiffness modifiers assumed when constructing the model. Increasing the stiffness modifier on the third story columns from 0.5 (see Structural Analysis Model section above) to 0.7 was found to greatly increase the bending moment demand, influencing any conclusion about the adequacy of the original column design and rectification measures.

Design Capacity Checks for Original Design

After determining the factored design demands for the key members, we calculated member design capacities, including the code-required capacity reduction factors for the beams and columns at the north elevation. ACI 318-14 was used to calculate the member design capacities. If the member factored design demand exceeded the member design capacities, then the member was deemed not to be code-compliant. If lateral wind load demands are considered, the demands in some members (e.g., columns) may increase even further beyond that considered in our analyses.

[2] See page 5 of 175 in *2024-04-25_CALC_HQ Roof Cantilever Field Fix at North Elevation.pdf* by SEOR

We determined that the following structural members at and north of gridline 2 were not code-compliant for strength in the original design.

- Roof cantilever beams: RSB-73, RSB-76, RSB-79, RSB-82
  - All four beams are not code-compliant for flexure
  - RSB-76 and RSB-79 are not code-compliant for shear and combined shear and torsion
- Third story columns (supporting Roof)
  - Columns G-2, K-2
    - These columns are code-compliant for axial plus biaxial bending strength with a column stiffness modifier of 0.5.
      - These columns would not have code-required strength if the column stiffness modifier is increased to 0.7. Biaxial bending due to the unbalanced moment appears to be a significant factor.
    - These columns do not satisfy code requirements for shear
      - Minimum shear reinforcement is not provided in accordance with ACI 318-14 Section 10.6.2. Note that any possible exceptions to this provision do not apply (see ACI 318-14 Section 9.6.3.1).
  - Columns H-2, J-2
    - These columns are code-compliant for axial plus biaxial bending strength with a column stiffness modifier of 0.5 or 0.7. The increased axial load demand on the interior columns provides a beneficial increase in the bending strength of the column in this case.
    - These columns do not satisfy code requirements for shear
      - Minimum shear reinforcement is not provided in accordance with ACI 318-14 Section 10.6.2. Note that any possible exceptions to this provision do not apply (see ACI 318-14 Section 9.6.3.1).
- First story columns (supporting Level 2)
  - Columns H-2, J-2
    - These columns do not have code-required strength for axial plus biaxial bending strength with a column stiffness modifier of 0.7.
    - These columns are code-compliant for shear

Deflection Checks for Original Design

While there are code limits that apply to calculated deflections, the fact that the curtain wall would not fit in the space between the third floor and the roof is demonstration enough that the design (relative to deflection) was inadequate to allow the building to be constructed.

Methods of analysis used to predict deflection are not standardized and different analysis approaches can result in widely varying predictions of deflection. The soffit beam system amplifies this issue. Relative to deflection, the building code did not anticipate the use of concrete formwork and how the use of soffit beam technology could affect member stiffness. To our knowledge, soffit beam systems have never been tested to see how their stiffness compares against conventional cast-in-place (CIP) concrete. Because soffit beam systems complicate predictions of deflections, our efforts were tailored towards understanding the behavior of the structure rather than trying to identify if the design strictly complied with code deflection requirements. For this reason, we selected a staged construction analysis methodology for our work.

Section 1604.3 of the 2020 FBC provides serviceability requirements, including deflection limits, for new buildings. Relevant deflection limits from the 2020 FBC are provided in Table 5 below. ACI 318-14 Table 24.2.2 provides maximum permissible computed deflections for floors and roofs based on the location (floor or roof) and type of nonstructural element that is being supported. The ACI 318-14 requirements are summarized in Table 6 below. Because the 2020 FBC directly defines the deflection criteria limits, rather than being silent or directing the user to ACI 318-14, we interpret the 2020 FBC provisions to be the controlling requirements, particularly where they are more stringent. However, consideration should be given to the ACI 318-14 provisions where they introduce requirements not addressed by the FBC.

The roof beams north of gridline 2 support a suspended acoustical ceiling (see Sheet HQ-A2-3-3A, dated June 10, 2022) which we consider to not be likely to be damaged by large deflections. The third floor beams support the exterior curtain wall, non-brittle finishes on the floors (see Sheet HQ-A2-4-3A, dated June 10, 2022), and a suspended stucco soffit (see Sheet HQ-A2-3-2A, dated November 15, 2023), which is considered a brittle finish.

Based on the output from the ETABS analyses used for assessment of the original building design, we developed estimates of immediate and long-term deflections under gravity service (unfactored) loads at the tip of the cantilevered roof and third floor beams and at the mid-span of the spandrel beams spanning between the tips of the cantilevered beams. For these analyses, we incorporated the 'staged construction' effects of removal of formwork and the installation of precast panels. We have not performed checks for deflection under wind loads. The staged construction analysis was based on a schedule of events including that the shoring and formwork was removed from the original structural members after 1 month; the precast concrete panels were installed 2 months after the concrete was placed; and the superimposed dead load was applied at 6 months. Relative to this discussion of the analysis-based assessment of deflection for the original building design, the structural rectifications (third story column enlargements and roof upturned beams) that were later installed were not modeled and did not influence the output from the analysis. Table 7 contains estimated beam deflections at the third floor and the roof at various points in time for the tip of the cantilever beam at gridlines G-2 and H-2. Also shown is the mid-span deflection of the spandrel beam spanning between the ends of the cantilever beams at gridlines G and H and the mid-span of the spandrel beam spanning between the ends of the cantilever beams at gridlines H and J. The deflections are reported as both the value in inches and as a fraction of the beam span, L. For the cantilevered beams, L was defined as two times the projection of the cantilever, as specified by the FBC. The analysis-based cantilever deflections are reported relative to the top of the nearest column at gridline 2. The analysis-based spandrel midspan deflection is reported relative to the column at gridline H-2. The analysis-based cantilever and mid-span deflections at similar locations north of gridline 2 are similar. For reference, but with the understanding that analysis-based deflections are not expected to be accurate predictions of measured out-of-levelness, the survey measurements on May 13, 2024, are shown in Table 7 as well. As conveyed by those measurements, mid-span "deflection" of the spandrel beam relative to the support can be estimated by taking the difference between the measured value for the spandrel beam at mid-span and the measured value for the cantilever tip. Contour plots showing analysis-base estimates of deflections at the roof and Level 3 after 10 years are shown in Figure 76 and Figure 77 for the as-designed configuration, respectively.

Table 5. Deflections Limits for Horizontal Members (adapted from Table 1604.3 of 2020 FBC)

Construction	Live Load	Dead Load + Live Load
Roof members supporting plaster or stucco ceiling	L/360	L/240
Roof members supporting non-plaster ceiling	L/240	L/180
Floor Members	L/360	L/240

The deflection limit for the Dead Load + Live Load load combination only applies to the deflection due to the creep component of long-term dead load deflection plus the short-term live load deflection.

For cantilever members, "L" shall be taken as twice the length of the cantilever.

Table 6. Maximum permissible calculated deflections from ACI 318-14 Table 24.2.2

<table>
<tr><th>Member</th><th colspan="2">Condition</th><th>Deflection to be considered</th><th>Deflection limitation</th></tr>
<tr><td>Flat roofs</td><td colspan="2" rowspan="2">Not supporting or attached to nonstructural elements likely to be damaged by large deflections</td><td>Immediate deflection due to maximum of Roof Live Load, Snow Load, and Rain Load</td><td>L/180[1]</td></tr>
<tr><td>Floors</td><td>Immediate deflection due to Live Load</td><td>L/360</td></tr>
<tr><td rowspan="2">Roofs or floors</td><td rowspan="2">Supporting or attached to non-structural elements</td><td>Likely to be damaged by large deflections</td><td rowspan="2">That part of the total deflection occurring after attachment of nonstructural elements, which is the sum of the time-dependent deflection due to all sustained loads and the immediate deflection due to any additional live load[2]</td><td>L/480[3]</td></tr>
<tr><td>Not likely to be damaged by large deflections</td><td>L/240[4]</td></tr>
</table>

[1]Limit not intended to safeguard against ponding. Ponding shall be checked by calculations of deflection, including added deflections due to ponded water, and considering time-dependent effects of sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

[2]Time-dependent deflection shall be calculated in accordance with 24.2.4, but shall be permitted to be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be calculated on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

[3]Limit shall be permitted to be exceeded if measures are taken to prevent damage to supported or attached elements.

[4]Limit shall not exceed tolerance provided for nonstructural elements.

The analysis-based estimated deflections reported in Table 7 are based on a staged construction analysis and long-term creep deflection analysis using the CEB-FIP 2010[3] model, which is the most advanced model for predicting long-term deflections that is available for use in ETABS. Past WJE experience on

[3] Comite Euro-International Du Beton, 2010

similar projects demonstrates that the default values associated with this model tend to underpredict field-measured long-term behavior. As such, we have incorporated the following factors into the analysis model: a creep coefficient of 1.5 (default=1.0), a shrinkage coefficient of 1.0 (default=1.0), and a concrete aging coefficient of 1.0 (default=0.8). Increasing the creep coefficient by 50 percent and the concrete aging coefficient by 25 percent was intended to help overcome some of the issues with concrete models underpredicting long-term deflections. The effects of shrinkage were set to their default value because the concrete is not suspected to exhibit unrestrained shrinkage outside the range of normal concrete behavior. The total long-term deflection shown in Table 7 is at 10 years. We are also reporting the immediate deflection under full (unfactored) live load and the long-term creep component of the total dead load (self-weight plus superimposed dead load) plus the immediate deflection under full (unfactored) live load.

Table 7. Analysis based long term deflection estimates for the as-designed condition

FL	GL	Loc.	IM SW (Pour +#mo)	SW +2mo (pre-PC)	SW After PC Install	PC Inc.	+3mo (~ May 2024)	Survey (May 13, 2024	Total LT	LT Creep		IM LL	
--	--	--	[in]	[in]	[in]	[in]	[in]	[in]	[in]	[in]	[L/xx]	[in]	[L/xx]
R	G/1	EC	0.02	0.1	0.2	0.1	0.3	1.1	0.6	0.2	955	0.02	13865
R	G.5/1	MS	0.5	1.2	1.6	0.4	2.2	2.0	3.3	1.0	--	0.08	--
R	H/1	EC	0.4	1.0	1.3	0.4	2.0	2.1	3.1	1.0	222	0.06	3494
R	H.5/1	MS	0.4	1.0	1.4	0.4	2.2	2.4	3.3	1.1	--	0.07	--
3	G/1	EC	0.1	0.3	0.4	0.1	0.5	--	0.9	0.4	532	0.05	4203
3	G.5/1	MS	0.7	1.7	1.9	0.2	2.4	--	4.2	1.6	--	0.32	--
3	H/1	EC	0.2	0.6	0.7	0.1	0.9	--	1.6	0.6	354	0.10	2338
3	H.5/1	MS	0.2	0.6	0.7	0.1	0.9	--	1.7	0.7	--	0.11	--

Notes: IM = Immediate, LT = Long Term (after 10 years), LL = Live Load, SW = Self Weight, PC = Pre-Cast Fascia Panel Weight, mo = month, EC = End of Cantilever, MS = mid-span of spandrel, R = Roof, GL = Gridline, Loc. = Location, Inc.= Increment, FL = Floor

Structural Rectification Design Review

To evaluate the installed and recommended rectification measures, WJE modified the ETABS model described in the preceding section to incorporate the rectification measures into our staged construction analysis. Specifically, the following modifications were made to the model (Figure 78):

- The geometry of the Level 3 columns (supporting the Roof level) was updated using shell elements to reflect the new geometry in the retrofitted configuration. Note that the retrofit portion was not modeled as extending or attaching to the Level 3 floor slab, consistent with it having been saw-cut and removed approximately 2 inches above the third floor in the current as-built configuration.
- The Level 1 columns (supporting Level 2) were enlarged on three sides, resulting in overall cross-sectional dimensions of 22 inches by 28 inches (modeled, but not depicted in Figure 78 due to stage of construction at which snapshot of the model was extracted).

The depth of the beams at the roof level oriented in the north-south direction (parallel to the cantilever) was increased by 12 inches on the top surface of the roof level (modeled, but not depicted in Figure 78).

Design Capacity Checks for Rectified Design

The capacities for the members were determined using the ACI 318-14 as described above for the building model based on the original design. The structural members that were previously deficient were found to have the following condition after the incorporation of the rectification measures into the model:

- Roof cantilever beams
  - RSB-73 and RSB-82
    - After the upturned beams at the roof were installed, these beams are code-compliant for flexural strength.
  - RSB-76 and RSB-79
    - These beams were still not code-compliant for flexural strength when only the column enlargements are considered but after the upturned beams at the roof are considered, these beams are code-compliant for flexural strength.
    - These beams are still *not* code-compliant for strength for shear and combined shear and torsion. We note that no rectifications to increase the shear strength of the beams have yet been proposed or implemented.
- Third story columns (supporting Roof)
  - Columns G-2, K-2
    - These columns are code-compliant for axial plus biaxial bending strength with a column stiffness modifier of 0.5.
      - These columns would not have code-required strength if the column stiffness modifier is increased to 0.7. Biaxial bending due to the unbalanced moment appears to be a significant factor.
    - These columns still do not satisfy code requirements for shear
      - Minimum shear reinforcement is not provided in accordance with ACI 318-14 Section 10.6.2. Note that any possible exceptions to this provision do not apply (see ACI 318-14 Section 9.6.3.1). We note that no rectifications to increase the shear strength of these columns have yet been proposed or implemented.
  - Columns H-2, J-2
    - These columns are not code-compliant for axial plus biaxial bending strength with a column stiffness modifier of 0.5, even after installation of the column enlargement.
      - This result is due to the condition at the base of the column where the column enlargement was demolished in combination with the biaxial bending in the column. If 12 inches of the column enlargement are grouted (see Figure 79) with non-shrink grout having a minimum 28-day compressive strength of 5,000 psi, then the column would be code-compliant for strength for axial plus biaxial bending.

            - If the full column base is re-grouted, then the capacity (particularly shear capacity) of the third floor beams underneath the base of the column enlargement needs to be revisited.
        - There is not a sufficient tie between the enlarged column/corbel at the top of the column and the existing column.
            - This tie should be designed for the horizontal component of the maximum diagonal strut force that is determined through staged-construction analysis. Long-term load transfer effects due to creep should be considered.
            - This tie should extend to the south face of the column.
        - These columns are not code-compliant for shear
            - While the column enlargement provides additional shear capacity, it is not sufficient for the additional shear demand as a result of the increased flexural demand on the combined column and column enlargement. Further at the base of the column, all of the shear demand must pass through the original column as a result of cutting the bottom of the column enlargement such that there is a shear lag effect. Additional rectification of these columns is required. See the Discussion and Recommendations section below for conceptual rectification options.
- First story columns (supporting Level 2)
    - Columns H-2, J-2
        - The rectified columns are code-compliant for axial plus biaxial bending strength with a column stiffness modifier of 0.7. See further comments on the proposed column jacket rectification in the proceeding Discussion and Recommendations section.

Deflection Checks for Rectified Design

Based on the output from the ETABS analyses used for assessment of the rectified building design, we developed estimates of immediate and long-term deflections under gravity service (unfactored) loads at the tip of the cantilevered roof and third floor beams and at the mid-span of the spandrel beams spanning between the tips of the cantilevered beams, incorporating the 'staged construction' effects of installing the rectification measures described several months after the structure was originally built and the precast panels were installed. We have not performed checks for deflection under wind loads. The staged construction analysis and long-term creep deflection analysis used the same CEB-FIP modeling parameters described above. All modeling parameters incorporated in the as-rectified building model were identical to those in the as-designed model. The staged construction analysis was based on a schedule of events including that the shoring and formwork was removed from the original structural members after 1 month; the precast concrete panels were installed 2 months after the concrete was placed; the rectifications (upturned roof beams and column enlargements at Levels 1 and 3) were assumed to be installed 3 months after installation of the precast panels; and the superimposed dead load was applied at 6 months. All rectifications were incorporated into the analysis model at the same timestep to represent a reasonable simplification which would not unduly affect the overall behavior predicted by the analysis.

Table 8 contains estimated beam deflections at the third floor and the roof at various points in time for the tip of the cantilever beam at gridlines G-2 and H-2. Also shown is the mid-span deflection of the spandrel beam spanning between the ends of the cantilever beams at gridlines G and H and the mid-span of the spandrel beam spanning between the ends of the cantilever beams at gridlines H and J. The deflections are reported as both the value in inches and as a fraction of the beam span, L. For the cantilevered beams, L was defined as two times the projection of the cantilever, as specified by the FBC. The analysis-based cantilever deflections are reported relative to the top of the nearest column at gridline 2. The analysis-based spandrel midspan deflection is reported relative to the column at gridline H-2. The analysis-based cantilever and mid-span deflections at similar locations north of gridline 2 are similar. For reference but with the understanding that analysis-based deflections are not expected to be accurate predictions of measured out-of-levelness, the survey measurements on May 13, 2024, are shown in Table 8 as well. As conveyed by those measurements, mid-span "deflection" of the spandrel beam relative to the support can be estimated by taking the difference between the measured value for the spandrel beam at mid-span and the measured value for the cantilever tip. Contour plots showing analysis-base estimates of deflections at the roof and Level 3 after 10 years are shown in Figure 80 and Figure 81 for the rectified configuration, respectively. The total long-term deflection shown in Table 8 is at 10 years. We are also reporting the immediate deflection under full (unfactored) live load and the long-term creep component of the total dead load (self-weight plus superimposed dead load) plus the immediate deflection under full (unfactored) live load.

Table 8. Analysis based long term deflection estimates for the rectified condition

FL	GL	Loc.	IM SW (Pour +#mo)	SW +2mo (pre-PC)	SW After PC Install	PC Inc.	+3mo (~ May 2024)	Survey (May 13, 2024	Total LT	LT Creep		IM LL	
--	--	--	[in]	[in]	[in]	[in]	[in]	[in]	[in]	[in]	[L/xx]	[in]	[L/xx]
R	G/1	EC	0.02	0.1	0.2	0.1	0.4	1.1	0.4	0.1	4043	0.005	48151
R	G.5/1	MS	0.5	1.2	1.6	0.4	2.2	2.0	2.9	0.6	--	0.06	--
R	H/1	EC	0.4	1.0	1.3	0.4	2.0	2.1	2.4	0.4	587	0.03	7635
R	H.5/1	MS	0.4	1.0	1.4	0.4	2.2	2.4	2.6	0.4	--	0.03	--
3	G/1	EC	0.1	0.3	0.4	0.1	0.5	--	0.9	0.4	542	0.05	4428
3	G.5/1	MS	0.7	1.7	1.9	0.2	2.4	--	4.2	1.6	--	0.32	--
3	H/1	EC	0.2	0.6	0.7	0.1	0.9	--	1.6	0.6	381	0.09	2582
3	H.5/1	MS	0.2	0.6	0.7	0.1	0.9	--	1.6	0.6	--	0.10	--

Notes: IM = Immediate, LT = Long Term (after 10 years), LL = Live Load, SW = Self Weight, PC = Pre-Cast Fascia Panel Weight, mo = month, EC = End of Cantilever, MS = mid-span of spandrel, R = Roof, GL = Gridline, Loc. = Location, Inc.= Increment, FL = Floor

DISCUSSION AND RECOMMENDATIONS

Summary of Current and Proposed Rectification Measures

Our review of the original design of the structural members at the north elevation of the building showed that the design of multiple members in the load path from the roof to the foundation did not satisfy code requirements regarding strength. With regard to deflection, it is factual that the installation of the exterior curtain wall was prevented by the deflections experienced by the framing at this elevation, thus demonstrating the unsuitability of the structural design to the architectural and construction requirements.

We understand that design errors were acknowledged by the SEOR, and they attributed the source of the design errors to the weight of the precast panels. Rectification measures to the third story columns at gridlines H-2 and J-2 were proposed by the SEOR. These measures consisted of enlarging these columns and adding a corbel-like feature at the top of the column. The effect of this rectification was to add axial and flexural capacity to these columns and to shorten the span of the cantilever beams that are supported by them, improving the long-term deflection performance of the beams. Later, the base of the column enlargement was saw-cut and removed at the suggestion of Lakdas so that the enlargement did not transmit additional loads to the beams supporting the third floor. To our knowledge, no remedial work has been proposed to date for the third-floor beams or to the columns that support them, nor to other structural members away from the north cantilever.

Soon after the third level column enlargements were installed, upturned beams were installed at the roof along portions of gridlines G, H, J, and K. Adding these beam extensions strengthened the cantilever roof beams and also stiffened them, reducing the long-term deflection at the tip of the cantilever (i.e., the north elevation of the building).

Around the same time that the rectification measures for the two interior third story columns were implemented, the footings for those columns were enlarged. However, no modifications to any first story columns was specified at that time. Later, the first story columns and their foundations were proposed to be modified such that all column foundations on gridline 2 were changed from spread footings bearing on soil to micropiles and a pile cap and three-sided column jackets were proposed to strengthen these columns and, we understand, to complete the load path from the existing structure to the new foundations. The column jackets and new foundations have not been implemented to date.

Beams at Third Floor and Roof

Third Floor Cantilever Beams

The cantilever third floor beams north of gridline 2 are code-compliant for strength as designed. No known issues related to deflection appear to have developed for these beams. In our opinion, no further action needs to be taken to strengthen or stiffen these beams, but the deflection of these beams, and more importantly the relative deflection between the third floor and roof beams, should be taken into account while designing the curtain wall system that will span between the third floor and the roof. See the discussion section on long term deflections below regarding this issue.

Roof Cantilever Beams

The cantilever roof beams north of gridline 2 on gridlines H and J are not code-compliant for shear strength or for combined shear and torsion strength, specifically, the section of the beam immediately north of the column enlargement. The column enlargement, which shortened the span of the cantilever, was not sufficient to resolve the shear strength inadequacy in these beams largely due to the facts that a) so much demand was already present in the beam due to the dead loads in-place at the time of the rectifications, and b) the corbel on the column enlargement does not extend further north on the beam. The SEOR appears to conclude in their calculations that the section of the beam just to the north of the column enlargement does have sufficient shear strength based on a combined shear resistance from the beam stirrups acting in a classical strut and tie shear resisting mechanism plus shear friction resistance that relies on longitudinal steel on the bottom side of the beam just above the precast soffit (Figure 82). However, in our understanding of the ACI provisions for beam shear and our experience[4], combining shear friction resistance with classical strut and tie resistance to calculate the total sectional shear capacity of any beam is neither appropriate from a mechanics perspective nor permitted by ACI.

In our opinion the beam sections north of the column enlargement (i.e., along gridlines H-2 and J-2), need to be strengthened. Strengthening could be accomplished through the addition of a beam jacket with closed stirrups (e.g. Figure 83).

As noted above for the third floor cantilever beams, the deflection of the roof beams, and more importantly the relative deflection between the third floor and roof beams, should be taken into account while designing the curtain wall system that will span between the third floor and the roof. See the discussion section on long term deflections below regarding this issue.

Spandrel Beams at Third Floor and Roof

We found that the spandrel beams at the third floor and roof are code-compliant for shear and flexural strength, and there is currently no data suggesting they are deflecting excessively. No rectifications on these beams are necessary; however, the deflection of these beams, and more importantly the relative deflection between the third floor and roof beams, should be taken into account while designing the curtain wall system that will span between the third floor and the roof. See the discussion section on long term deflections below regarding this issue.

Long-term Deflections and Implications on Building Performance

Structures should be designed so that the performance of nonstructural elements, such as floor and ceiling finishes or facade elements, attached to structural members are not adversely affected. The structural member deflections that are of importance are the long-term deflections under sustained loads (typically dead loads) and the immediate deflections under live load.

Long-term deflections associated with creep and shrinkage in concrete buildings are generally considered to be substantially complete between five and ten years after construction, with the majority of those deflections occurring in the first one to two years after sustained loads are applied. Figure 84 (adapted from ACI 318-14) shows an idealization of how deflections due to creep progress over time after application of the loads that will be sustained over time. New instances of long-term creep are created when new sustained loads are applied. Installation of architectural components that are sensitive to structural deflections (e.g., floor finishes, ceiling finishes, and curtain walls) would ideally be installed as late as reasonably practicable in the construction process to minimize effects of long-term deflections on their performance.

Predicting long term deflections in concrete structures using analytical models is challenging. In-field measurements of the floor level elevation of cast-in-place concrete construction often suggest that deflections are far greater than those predicted by even the most sophisticated engineering analyses.

[4] A co-author of this report, Jeff Rautenberg, serves on ACI Committee 445 – Shear and Torsion. Another co-author, Zack Coleman, serves on ACI Sub-Committee 445-0F – Interface Shear.

However, when scrutinized closely, the apparent mismatches between predicted floor elevation profiles and measured in situ profiles is often illusory. There are numerous reasons for this, though discussion of this subject in detail is far beyond the scope of this report. The deflection calculations developed by WJE from our structural analysis models for the original design and the rectified design and our comparisons with survey data were more tailored towards understanding the behavior of the structure than trying to identify if the design strictly complied with code deflection requirements.

Deflections for Original Design

It is clear due to the fact that the curtain wall would not fit in the space between the third floor and the roof that the design (relative to deflection) was inadequate to allow the building to be constructed. For the same reason, it is also clear that the SEOR did not fully consider long term deflections in the design of the structural framing north of gridline 2.

Deflections for Rectified Design

We understand that the curtain wall is being redesigned and refabricated based on the rectified structure. However, if long term deflections of structural members are not properly taken into consideration, then there may be future issues with the curtain wall. The potential for relatively large long-term deflections at the north cantilever should be taken into consideration when detailing these components and their connection to the structure; sufficient allowance for relative displacements between Level 3 and the roof cantilevers should be accommodated.

Our analysis and assessment of the long term deflections contained within this report is a simplified assessment. The SEOR needs to perform their own long term assessment of the deflections of the structural members considering additional factors such as whether or not additional slab topping will be necessary to address sloping floors or other occupant comfort issues (e.g., desks being uncomfortably out of level).

Given that the magnitude of roof deflections at the north elevation was not fully considered during the design of the structure, it is possible that the roof area adjacent to the north parapet is no longer adequately sloped towards the primary drainage system. The survey elevation measurements indicate that there is more than 2 inches of relative downward movement from gridline 2 to the tip of the cantilevers at gridlines H and J. Sheet HQ-A2-4-4A, dated June 10, 2022, indicates that the roofing at these locations is to be sloped at 1/8 inch per foot towards gridline 2, which is a total of 1.3 inches for the cantilever. It is not clear if the roofing and roof slope was designed and installed based on the undeflected or deflected conditions. Accordingly, ponding (i.e., the accumulation of standing water) may be possible on the roof near the north elevation, creating more load which may exacerbate deflections. Typically, the structural engineer would design to control deflections to prevent ponding based on the roof slope provided by the architect. We have not considered additional load due to potential ponding in our analysis. We recommended that the SEOR, as part of their evaluation of the deflections of the structural framing north of gridline 2, evaluate the implications of rain on the structural performance of the roof system or ensure that the roof sloping and drainage systems are adequate to preclude ponding.

Third Story Columns

Columns at Gridlines G-2 and K-2

The third story columns at gridlines G-2 and K-2 have sufficient axial plus biaxial bending capacity in the as-designed condition when a bending stiffness modifier of 0.5 on the gross moment of inertia is used for the columns in the structural analysis model used to determine the loads in the column. However, ACI 318-14 Table 6.6.3.1.1 (a) allows for a bending stiffness modifier of 0.7 for elastic analysis of columns at the factored load level. Alternatively, ACI 318-14 Table 6.6.3.1.1 (b) provides an equation that can be used to calculate a bending stiffness modifier ranging between 0.35 and 0.875 based on the axial load and bending moment in the column. Based on the stiffness modifier calculated using this equation, a bending stiffness factor of 0.5 on columns is low and it may not be justified. If the bending stiffness modifier is increased to 0.7, then the amount of bending moment required to be resisted by the column will increase to the point where the column is no longer code-compliant for axial plus biaxial bending of the column. As noted below, rectifications for these columns are necessary to make them code-compliant for shear. We recommend that the flexural strength of these columns also be increased as part of the rectifications.

More importantly, and the primary reason that accurate assessment of the bending moment demand in these columns is a key factor to be addressed, is that these columns, as-designed, are not code-compliant for shear. Because the shear demand is driven by unbalanced moments from the structural framing above and below, the stiffness of the columns governs the shear they attract. Even without consideration of any contribution from lateral wind loading, the shear demand in the concrete is great enough that the minimum shear reinforcement provision of ACI 318-14 Section 10.6.2 is triggered. The column ties provided, if more closely spaced than was designed (less than d/2, or approximately 6 5/8 inches), would have been adequate as the shear reinforcement. However, the column tie spacing is noted on Sheet HQ-S4-2-01, dated May 24, 2024, as "not to exceed least column dimension or 16 vertical bar diameters." For a 16 inch by 16 inch column with No. 10 vertical bars, the tie spacing would be 16 inches, which is far greater than 6 5/8 inches, meaning that the ties with the spacing called for in the drawings cannot serve as the required shear reinforcement.

To make the columns code-compliant for shear, it will be necessary to provide shear reinforcement to supplement the ties currently spaced at 16 inches. One common way of providing additional shear reinforcement to columns is to wrap them with fiber reinforced polymers (e.g., carbon fiber). However, the use of fiber reinforced polymer column wraps may not be appropriate here, may not be allowed, or may have special requirements depending on specific code provisions. If wraps are deemed to be acceptable for this application, these wraps should be designed and detailed by the SEOR or can be delegated. Another common method to increase the shear strength of the columns is to jacket the columns and provide closed closely spaced ties as part of the rectification. The closely spaced ties serve as the shear reinforcement.

Columns at Gridlines H-2 and J-2

The third story columns at gridlines G-2 and K-2 had sufficient axial plus biaxial bending capacity in the original as-designed configuration. However, the addition of the column enlargement increases the bending moment demand in these columns. Since the bottom two inches of the column enlargement at the third floor was cut and demolished, all of the bending moment demand within this two inch height must be resisted by the original column. We determined that once the enlargement was added and then the lower portion removed, the remaining original column cross-section is no longer code-compliant for axial plus biaxial bending due to the cut but could be brought back into compliance if 12 inches of the base of the column are regrouted (see Figure 79).

Moreover, these columns were not code-compliant for shear as originally designed. In the as-rectified configuration (with the enlargement), there is additional shear demand on the column as the result of the column enlargement and corbel acting as a strut to transfer some of the bending moment demand from the beam above. The column enlargement does provide some additional shear capacity, but the actual distribution and load sharing of shear demand between the original column and the column enlargement is difficult to ascertain. Further, the total shear demand on the total cross section (original plus column enlargement) is large enough in magnitude that ACI 318-14 Section 10.6.2 requires that minimum shear reinforcement be installed, and currently no shear reinforcement extending the full depth of the combined cross section is provided. As such, it is our opinion that these columns require additional measures to increase the shear capacity to a level sufficient for the demand. We believe a reasonable rectification is to wrap the columns with fiber reinforced polymers (if allowed by code provisions, see preceding section) or to jacket the columns and add closely spaced ties as discussed for columns G-2 and K-2 above. Alternatively, threaded rods with washer plates and nuts could be used to tie the two columns together as shown conceptually in Figure 86.

Corbel at Top of Columns

The column enlargement and corbel at the top of the enlarged column are connected to the original column with "U"-shaped No. 5 bar stirrups at 12 inches on center (see Figure 12 and Figure 13). While this distributed connection "stitches" the two columns together, it is our opinion that these "U"-shaped stirrups do not sufficiently resolve the horizontal force that is developed as a result of the inclined "strut" in the corbel. To sufficiently resolve the horizonal force, a tie that spans from the north side of the corbel to the south side of the original column is appropriate. Lakdas also suggested such a tie in their report (Figure 87). This tie should be designed for the horizontal component of the maximum diagonal strut force that is determined through staged-construction analysis. Long-term load-transfer effects due to creep should be considered in the design.

First Story Columns

We found from our analyses that the first story columns at gridlines H-2 and J-2 as designed and currently installed do not have sufficient design strength to resist the full factored design demand required by the 2020 FBC. Currently, the demand from the current load on these columns, even without live load, is near their factored design strength. It will be necessary to strengthen these columns to meet the load requirements of the building code before the building can be finished and occupied. Based on our analysis, the column jackets proposed to be added by the SEOR will strengthen the columns, and after these rectifications are completed, the first story columns should have sufficient capacity to resist the full factored design demand required by the 2020 FBC.

That said, we have some reservations about how the column jackets will be implemented. Currently, the ties for the column jacket (Figure 18), have their termination on the northern side of the column, embedded 4 inches into the column using epoxy adhesive. Terminating the tie in this manner will not provide a closed tie. Closed ties will provide a more reliable means of restraining and bracing the flanges of the column bracket and will also serve to provide additional confinement to the original column. We recommend that the tie detail be modified such that a closed tie can be provided. One possible option to achieve a closed tie is shown conceptually in Figure 85.

In addition to modifying the column jacket details to achieve a closed tie, we also recommend that the surface of the existing column be roughened to an amplitude of 3/8 inch (peak-to-trough) and cleaned prior to installation of the column jacket. This roughening will ensure that the jacket is fully engaged in load resistance by enabling the required transfer of loads to take place through shear friction between the original column and the column jacket.

In conjunction with the above recommendations, we also recommend that the SEOR review the design of the existing first story columns and the rectifications sequence to make sure that sufficient strength is maintained in the existing structural members for current loading conditions throughout the installation of the rectifications.

Foundations

WJE reviewed the current and proposed foundation modifications. We provided review comments for the design of the micropiles and pile cap design in a previous letter, dated November 6, 2024, that is attached to this report. We believe, based on our review of the documents and calculations provided to us and on confirmation of demands through our own analyses, that modifications to the current foundations are necessary and the micropile and pile cap solution proposed by the SEOR is a reasonable way to strengthen the foundations.

In the pile cap design proposed by the SEOR, there are relatively few dowels between the top of the existing footing and the new pile cap. The concrete for the new column jacket is intended to be placed such that the concrete is poured tight against the second floor beams; however, shrinkage of the new column jacket concrete may cause some slight gaps to open at the top and bottom of the column. Additionally, relative settlement between the two foundation systems may also occur. The cumulative deformation from these sources may result in deformation demand between the bottom of the new pile cap and the top of the existing pile cap. This deformation demand can either be resisted (i.e., the new and existing structural members are coupled so that they deform and move together), or the structural elements can be decoupled and a gap allowed to form. The current design of the column jacket provides for some coupling through the column tie anchorages. Roughening the surface of the existing column, as described in the section above, will serve to more tightly couple the new and existing columns together. Additionally, shrinkage reducing admixtures and extended moist curing may be appropriate to mitigate shrinkage cracking.

If the original and new structural elements are not tightly coupled together, a gap will try to form between the new pile cap and existing footing. Since the proposed dowels tying these two elements together have a short length, the imposed deformation demand may result in breakout of the ties from the elements, possibly impacting the strength of the pile cap. We recommend that the SEOR reevaluate these ties in conjunction with the column roughening recommendation as additional ties between the new pile cap and existing footing may be necessary.

LIMITATIONS AND NEXT STEPS

Based on the limited peer review of the design of the structural members at the north elevation of the building that we performed, WJE has identified conditions and potential deficiencies in the original design and in the design of the rectification measures that do not appear to have been addressed which should be reviewed by the SEOR (TT) for their consideration and action. As the licensed design professional in responsible charge for the structure, the SEOR should determine if further structural modifications are necessary to ensure that the structure is code-compliant; protects occupant safety; and provides for serviceability over the long term. Updated drawings, calculations, and other documentation should be provided for review, coordination, approval, and record purposes.

The estimated deflection values generated by the studies that we performed and provided in this report were used by WJE to develop comparisons with survey data collected by others and to more generally further our understanding of the performance of the structural framing north of gridline 2. Prediction of long-term deflections in concrete structures based on analytical methods can be misleading if they understood to be anything other than rough estimates of actual in-field performance. The actual long-term deflection of the structural members may vary significantly from the estimates developed and reported by WJE herein. The SEOR should develop their own understanding of the future deflection of the relevant structural framing to be used for the design of the curtain wall between the third floor and the roof and for other serviceability considerations such as drainage.

The peer review services provided by WJE are intended to call attention to areas of ambiguity, possible deficiency, or other anomalies that were identified during a relatively limited review of available documents. Our review necessarily relied on the information in those documents and inaccuracies in those documents may be reflected in our conclusions. The services provided by WJE should be viewed in a proper context and not be construed as replacing or otherwise altering the contractual responsibilities of the Project Team members as they relate to the design and construction of the building. Although we have endeavored to identify areas of concern, our scope of services has not included an exhaustive or minutely detailed analysis of each design, component, or system specified on the drawings. Accordingly, the responsibility for a proper design remains solely with the design professional whose seal appears on the drawings.

WJE will continue to evaluate the building design for the full building structure as has been requested and will provide peer review comments for the Phase 2 scope of our work in a separate letter.

FIGURES

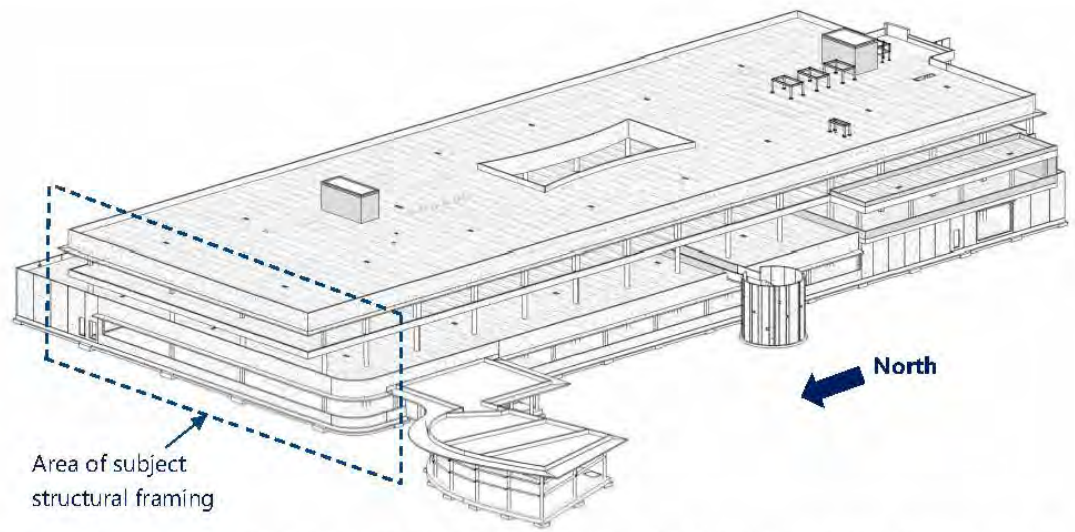


Figure 1. Isometric view of the Fort Lauderdale Police Headquarters building from Sheet HQ-S0-1-00 (dated June 10, 2022) of the structural design drawings. Annotations by WJE.

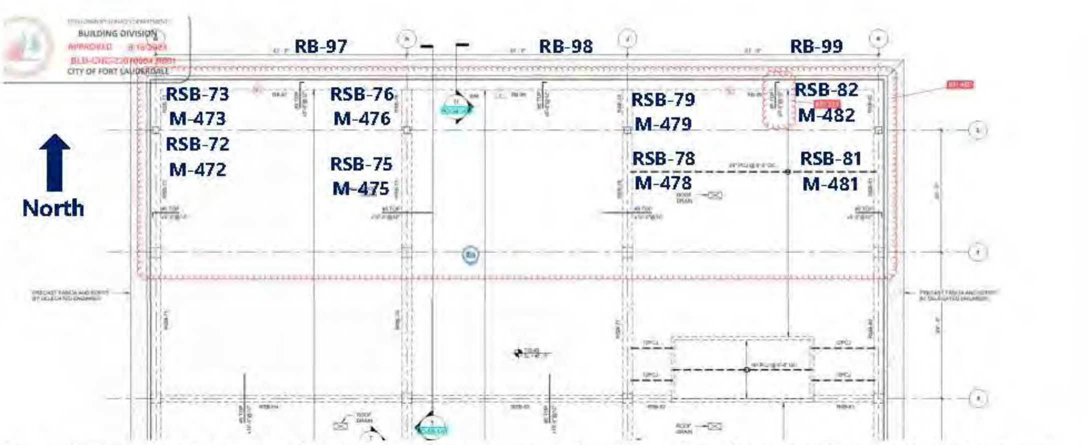


Figure 2. Plan view of structural framing at roof at north end of building (from Sheet HQ-S2-2-4A dated November 15, 2023). Annotations in blue by WJE including structural drawing (top) and precast shop drawings (bottom) beam designations. Area of affected framing is generally at and north of gridline 2.

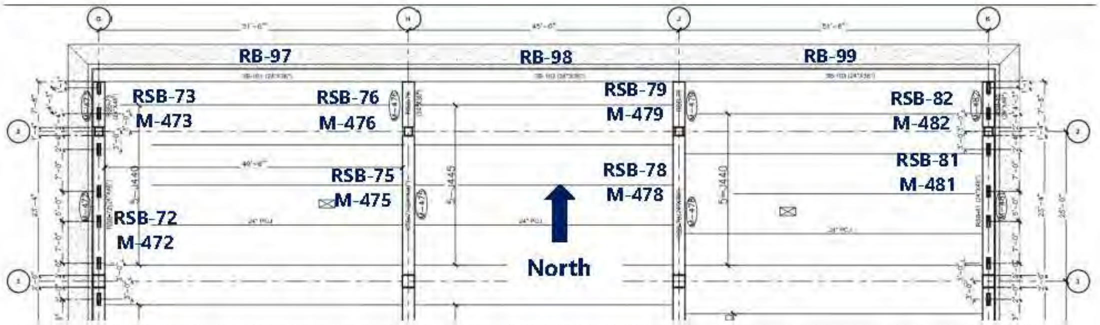


Figure 3. Plan view of structural framing at roof at north end of building (from precast shop drawings Sheet S-3A-PL dated August 16, 2022). Annotations in blue by WJE including structural drawing (top) and precast shop drawings (bottom) beam designations. Area of affected framing is generally at and north of gridline 2.

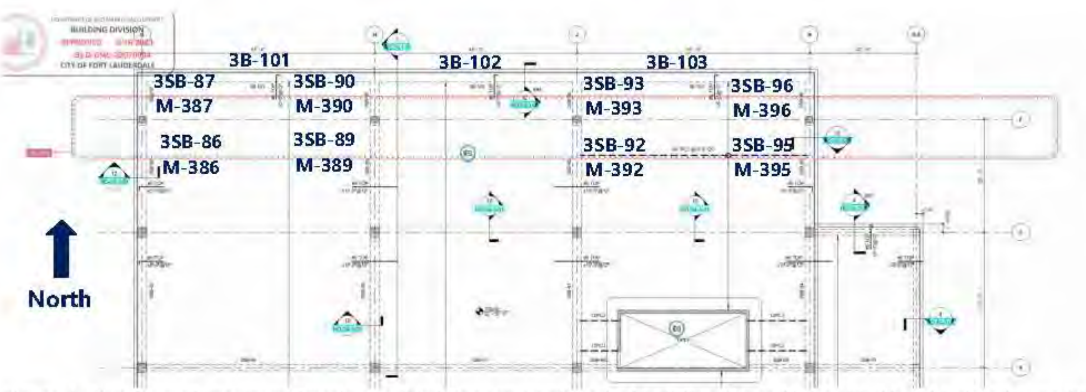


Figure 4. Plan view of structural framing at third floor at north end of building (from Sheet HQ-S2-2-3A dated June 10, 2022). Annotations in blue by WJE including structural drawing (top) and precast shop drawings (bottom) beam designations. Area of affected framing is generally at and north of gridline 2.

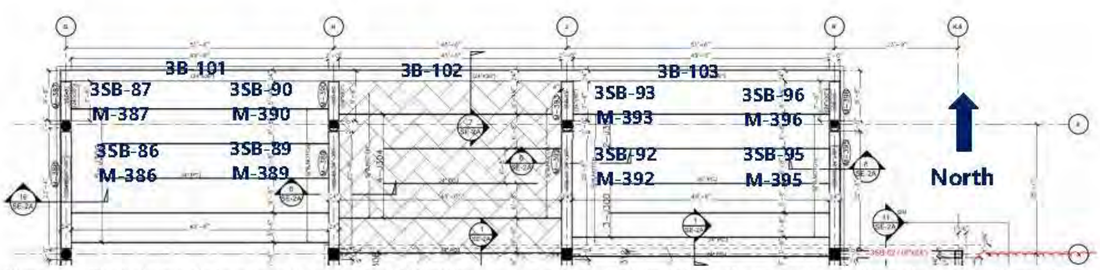


Figure 5. Plan view of structural framing at third floor at north end of building (from precast shop drawings Sheet S-2A dated July 6, 2023, 2022). Annotations in blue by WJE including structural drawing (top) and precast shop drawings (bottom) beam designations. Area of affected framing is generally at and north of gridline 2.

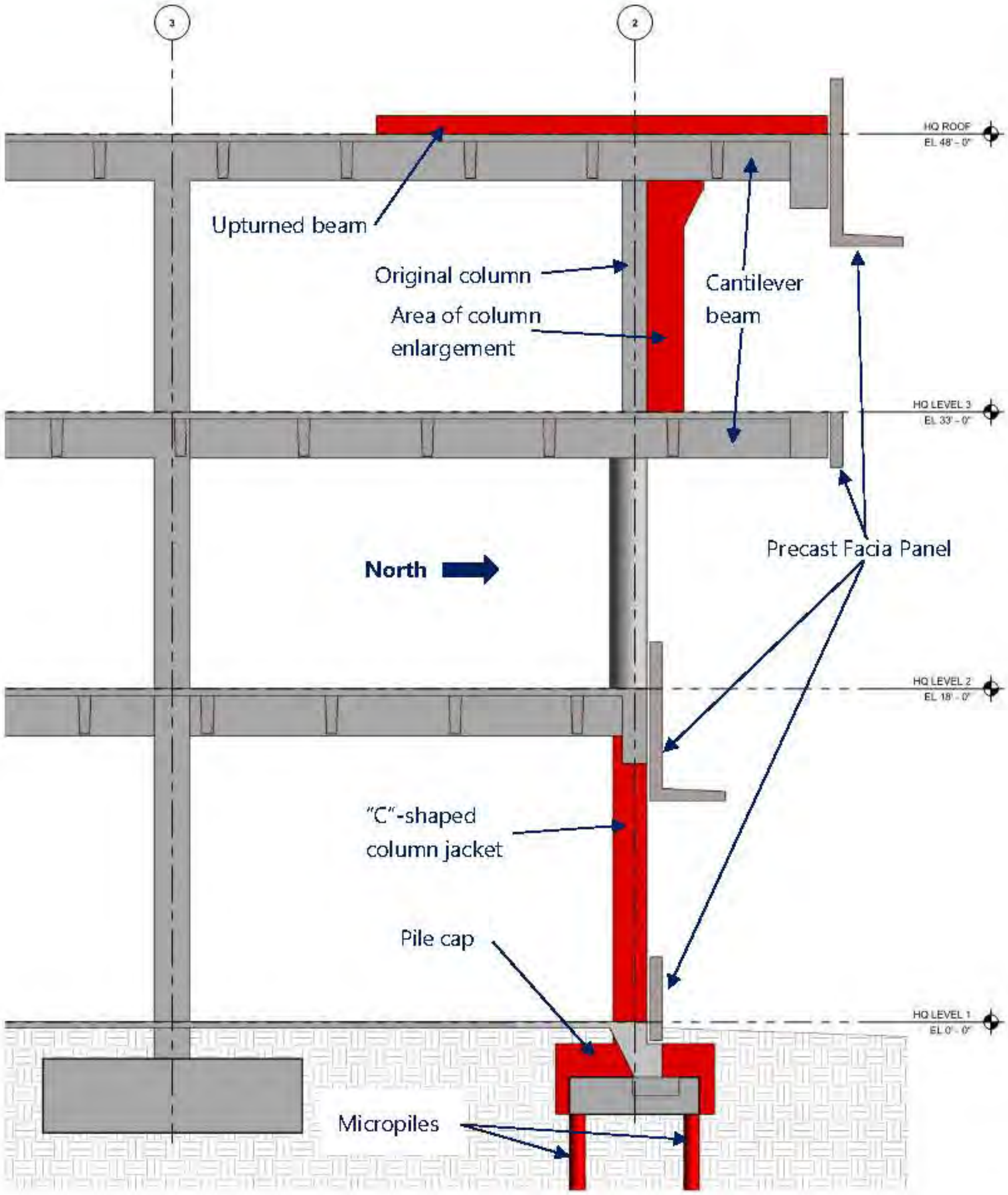
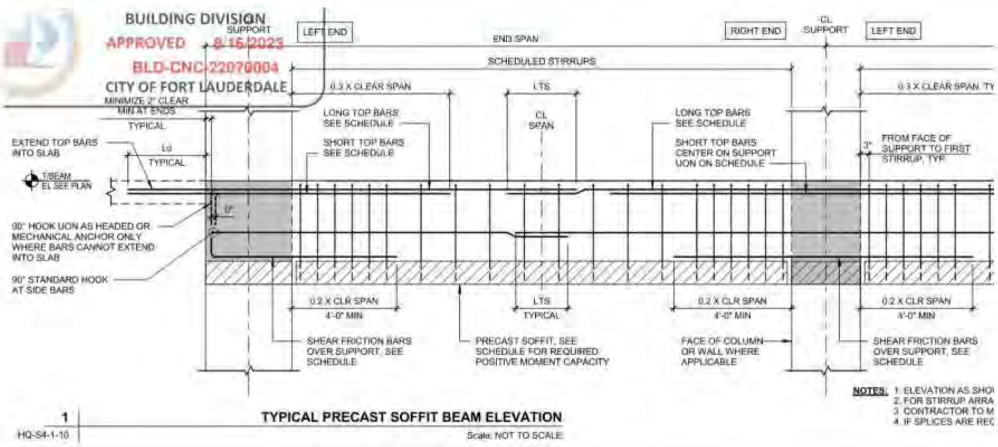


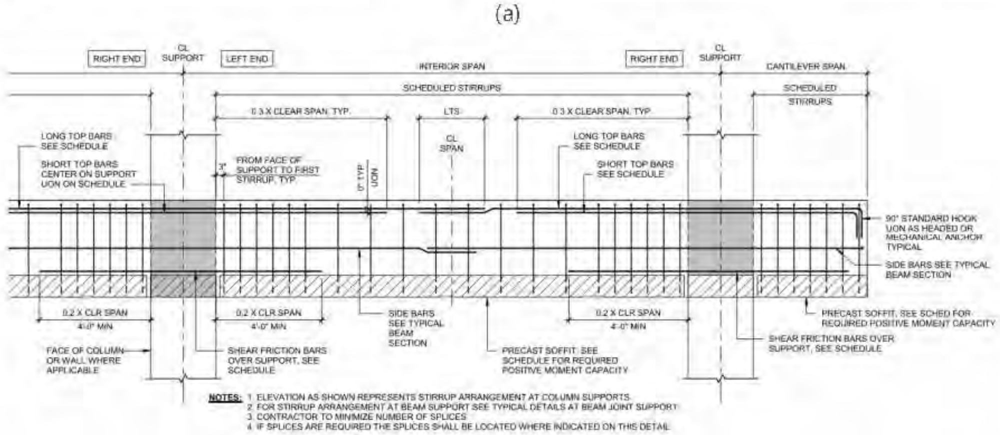
Figure 6. Building section at gridline H showing area north of gridline 3 (from file *Police HQ Views*). Annotations in blue by WJE. Section shows implemented rectifications at third story and proposed rectifications at first story and foundations in red.

BEAM MARK	SIZE		REINFORCEMENT											
	W (IN)	H (IN)	LEFT END TOP BARS		RIGHT END TOP BARS		SHEAR FRICTION BARS	BOTTOM BARS or M_u		SIDE BARS EACH FACE	STIRRUPS or V_u		
			LONG	SHORT	LONG	SHORT		LONG	SHORT		TYPE	SIZE	SPACING FROM EACH END
RSB-72	24	48	–	–	4#9	2#6	3#6	250 ft-k	–		–	–	Vu = 95 k
RSB-73	24	48	–	–	4#9	2#6	3#7	125 ft-k	–		–	–	Vu = 115 k
RSB-75	24	30	–	–	7#9	3#6	3#8	300 ft-k	–		–	–	Vu = 150 k
RSB-76	24	30	–	–	2#9	1#6	4#7	300 ft-k	–		–	–	Vu = 175 k
RSB-78	24	30	–	–	6#9	2#9	4#7	300 ft-k	–		–	–	Vu = 175 k
RSB-79	24	30	–	–	2#9	2#6	4#7	125 ft-k	–		–	–	Vu = 175 k
RSB-81	24	48	–	–	4#9	2#6	3#6	250 ft-k	–		–	–	Vu = 95 k
RSB-82	24	48	–	–	4#9	2#6	3#6	250 ft-k	–		–	–	Vu = 95 k
RB-97	24	48	6#9	–	6#9	–	–	6#9	–	5#5	2C	#4	10"
RB-98	24	48	–	–	6#9	–	–	6#9	–	5#5	2C	#4	10"
RB-99	24	48	–	–	6#9	–	–	6#9	–	5#5	2C	#4	10"
3SB-86	24	30	–	–	7#9	2#6	3#8	250 ft-k	–	2#5	–	–	Vu = 135 k; Tu =70ft-k
3SB-87	24	30	–	–	5#9	2#6	3#8	125 ft-k	–	–	–	–	Vu = 150 k
3SB-89	24	30	–	–	9#10	4#7	4#9	400 ft-k	–	–	–	–	Vu = 230 k
3SB-90	24	30	–	–	2#9	1#7	4#9	400 ft-k	–	–	–	–	Vu = 290 k
3SB-92	24	30	–	–	10#9	3#9	4#9	400 ft-k	–	–	–	–	Vu = 250 k
3SB-93	24	30	–	–	2#9	1#6	4#9	400 ft-k	–	–	–	–	Vu = 290 k
3SB-95	24	30	–	–	6#9	2#6	3#8	250 ft-k	–	2#5	–	–	Vu = 150 k; Tu=70 ft-k
3SB-96	24	30	–	–	4#9	2#6	3#8	250 ft-k	–	–	–	–	Vu = 150 k
3B-101	24	30	6#9	–	6#9	–	–	6#9	–	–	2C	#4	8"
3B-102	24	30	–	–	6#9	–	–	6#9	–	–	2C	#4	8"
3B-103	24	30	–	–	6#9	–	–	6#9	–	–	2C	#4	8"

Figure 7. Excerpts from beam schedules (Sheet HQ-S4-1-01, dated November 15, 2023, and Sheet HQ-S4-1-05 dated July 28, 2023) showing beam sizes, reinforcement, and other design information for beams north of grid line 3 at third floor and roof. See Figure 2 and Figure 4 for beam designations at roof and third floor, respectively.



(a)



(b)

Figure 8. Soffit beam reinforcement details from Detail 1 of Sheet HQ-S4-1-10, dated June 10, 2022.

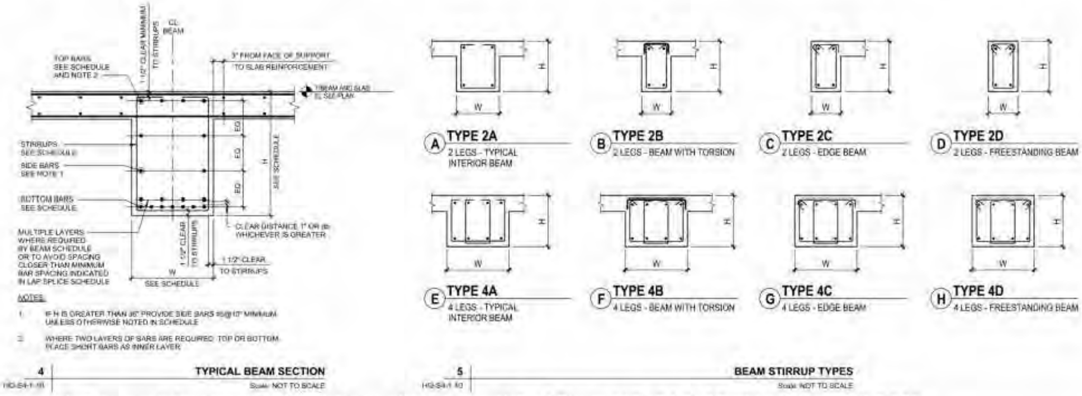


Figure 9. Beam reinforcement details from details 4 and 5 of Sheet HQ-S4-1-10, dated June 10, 2022.

SOFFIT BEAM SCHEDULE																	2993 FT. LAUDERDALE Police Hdqtrs	Floor: Roof A, 3rd Floor A
MARK NUMBER	type		LENGTH		QUANT	STRANDS DESIGN	ADDITIONAL REBAR	SHEAR STEEL	STIRRUP TYPE	PULL %	REL PSI	CROWN	Mu F-K	Vu K	Tu K	special conditions		
	width	depth	FT	IN														
PL M-472	24	48	23	2	1	7@3"	3#7X20'@2",3#7X20' T.O.S.	#4(12@6",BAL 12")	3	60%	3500	1/2"	250	95				
PL M-473	24	48	7	6	1	7@3"	3#7X20'@2",3#7X20' T.O.S.	#4@6"	3	60%	3500		125	115				
M-475	24	30	23	2	1	7@3"	2#6X20'@2"	#4(9@4",6@6",4@10",BAL 12")	3	60%	3500	1/2"	300	150				
M-476	24	30	7	6	1	7@3"	2#6X7'@2"	#4(9@4",BAL 6")	3	60%	3500		300	175				
M-478	24	30	23	2	1	7@3"	2#6X20'@2"	#4(9@4",6@6",4@10",BAL 12")	3	60%	3500	1/2"	300	175				
M-479	24	30	7	6	1	7@3"	2#6X7'@2"	#4(9@4",BAL 6")	3	60%	3500		125	175				
PL M-481	24	48	23	2	1	7@3"	3#7X20'@2",3#7X20' T.O.S.	#4(11@6",BAL 12")	5	60%	3500	1/2"	250	95	0			
PL M-482	24	48	7	6	1	7@3"	3#7X7.5'@2",3#7X7.5' T.O.S.	#4@6"	5	60%	3500		250	95	0			
M-386	24	30	23	2	1	7@3	2#6X15'@2"	#4(8@5",3@8",BAL 12")	5	90%	3500	1/2"	250	135	70			
M-387	24	30	7	6	1	7@3	2#6X7'@2"	#4@5"	5	90%	3500		125	150				
M-389	24	30	22	6	1	7@3	2#6X15'@2"	#5(14@4",3@12",BAL#4@12")	3	60%	3500	1/2"	400	230				
M-390	24	30	7	6	1	7@3	2#6X7'@2"	#5@2.5"	3	60%	3500		400	290				
M-392	24	30	22	6	1	7@3	2#6X15'@2"	#5(18@3",3@9",BAL#4@12")	3	90%	3500	1/2"	400	250				
M-393	24	30	7	6	1	7@3	2#6X7'@2"	#5@2.5"	3	60%	3500		400	290				
M-395	24	30	22	6	1	7@3	2#6X15'@2"	#4(8@5",4@8",BAL 12")	5	90%	3500	1/2"	250	150	70			
M-396	24	30	7	6	1	7@3	2#6X7'@2"	#4@5"	5	90%	3500		250	150				

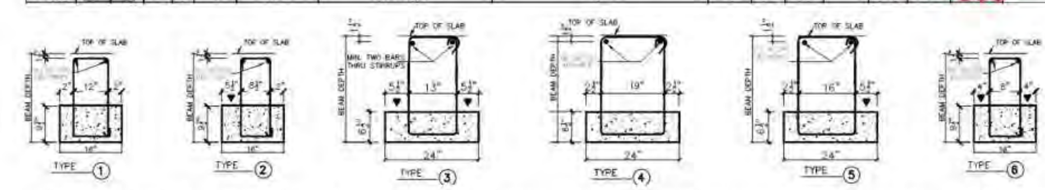


Figure 10. Excerpts from precast soffit beam schedules (Sheet SC-3A, dated October 25, 2023, and Sheet SC-2A dated July 6, 2023) showing beam sizes, reinforcement, and other design information for beams north of grid line 3 at third floor and roof. See Figure 3 and Figure 5 for beam designations at roof and third floor, respectively.

GRID LINE / FLOOR	G 2	G 3	H 2	H 3	J 2	J 3	K 2	K 3
EL 48'-0"								
EL 33'-0"	16x16 4#10	24x24 8#11	16x16 4#10	24x24 8#11	16x16 4#10	24x24 8#11	16x16 4#10	24x24 8#11
EL 18'-0"	24" DIA 8#8	24" DIA 8#11	24" DIA 8#8	24x24 8#11	24" DIA 8#8	24x24 8#11	24" DIA 8#8	24x24 8#8
EL 0'-0"	16x16 4#10	24x24 8#11	16x16 4#10	24x24 8#11	16x16 4#10	24x24 8#11	16x16 4#10	24x24 8#11
EL -2'-0"								
EL -3'-0"								
REMARKS								

Figure 11. Excerpts from column schedule (Sheet HQ-S4-2-01, dated May 24, 2024) showing column sizes and reinforcement for columns at and north of grid line 3.

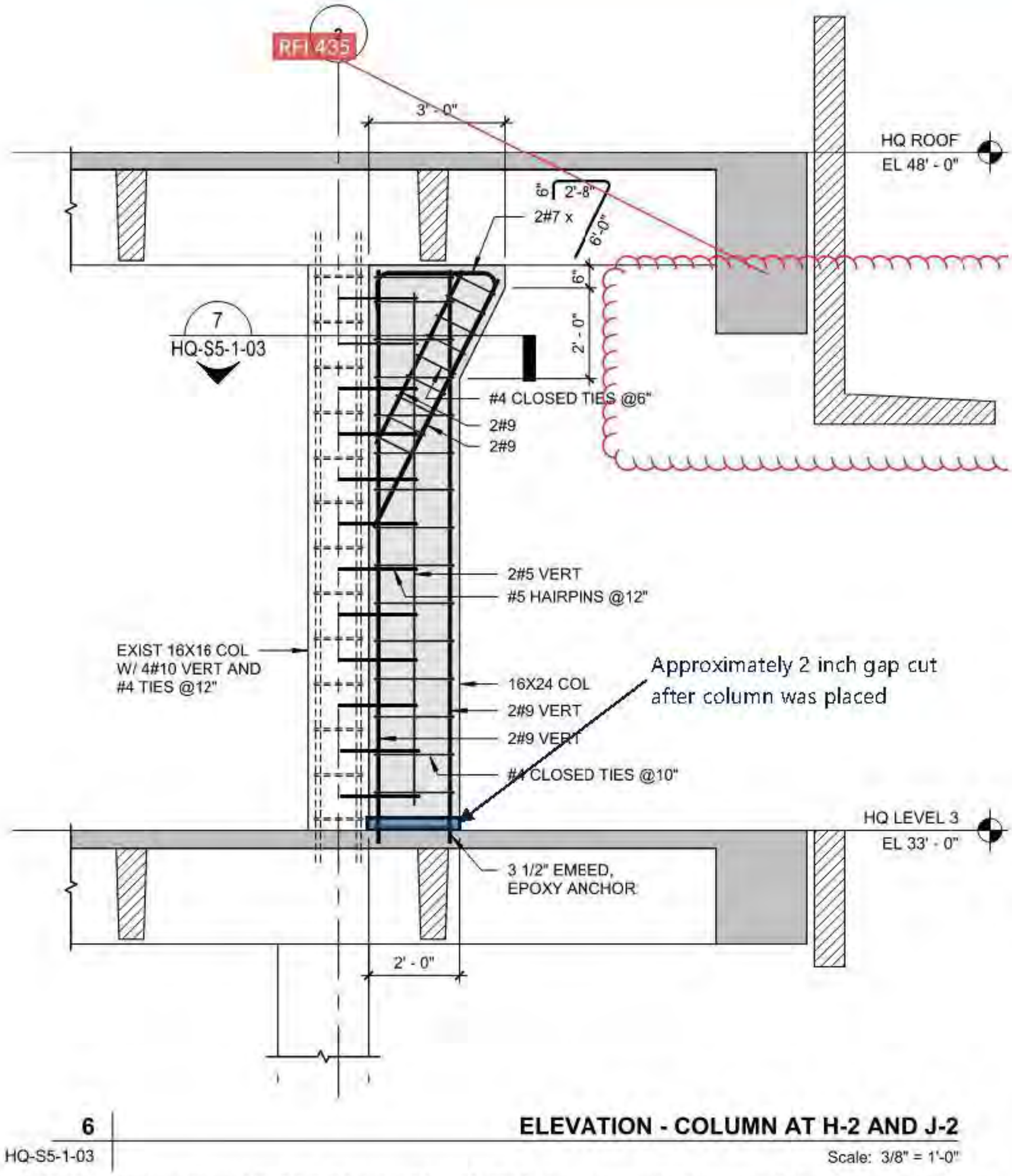


Figure 12. Detail 6 from Sheet HQ-S5-1-03 dated May 24, 2024, showing rectifications details for third story columns at gridlines H-2 and J-2. Annotations in blue by WJE. Note that later the rectifications were modified so an approximately 2 inch gap was cut between the bottom of the column enlargement and the third floor slab.

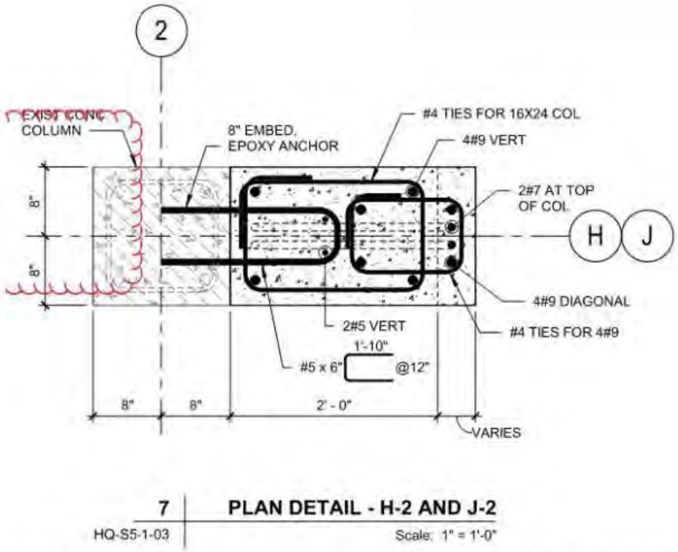


Figure 13. Detail 6 from Sheet HQ-S5-1-03 dated May 24, 2024, showing section through column enlargement rectification for third story columns at gridlines H-2 and J-2.

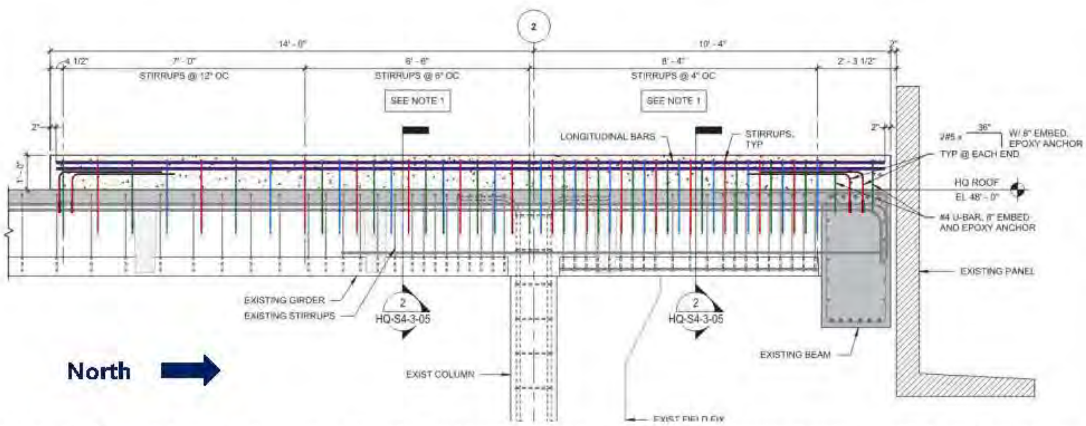


Figure 14. Section from "Rebar Installation Color Sheet.pdf" showing upturned beam at roof. Annotations in blue by WJE.

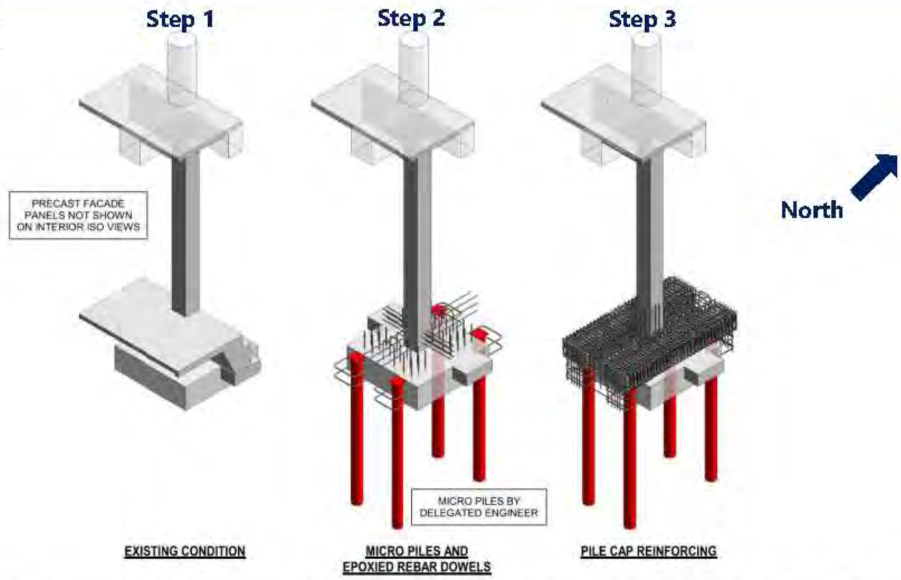


Figure 15. Installation sequence of new micropile and first story column jacket at gridline 2 from detail 2 of Sheet HQ-S6-1-04 dated September 12, 2024. Annotations in blue by WJE. See Figure 16 for subsequent steps.

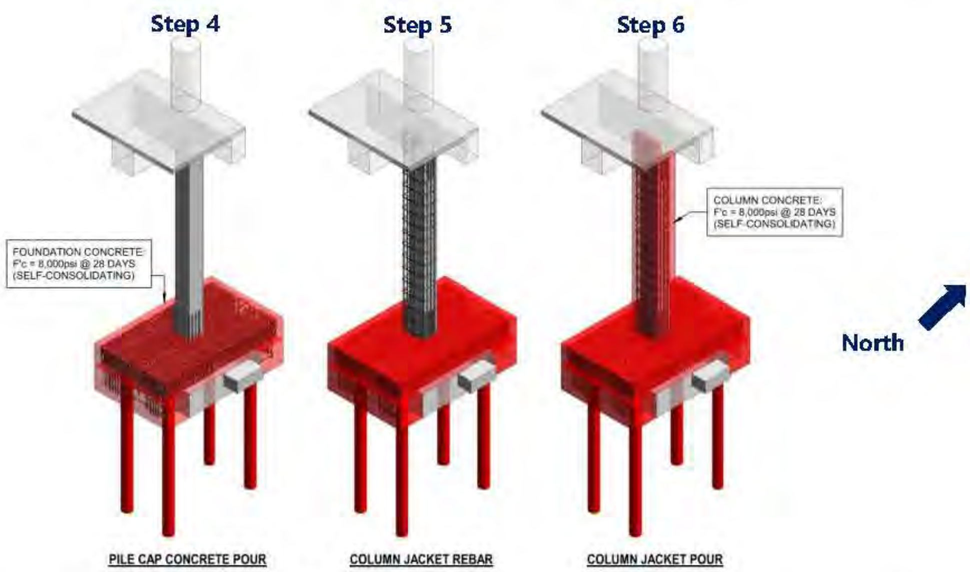


Figure 16. Installation sequence of new micropile and first story column jacket at gridline 2 from detail 2 of Sheet HQ-S6-1-04 dated September 12, 2024. Annotations in blue by WJE. See Figure 15 for preceding steps.

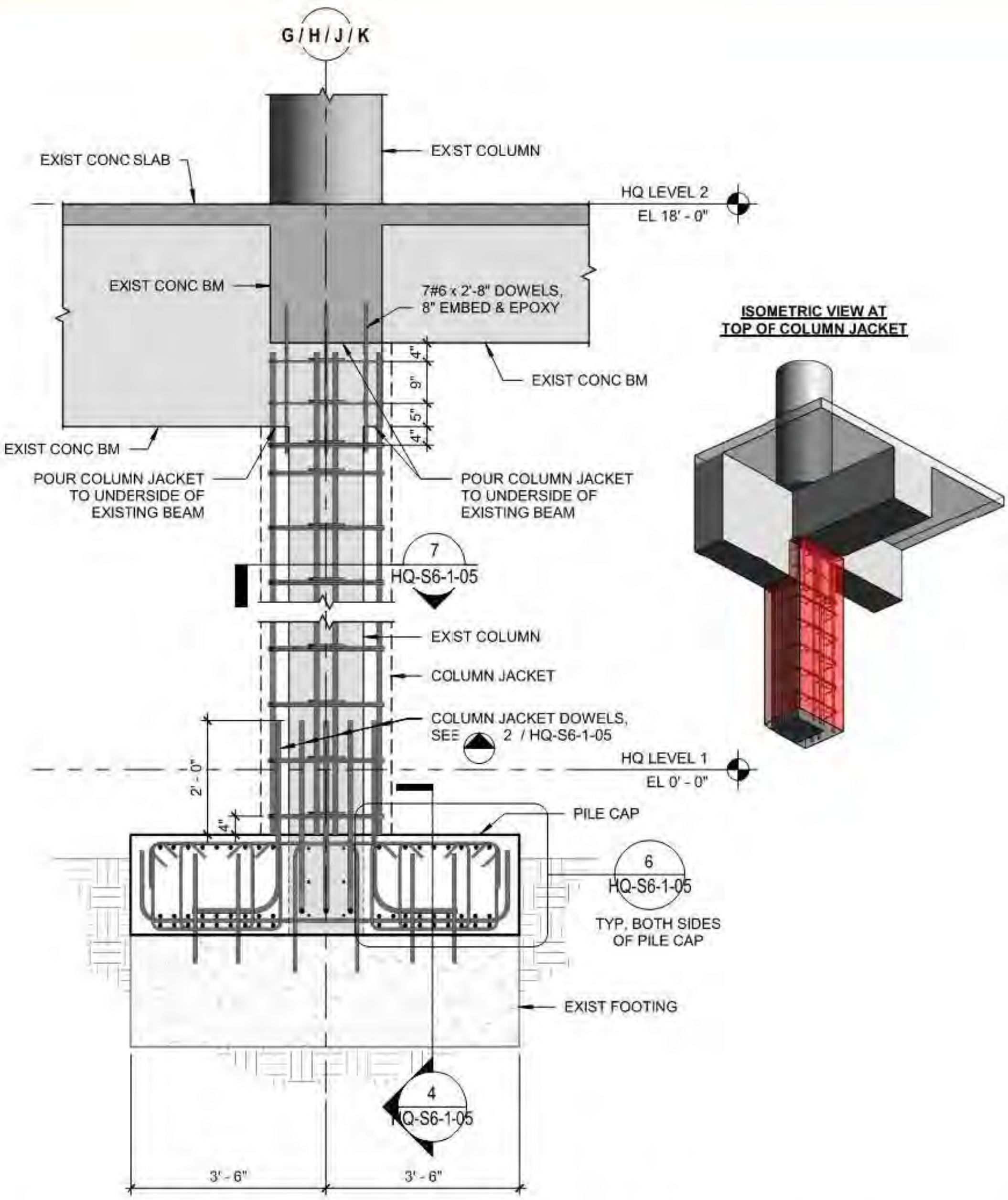


Figure 17. Section and isometric view from detail 3 of Sheet HQ-S6-1-05 (dated September 12, 2024) showing new column jacket and pile cap at first story columns at gridline 2. Section view is facing north.

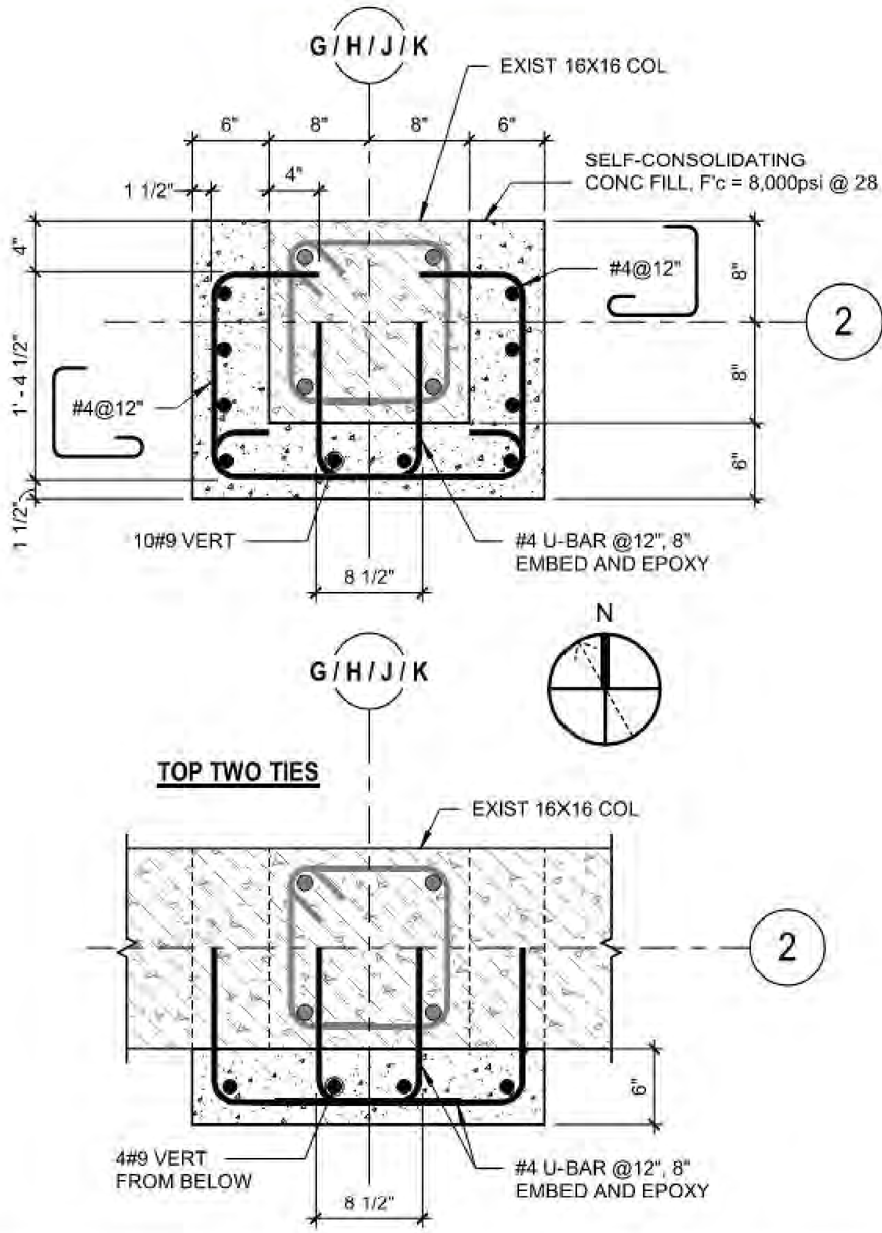


Figure 18. Detail 7 of Sheet HQ-S6-1-05 (dated September 12, 2024) showing column reinforcement for new column jacket at first story columns at gridline 2. Upper detail is for all but the top of the column while the lower detail is for the portion of the column that extends up past the east-west beams to the soffit of the north-south second floor beam.

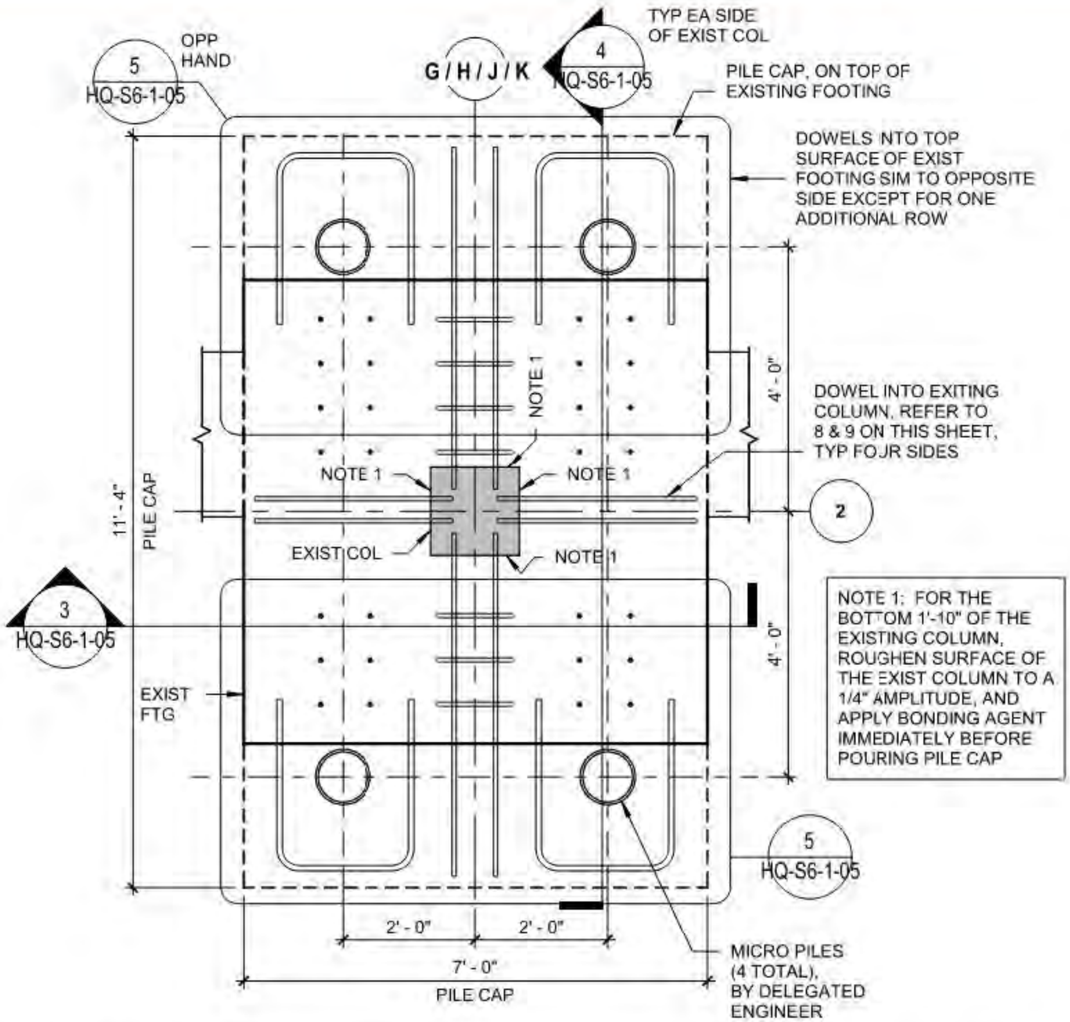


Figure 19. Detail 1 of Sheet HQ-S6-1-05 (dated September 12, 2024) showing partial plan view of new pile cap at columns at gridline 2.

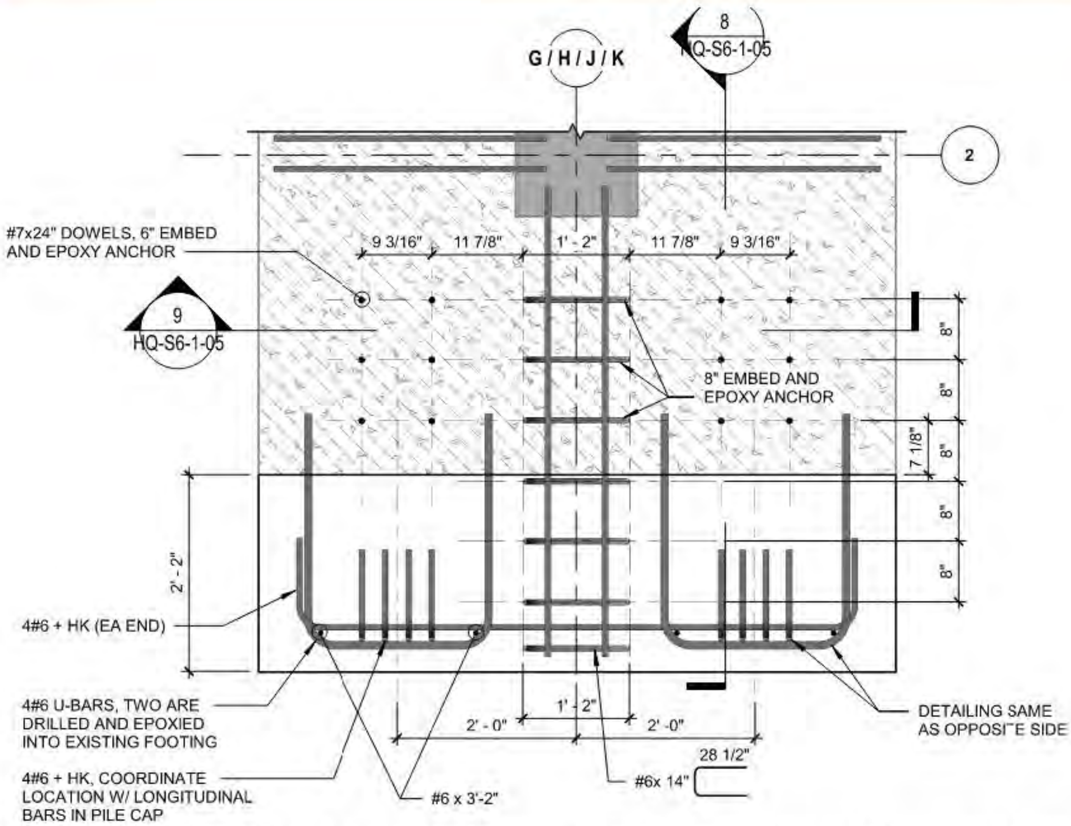


Figure 20. Detail 5 of Sheet HQ-S6-1-05 (dated September 12, 2024) showing partial plan detail view of new pile cap at columns at gridline 2.

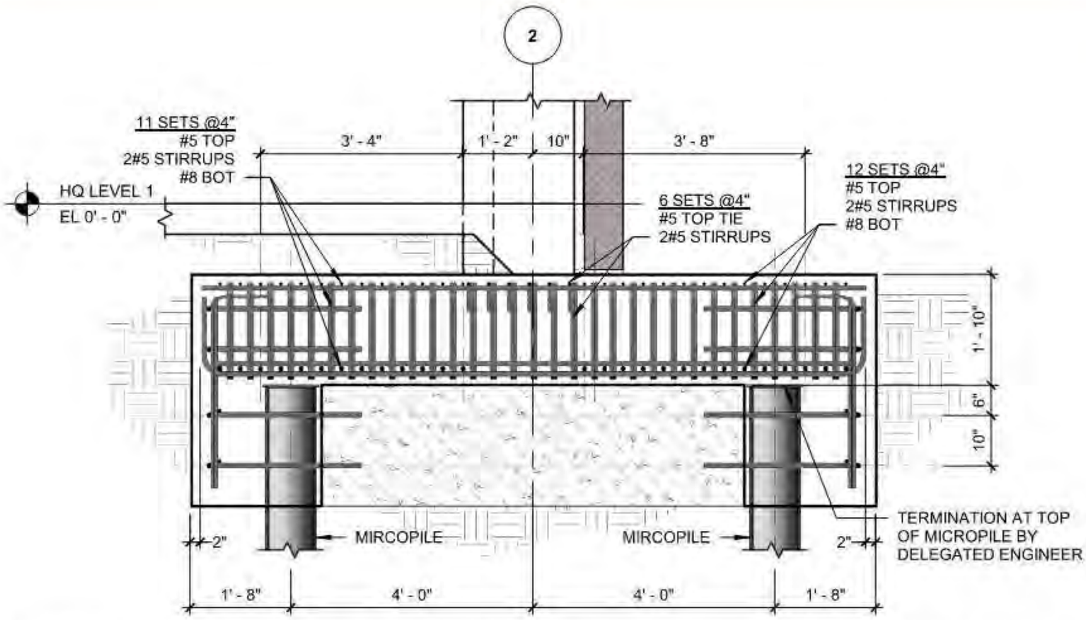


Figure 21. Detail 4 of Sheet HQ-S6-1-05 (dated September 12, 2024) showing section view of pile cap and micropiles at gridline 2.

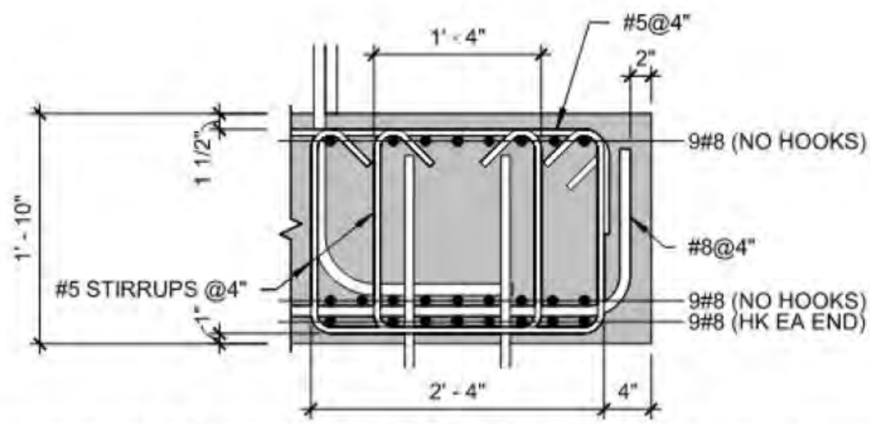


Figure 22. Detail 6 of Sheet HQ-S6-1-05 (dated September 12, 2024) with section view of pile cap showing reinforcement of pile cap.

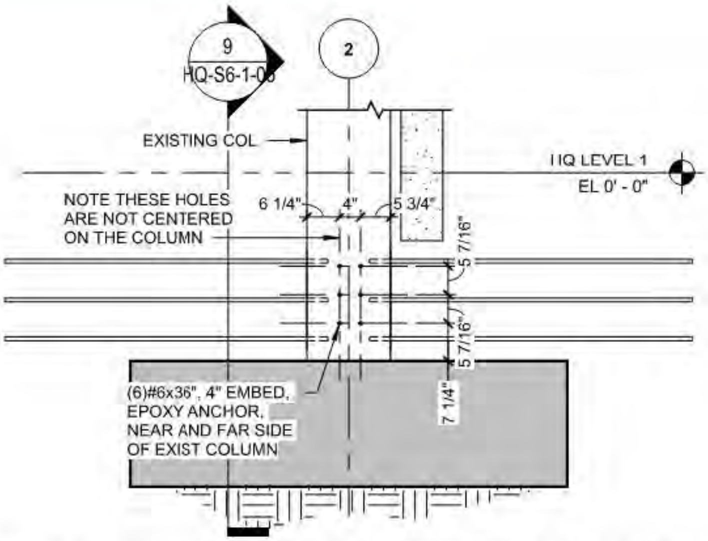


Figure 23. Detail 8 of Sheet HQ-S6-1-05 (dated September 12, 2024) showing section view of dowel reinforcement into existing column.

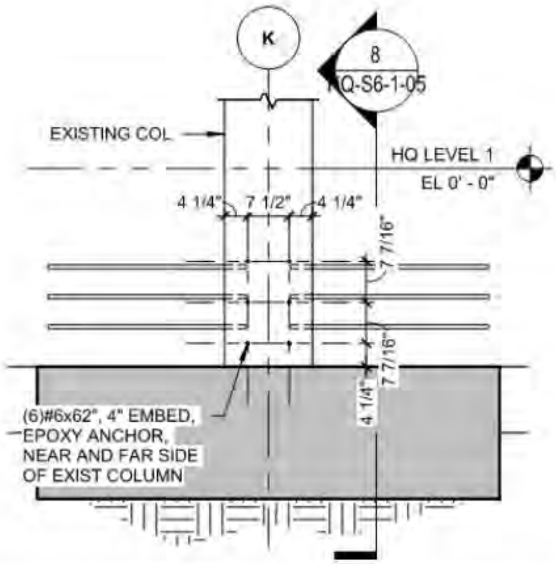


Figure 24. Detail 9 of Sheet HQ-S6-1-05 (dated September 12, 2024) showing section view of dowel reinforcement into existing column.

Figure 25. Photograph from Threshold Inspection Report No. 125 showing conditions on top of roof slab at north end of building.

Figure 26. Photograph from Threshold Inspection Report No. 125 showing conditions on top of roof slab at north end of building.

Figure 27. Photograph from Threshold Inspection Report No. 125 showing conditions on top of roof slab at north end of building.

Beam RSB-79

Figure 28. Photograph from Threshold Inspection Report No. 125 showing conditions at underside of roof slab and at beam RSB-79 (gridline J-2) at north end of building.

Figure 29. Photograph from Threshold Inspection Report No. 127 showing water which was sprayed onto the top of the roof slab at the north end of building.

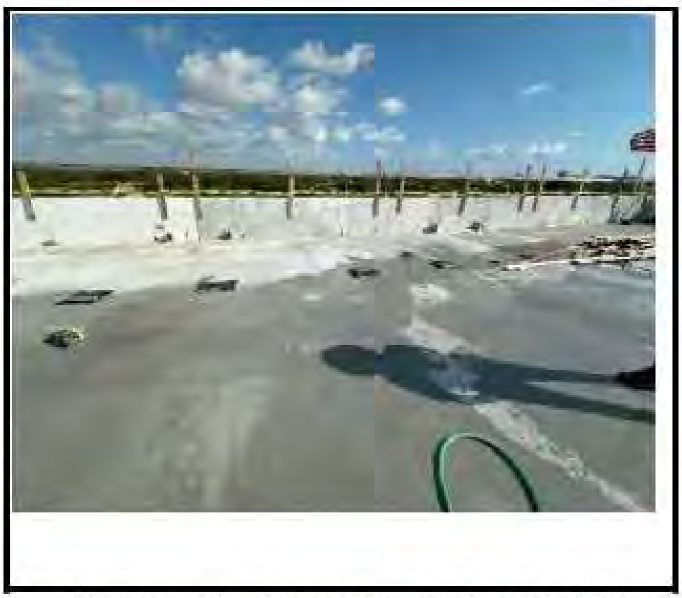


Figure 30. Photograph from Threshold Inspection Report No. 127 showing water which was sprayed onto the top of the roof slab at the north end of building.

Figure 31. Photograph from Threshold Inspection Report No. 127 showing underside of roof slab at north end of the building showing water which has filtered through from the top of the roof slab at cracked slab locations.

Figure 32. Photograph from Threshold Inspection Report No. 127 showing underside of roof slab at north end of the building showing water which has filtered through from the top of the roof slab at cracked slab locations.

Figure 33. Photograph from Threshold Inspection Report No. 127 showing underside of roof slab at north end of the building showing water which has filtered through from the top of the roof slab at cracked slab locations.

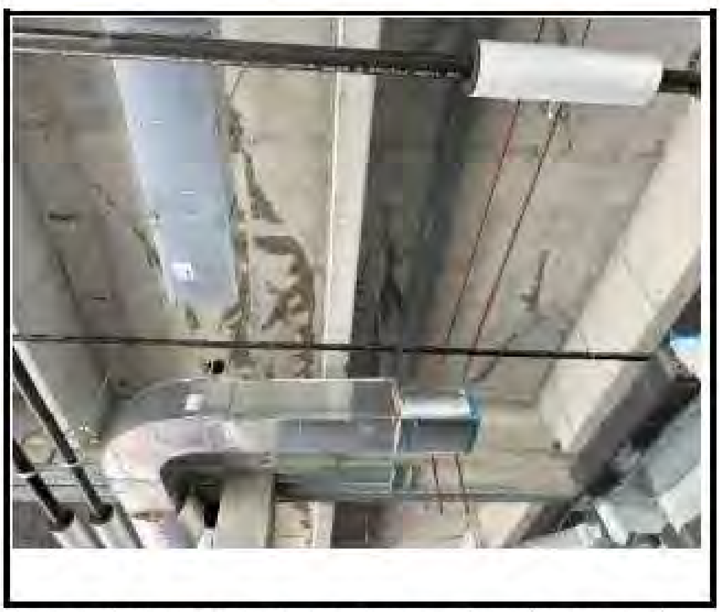


Figure 34. Photograph from Threshold Inspection Report No. 127 showing underside of roof slab at north end of the building showing water which has filtered through from the top of the roof slab at cracked slab locations.

Figure 35. Photograph from Threshold Inspection Report No. 137 showing holes drilled in the north side of column H-2 to receive rebar dowels as part of the rectification of the column and roof structure.

Figure 36. Photograph from Threshold Inspection Report No. 138 showing installed dowels on the north side of column J-2 part of the rectification of the column and roof structure.

Figure 37. Photograph from Threshold Inspection Report No. 138 showing installed dowels on the north side of column J-2 part of the rectification of the column and roof structure.

Figure 38. Photograph from Threshold Inspection Report No. 138 showing installed dowels on the north side of column J-2 part of the rectification of the column and roof structure.

Figure 39. Photograph from Threshold Inspection Report No. 139 showing installed vertical reinforcement and horizontal ties at third story column enlargement.

Figure 40. Photograph from Threshold Inspection Report No. 139 showing work to enlarge the footing at the base of either column H-2 or J-2.

Figure 41. Photograph from Threshold Inspection Report No. 140 showing completed reinforcement installation at third story column enlargement.

Figure 42. Photograph from Threshold Inspection Report No. 140 showing completed reinforcement installation at third story column enlargement.

Figure 43. Photograph from Threshold Inspection Report No. 140 showing continuing work on column footing enlargement.

Figure 44. Photograph from Threshold Inspection Report No. 140 showing continuing work on column footing enlargement.

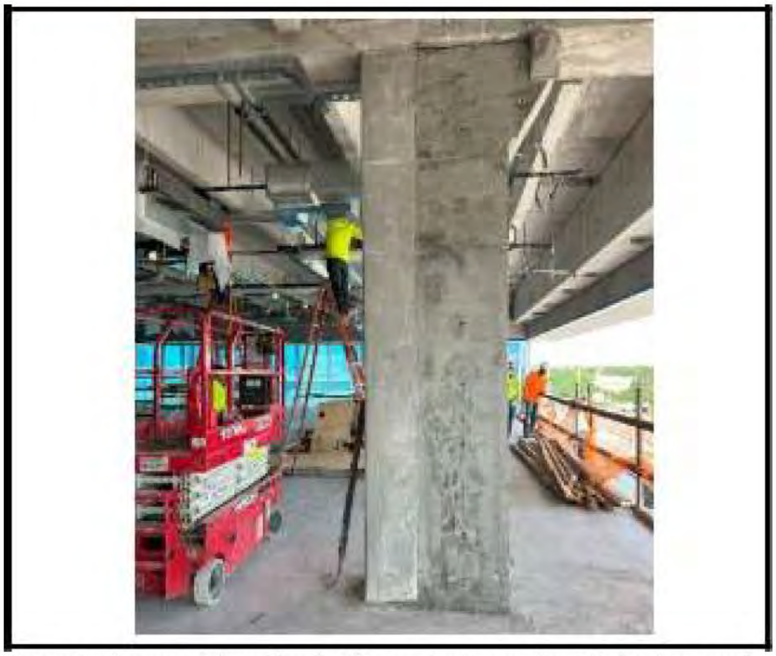


Figure 45. Photograph from Threshold Inspection Report No. 141 showing completed column enlargement.

Figure 46. Photograph from Threshold Inspection Report No. 141 showing completed column enlargement.

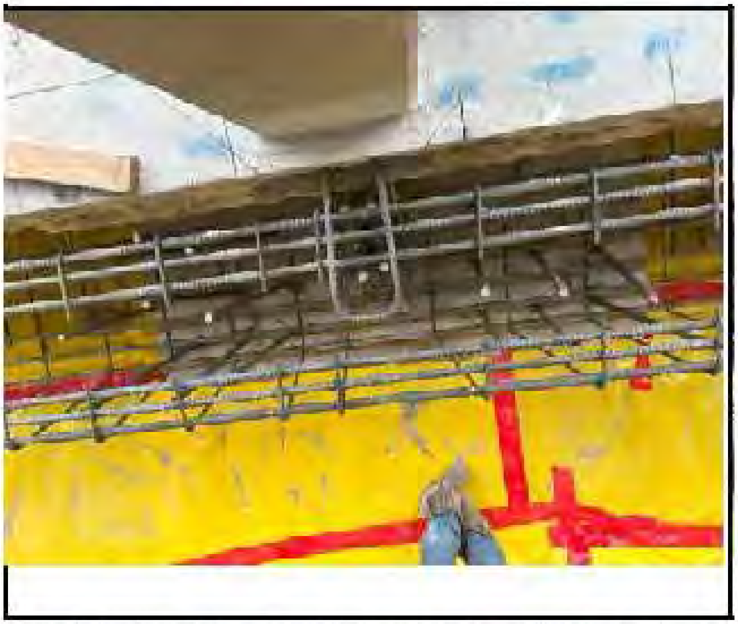


Figure 47. Photograph from Threshold Inspection Report No. 141 showing continuing work on column footing enlargement.

Figure 48. Photograph from Threshold Inspection Report No. 141 showing continuing work on column footing enlargement.

Figure 49. Photograph from Threshold Inspection Report No. 142 showing completed reinforcement placement at column footing enlargement.

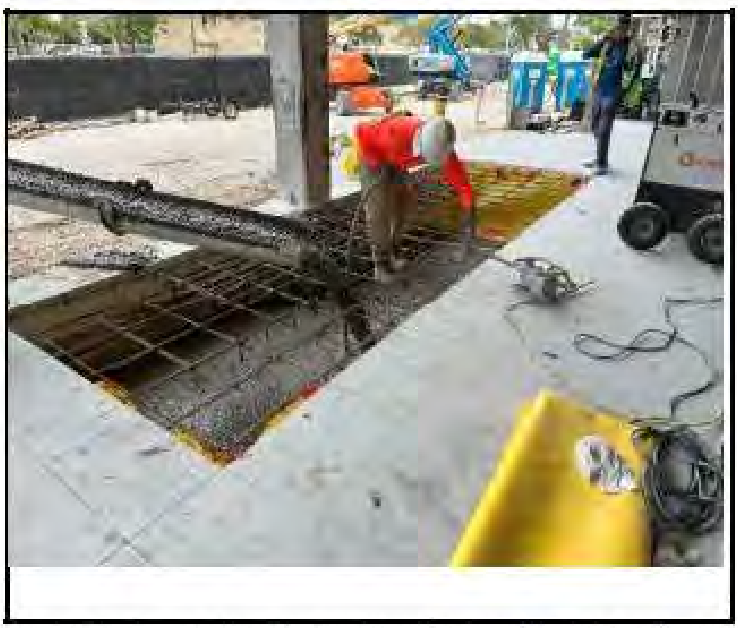


Figure 50. Photograph from Threshold Inspection Report No. 142 showing concrete placement at column footing enlargement.

Figure 51. Photograph from Threshold Inspection Report No. 200 showing drilling of holes for inverted "U" shaped stirrups in upturned beams.

Figure 52. Photograph from Threshold Inspection Report No. 200 showing crack preparation and injection ports for epoxy injection of cracks in roof slab.

Figure 53. Photograph from Threshold Inspection Report No. 201 showing installed inverted "U" shaped stirrups in upturned beams.

Figure 54. Photograph from Threshold Inspection Report No. 201 showing epoxy crack injection of cracks in roof slab.

Figure 55. Photograph from Threshold Inspection Report No. 202 showing installed inverted "U" shaped stirrups in upturned beams.

Figure 56. Photograph from Threshold Inspection Report No. 202 showing installed inverted "U" shaped stirrups and end "L" bars in upturned beams.

Figure 57. Photograph from Threshold Inspection Report No. 207 showing completed installation of reinforcement in in upturned beam.

Figure 58. Photograph from Threshold Inspection Report No. 207 showing completed installation of reinforcement in in upturned beam.

Figure 59. Photograph from ScanTek GPR report at Spot 1 (beams RSB-72 and RSB-73).

Figure 60. Photograph from ScanTek GPR report at Spot 2 (beam RB-97).

Figure 61. Photograph from ScanTek GPR report at Spot 3 (beams RSB-75 and RSB-76).

Figure 62. Photograph from ScanTek GPR report at Spot 4 (precast facade panel at roof).

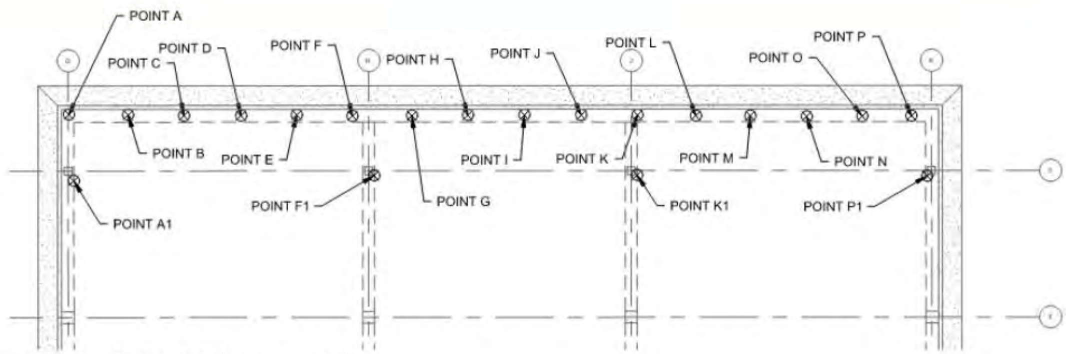


Figure 63. Roof plan showing survey points

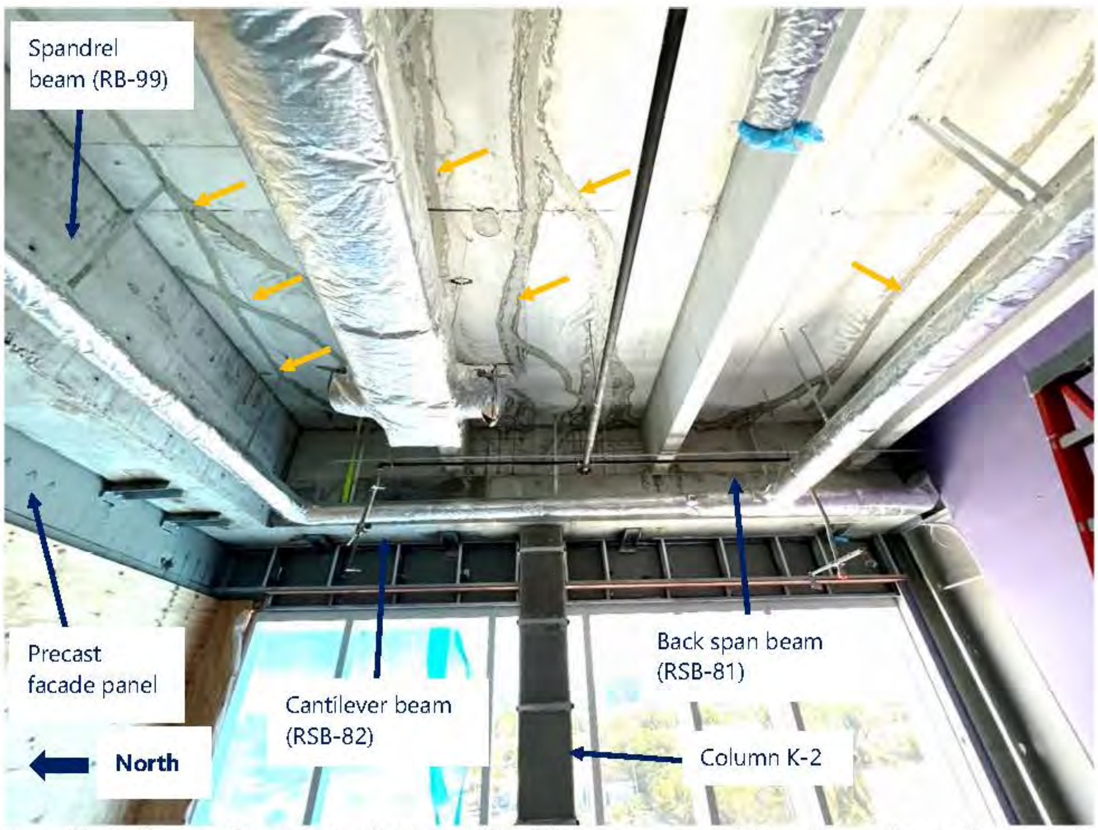


Figure 64. Cantilever roof beam and roof slab at gridline K-2. Orange arrows point to evidence of epoxy filled cracks in roof slab.

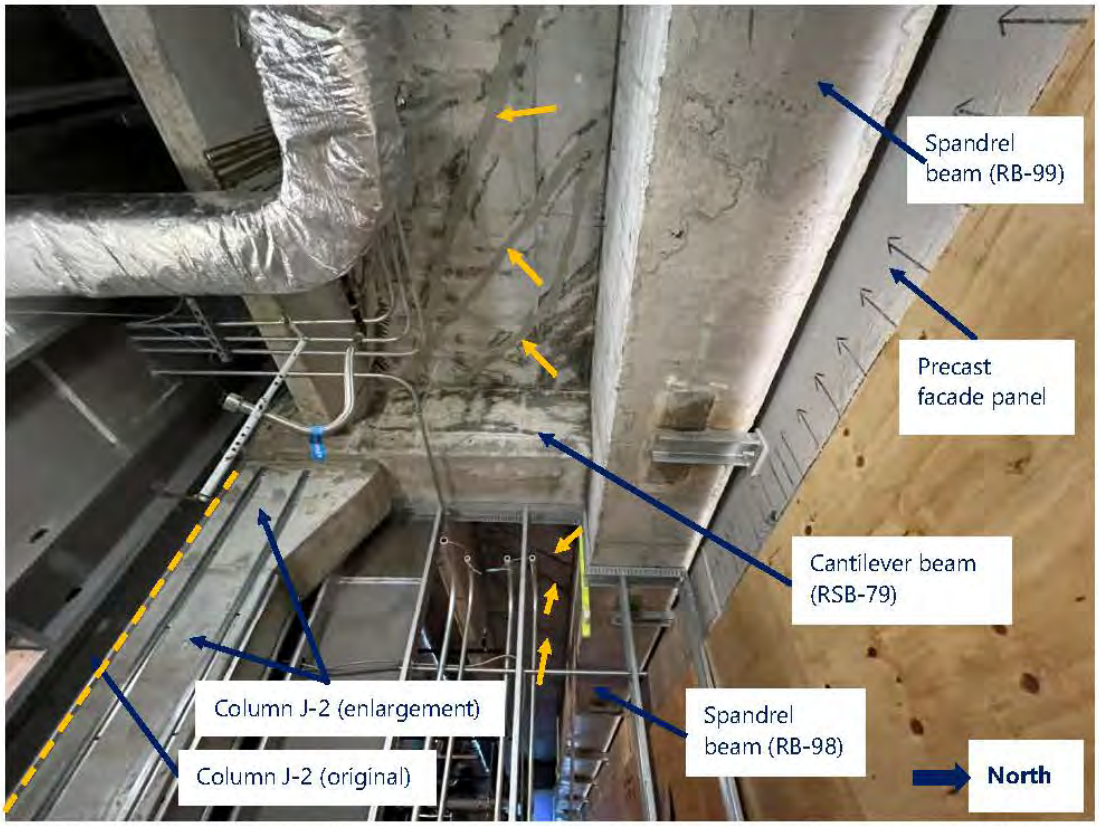


Figure 65. Cantilever roof beam and roof slab at gridline J-2. Orange arrows point to evidence of epoxy filled cracks in roof slab.

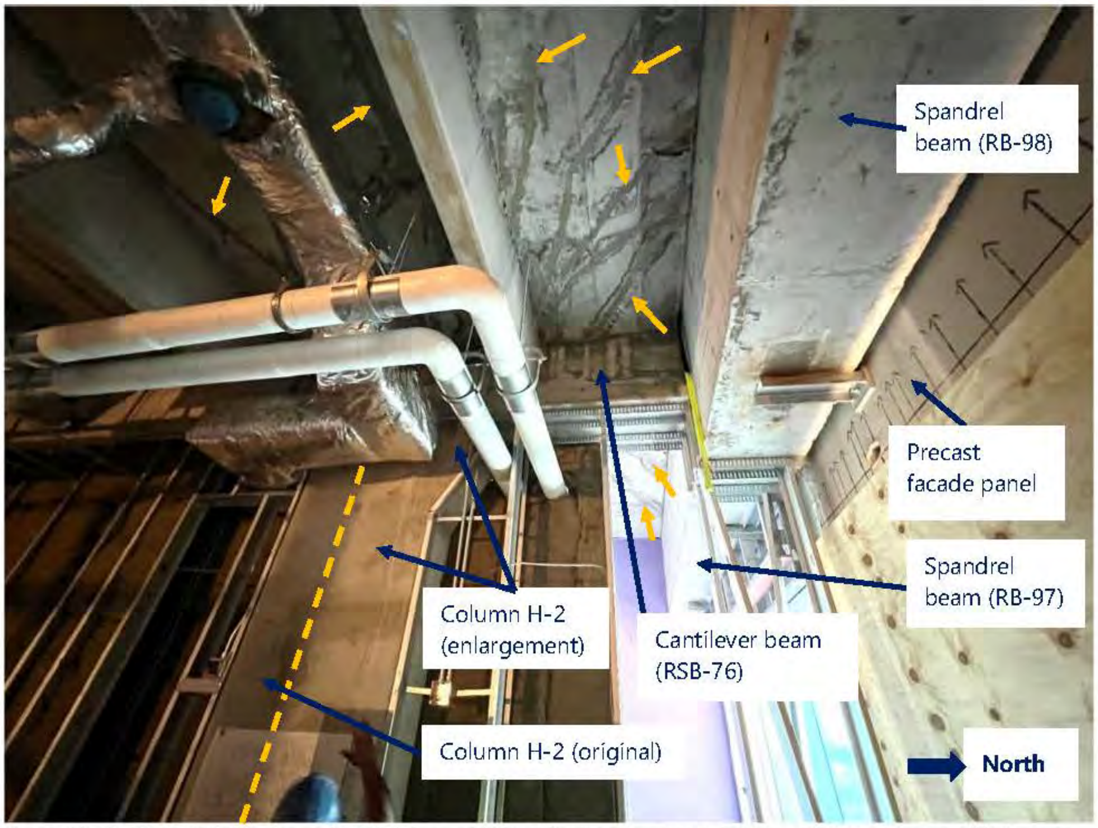


Figure 66. Cantilever roof beam and roof slab at gridline H-2. Orange arrows point to evidence of epoxy filled cracks in roof slab.

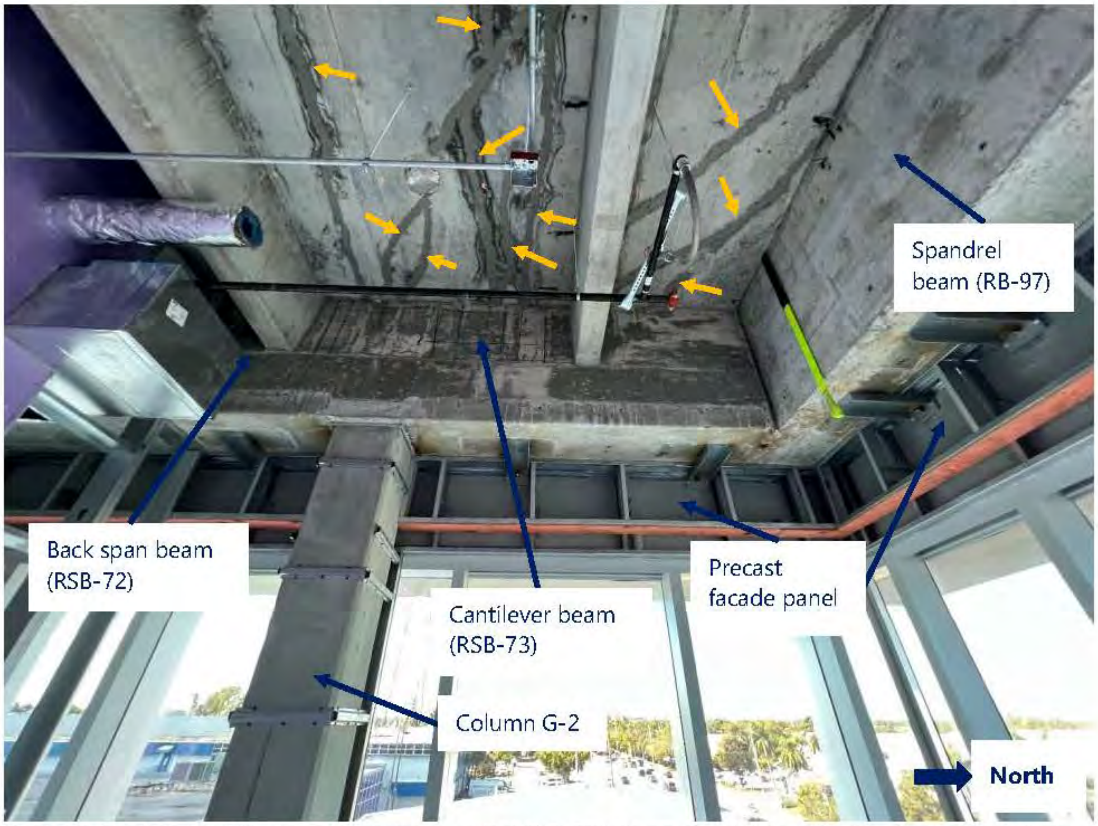


Figure 67. Cantilever roof beam and roof slab at gridline G-2. Orange arrows point to evidence of epoxy filled cracks in roof slab.

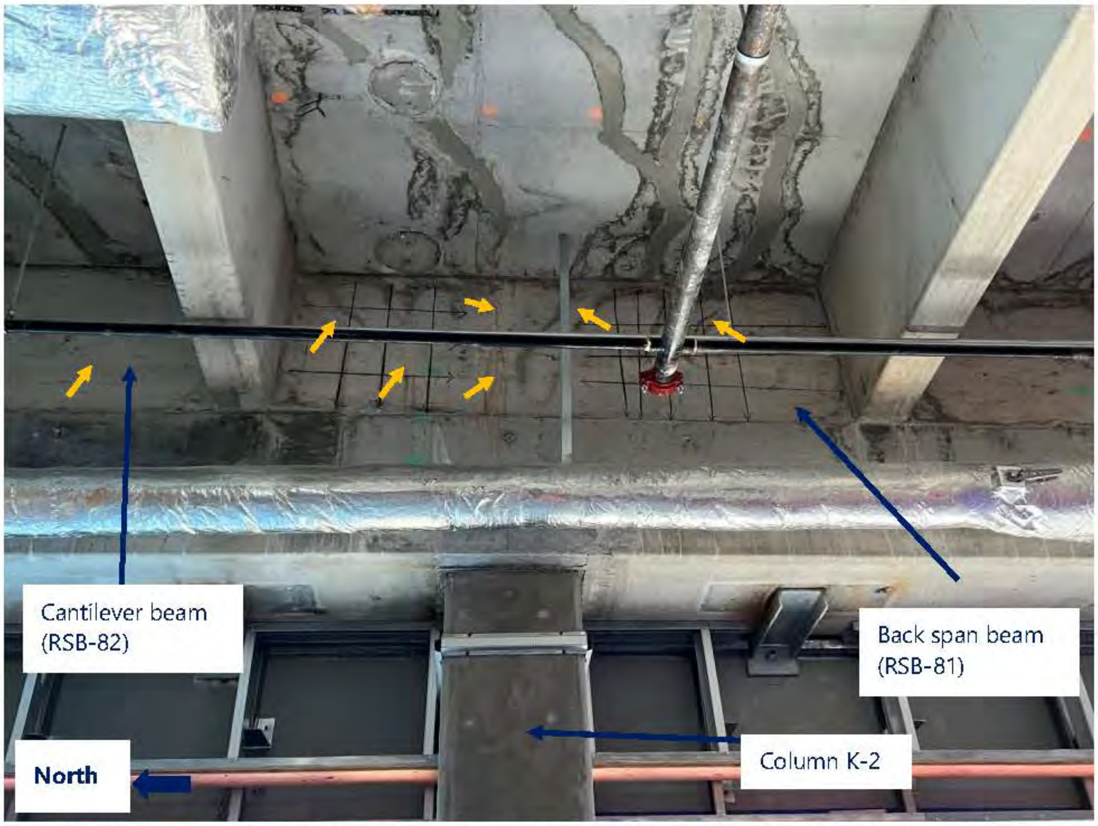


Figure 68. Cantilever roof beam back span roof beam at gridline K-2. Orange arrows point to cracks or evidence of possible cracks in beam.

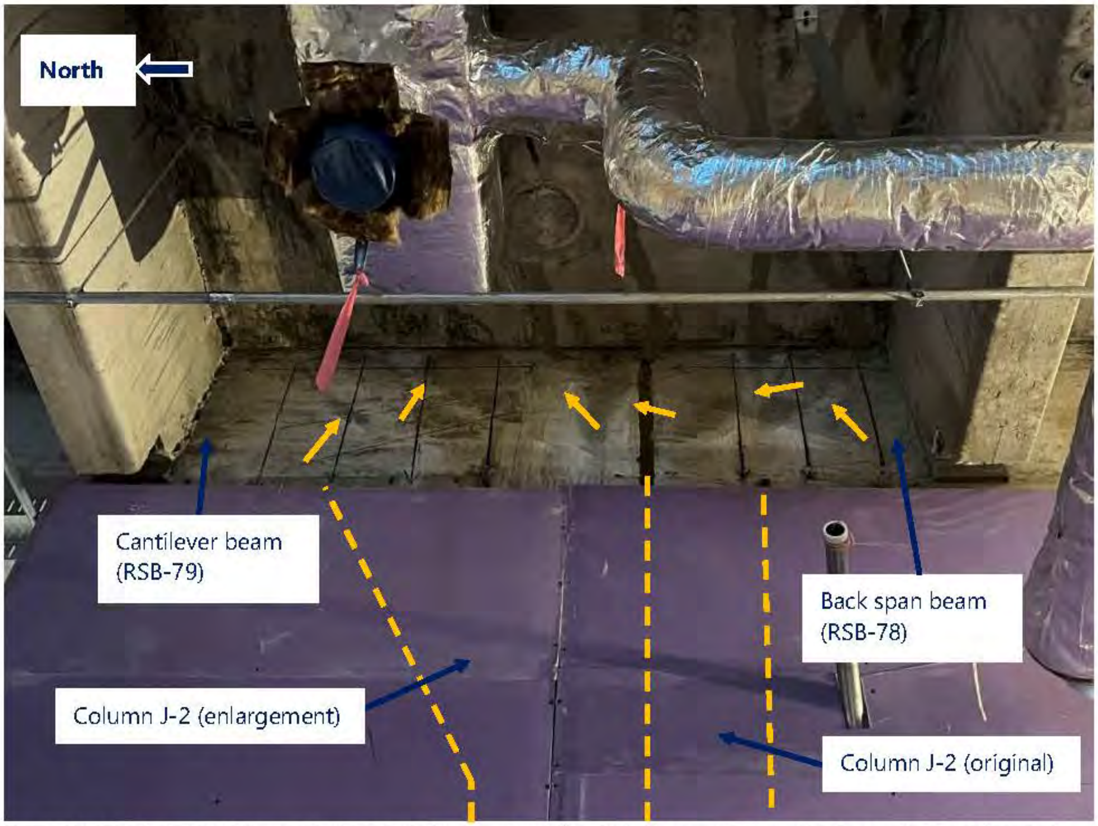


Figure 69. Cantilever roof beam back span roof beam at gridline K-2. Orange arrows point to cracks or evidence of possible cracks in beam. Column location shown is approximate.

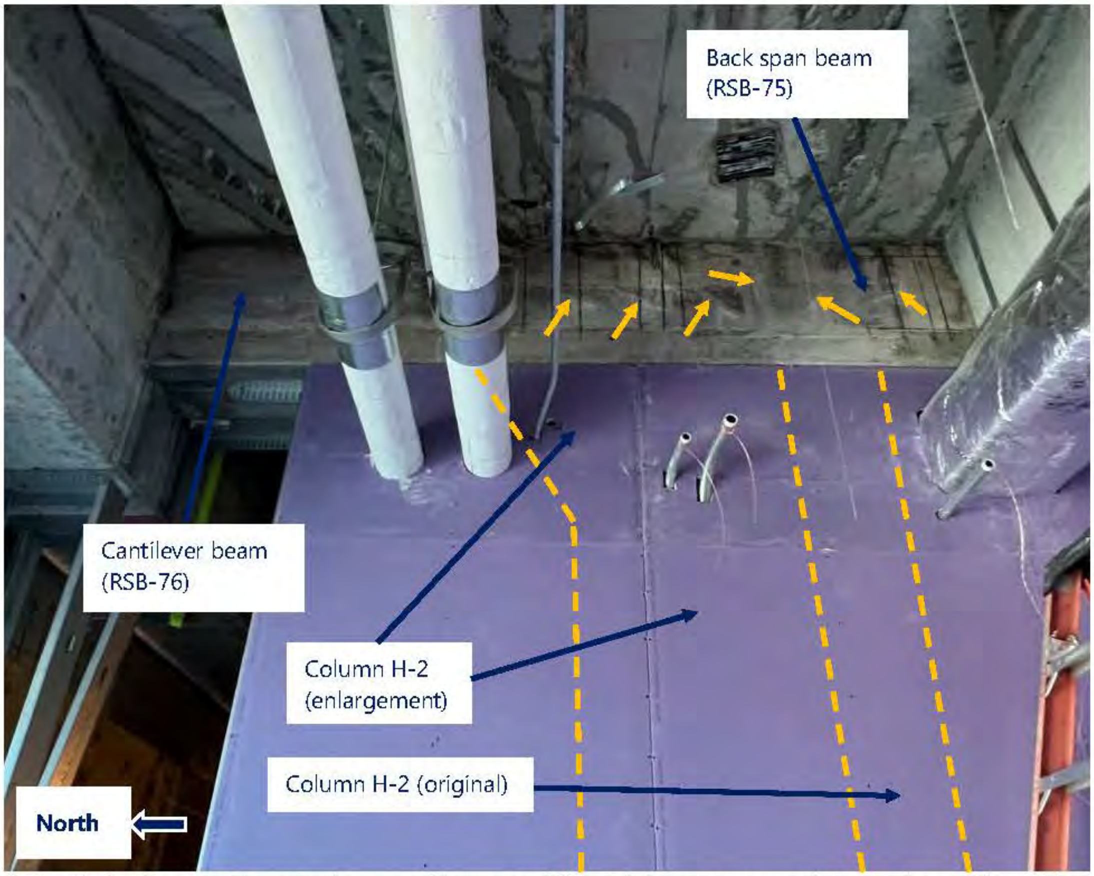


Figure 70. Cantilever roof beam back span roof beam at gridline H-2. Orange arrows point to cracks or evidence of possible cracks in beam. Column location shown is approximate.

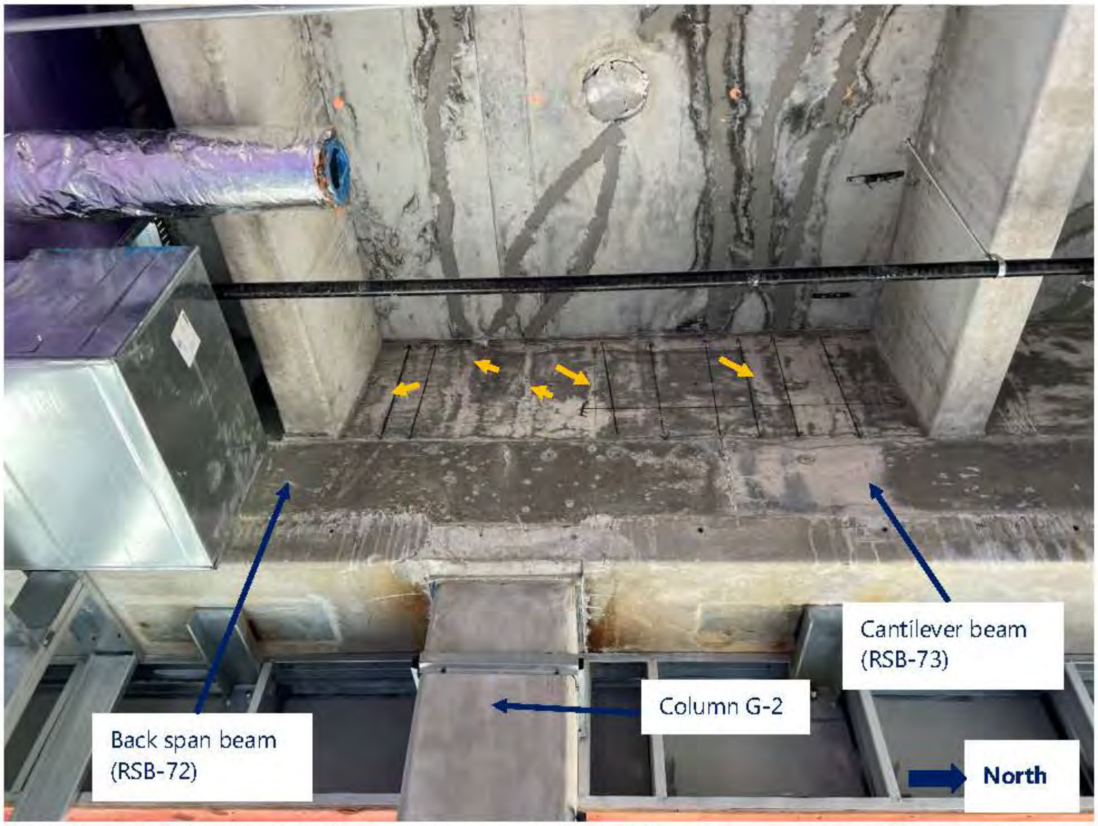


Figure 71. Cantilever roof beam back span roof beam at gridline G-2. Orange arrows point to cracks or evidence of possible cracks in beam.

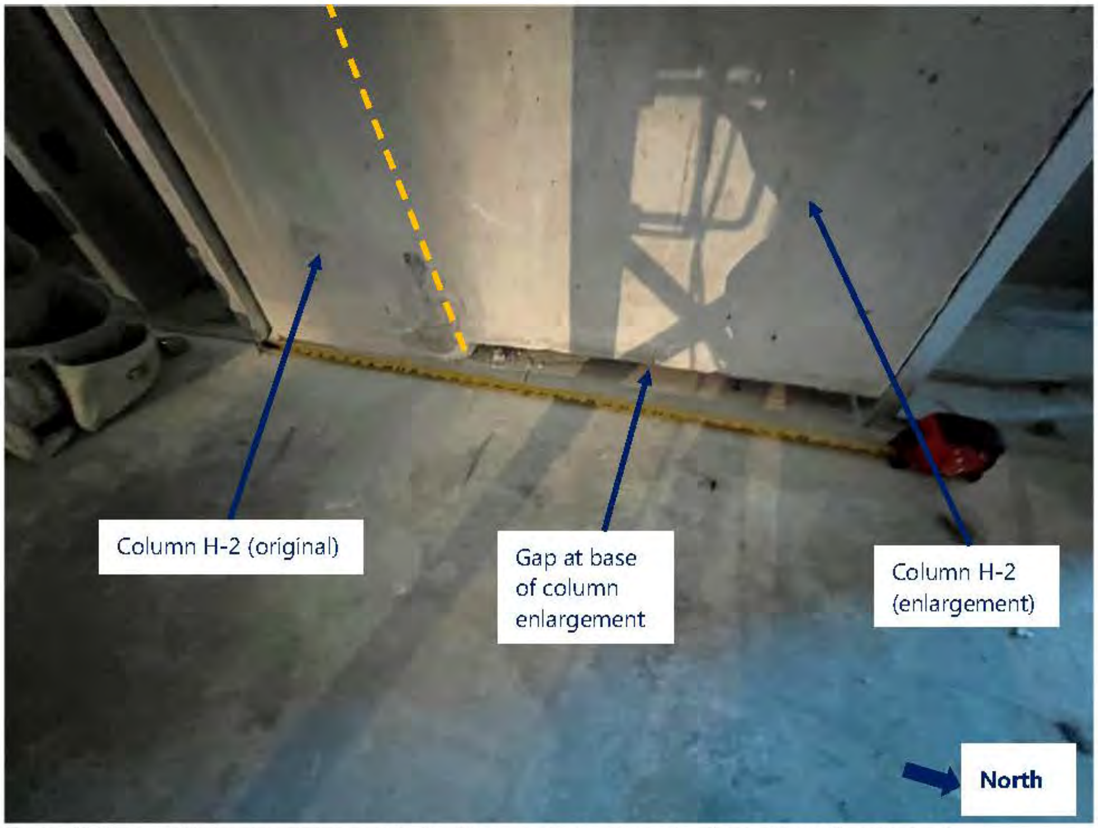


Figure 72. Base of third story column at H-2 showing gap after bottom of column enlargement was cut.

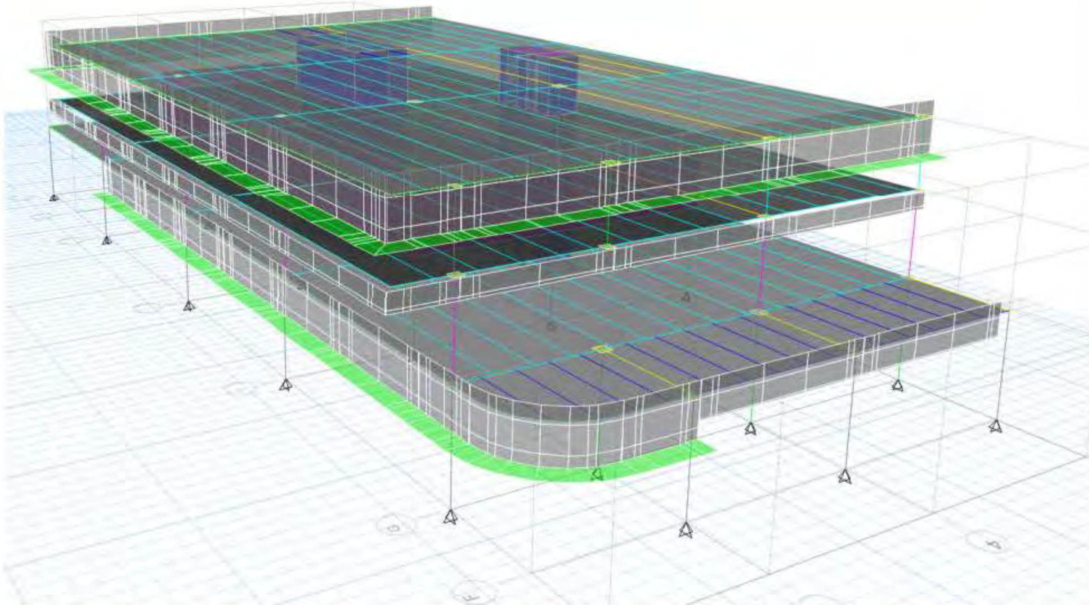


Figure 73. Isometric view of analytical model created in ETABS.

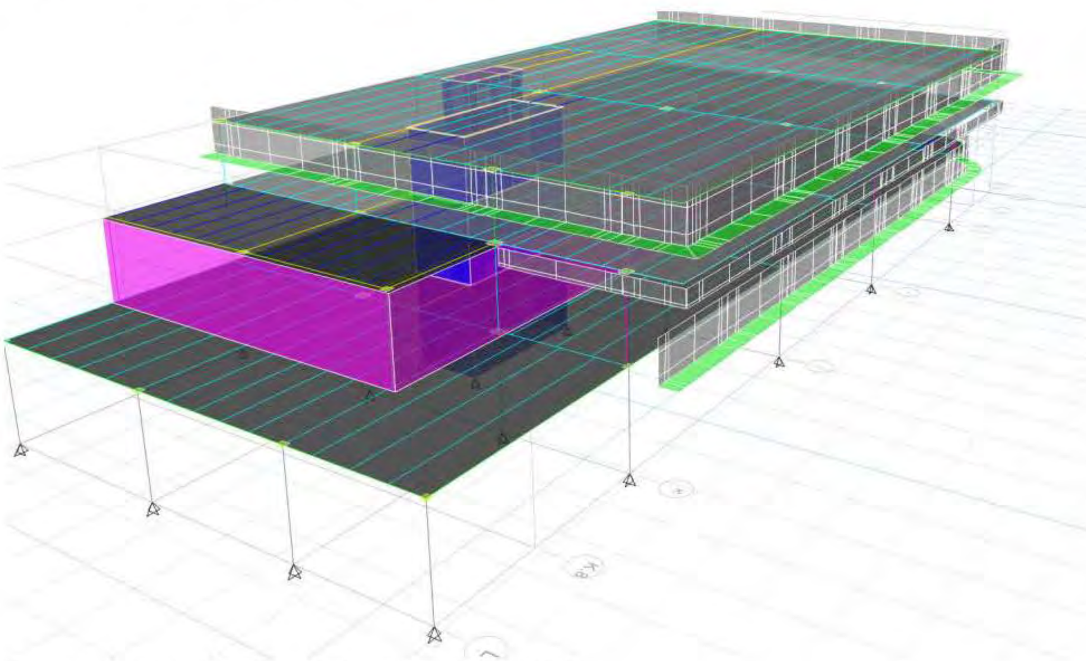


Figure 74. Isometric view of analytical model created in ETABS.

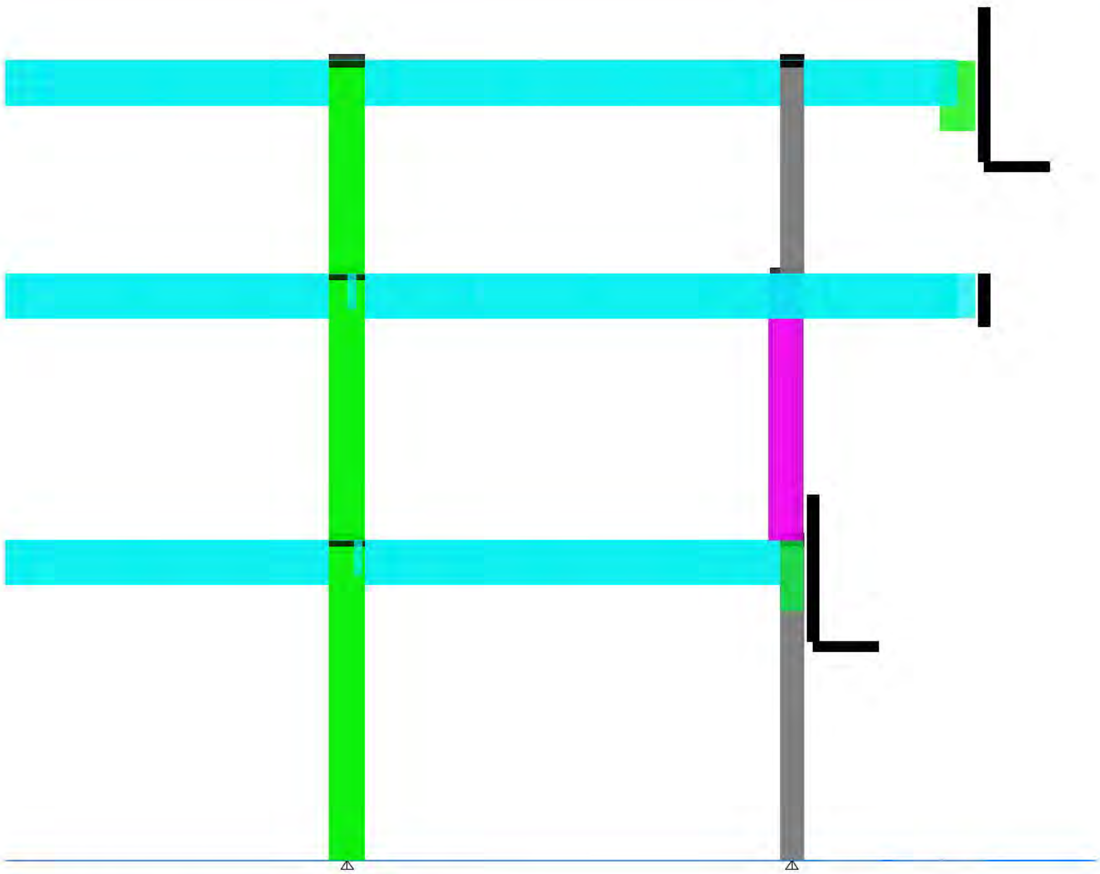


Figure 75. Sectional view at north edge of ETABS model.

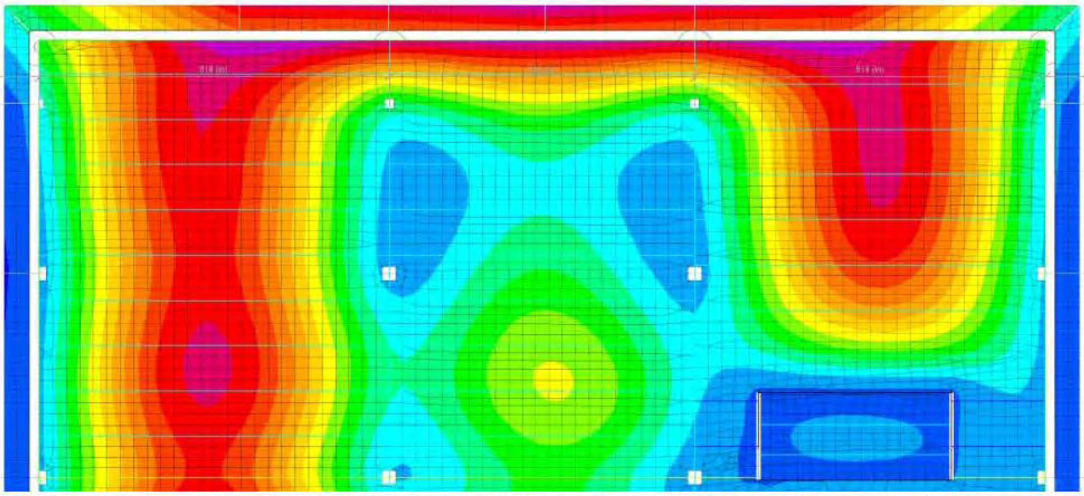


Figure 76. Roof-level deflected shape prediction by ETABS after 10 years of service; as-designed building configuration; purple corresponds with 4 inches of deflection.

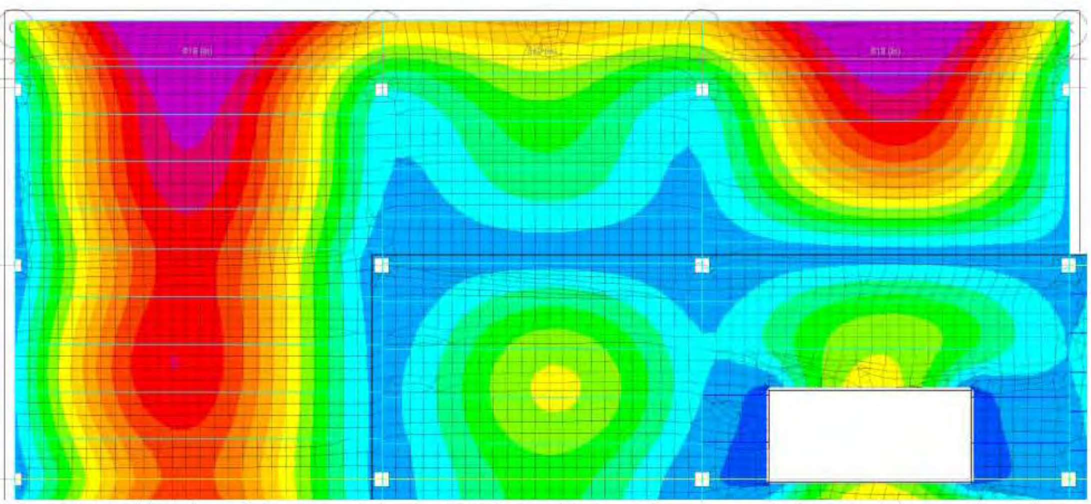


Figure 77. Level 3 deflected shape prediction by ETABS after 10 years of service; as-designed building configuration; purple corresponds with 4 inches of deflection.

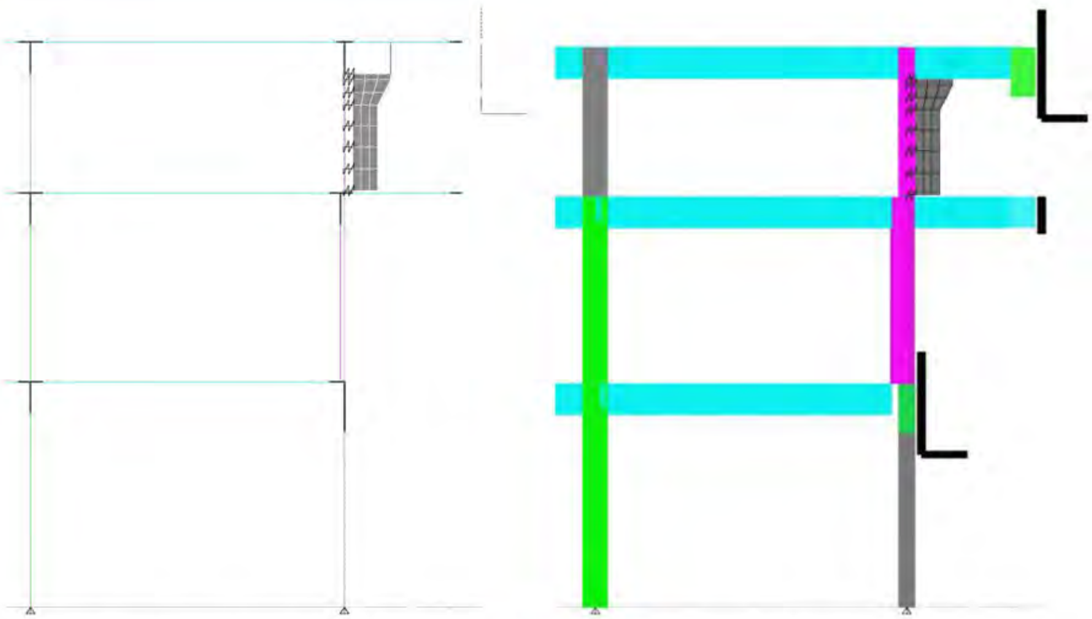


Figure 78. Cross section of analysis model in as-rectified configuration (unextruded left, extruded right).

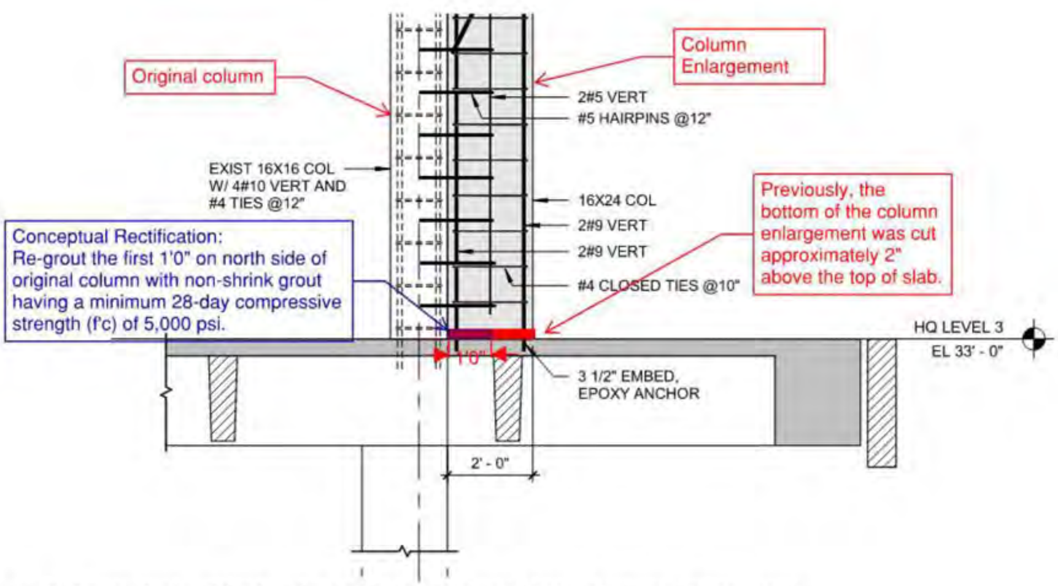


Figure 79. Conceptual rectification for partial re-grouting of base of column enlargement

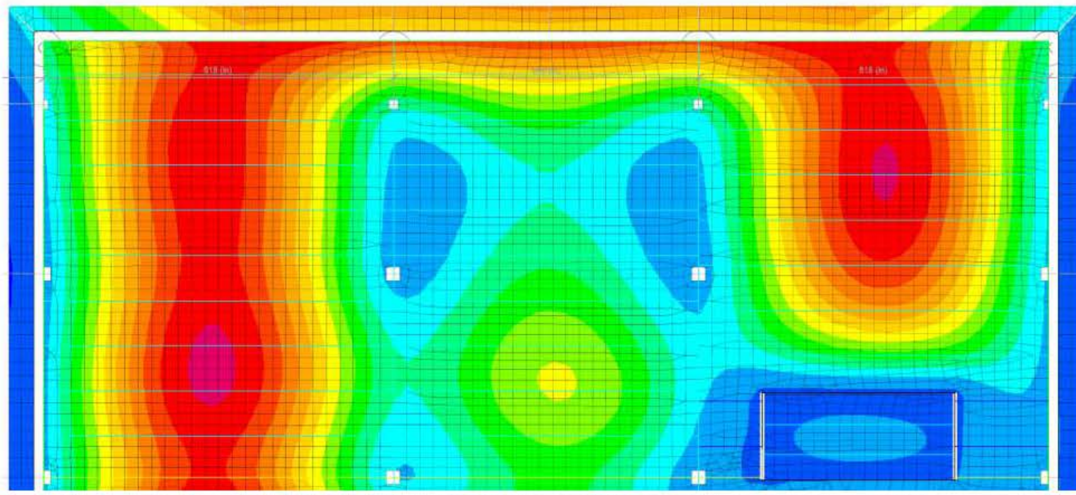


Figure 80. Roof-level deflected shape prediction by ETABS after 10 years of service; rectified building configuration; purple corresponds with 4 inches of deflection.

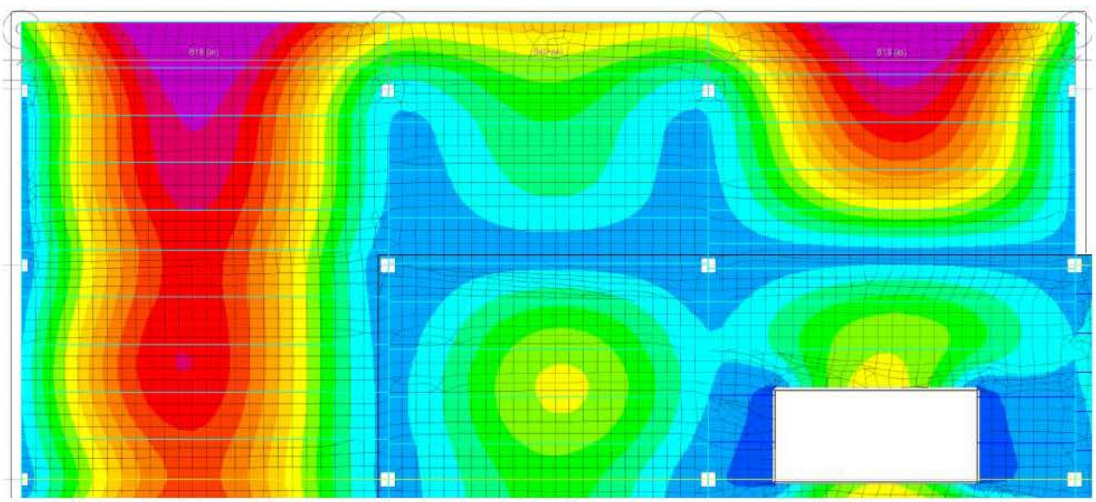


Figure 81. Level 3 deflected shape prediction by ETABS after 10 years of service; rectified building configuration; purple corresponds with 4 inches of deflection.

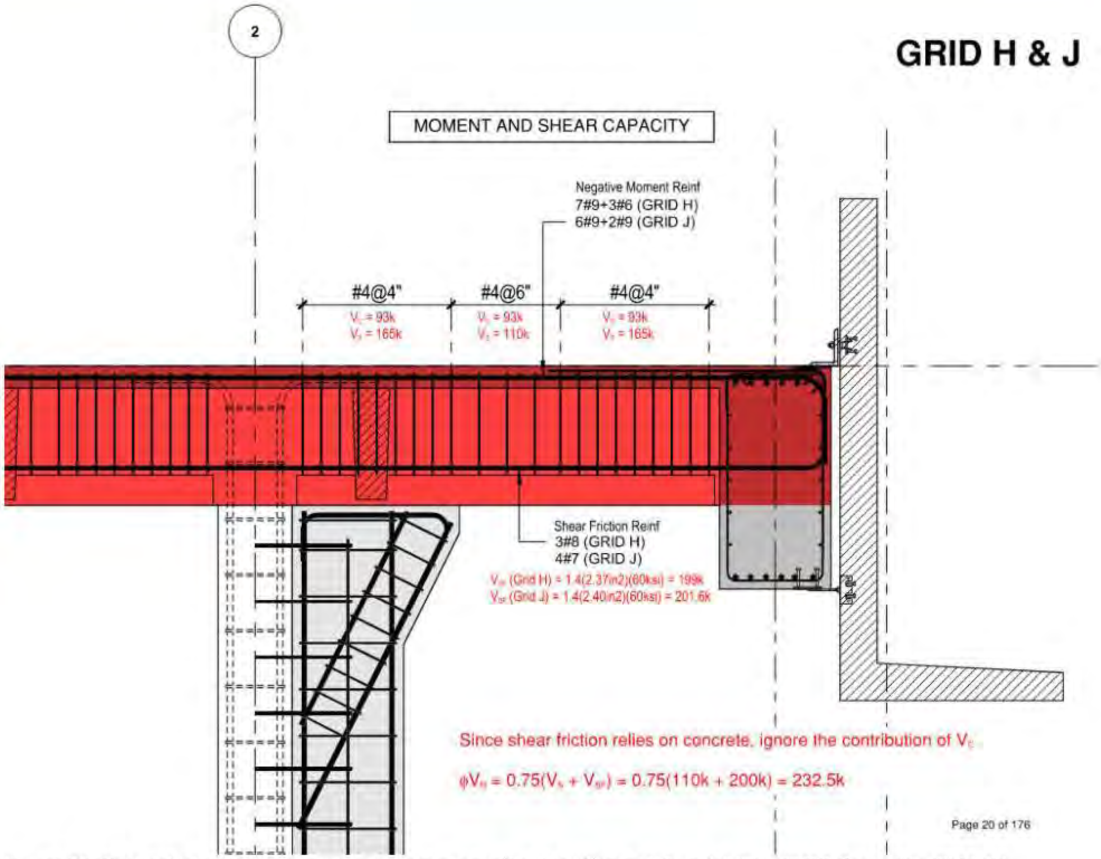


Figure 82. Calculations of cantilever beam shear capacity at gridlines H-2 and J-2 by SEOR (from File *2024-04-25_CALC_HQ Roof Cantilever Field Fix at North Elevation.pdf*) showing assumption of including shear friction of bars in bottom side (compression side) of the beam.

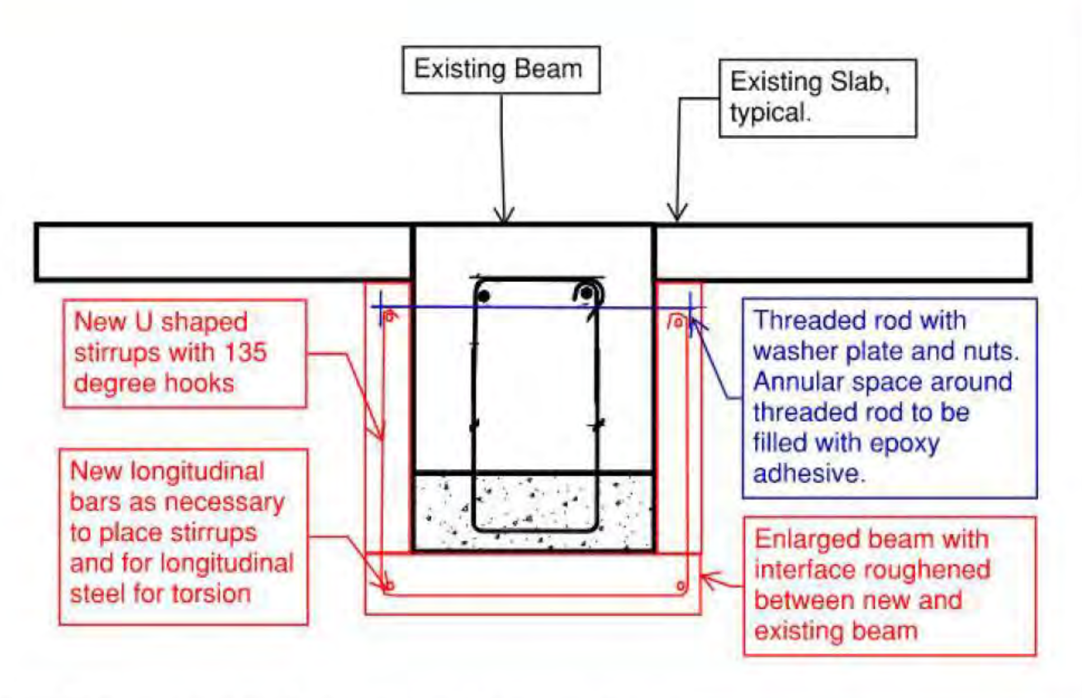


Figure 83. Conceptual solution for strengthening of cantilever roof beams

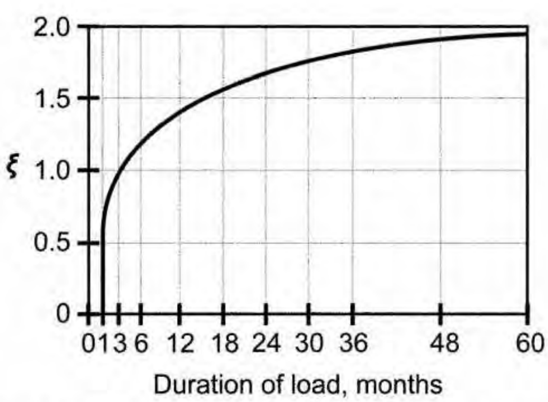


Figure 84. Curve depicting progression of long-term deflections over time for reinforced concrete elements; from ACI 318-14 Figure R24.2.4.1

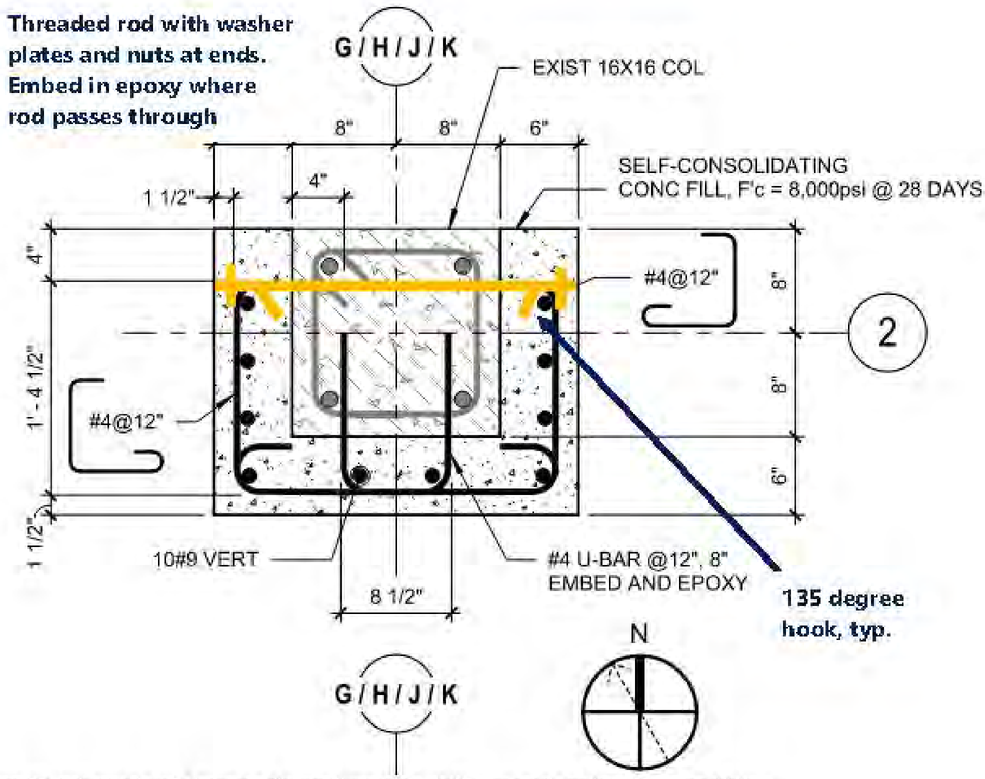


Figure 85. Schematic of alternate tie configuration. WJE annotations in orange and blue.

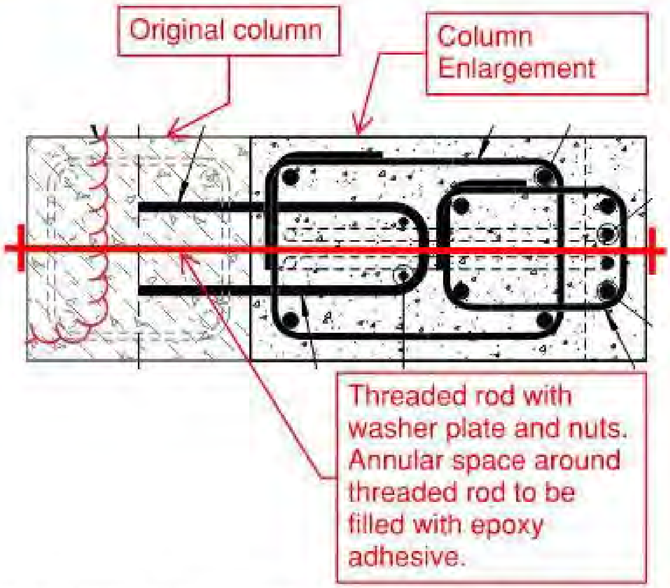


Figure 86. Conceptual rectification to tie original column and column enlargement together for shear strengthening

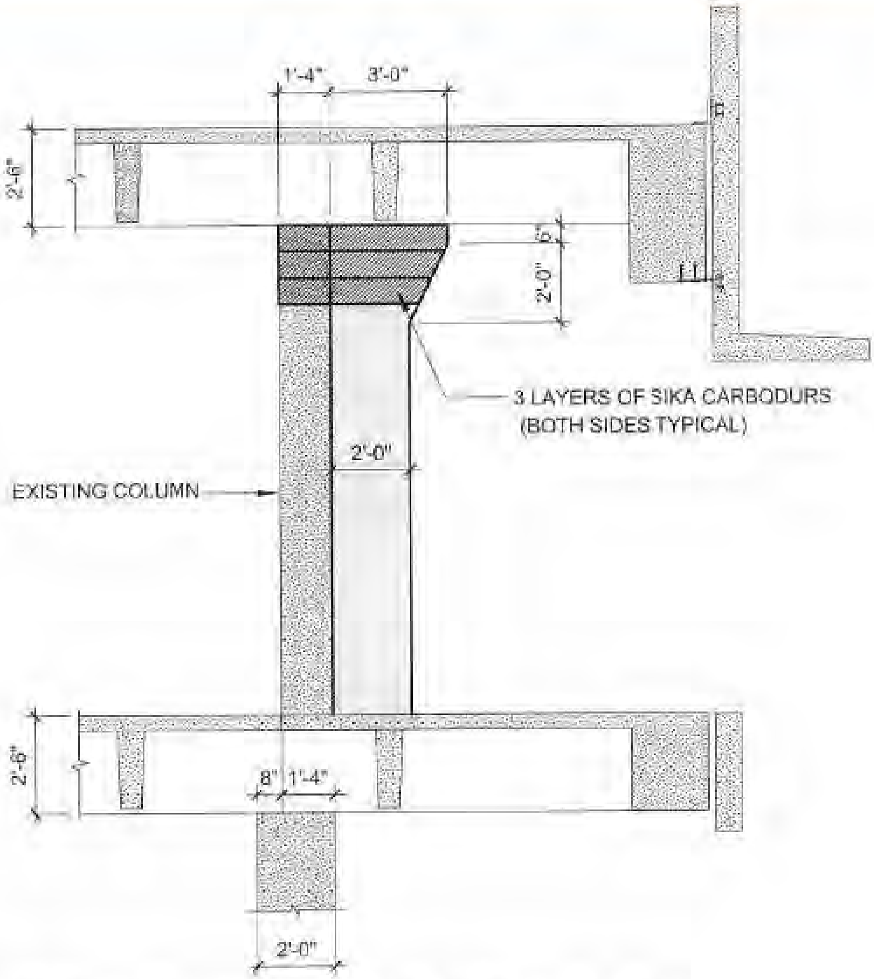


Figure 87. Conceptual rectification proposed by Lakdas to provide tie at corbel

LETTER: WJE PEER REVIEW – INTERIM FINDINGS REGARDING FOUNDATIONS

WJE

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Fort Lauderdale, Florida 33301
561.226.1220 tel
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November 7, 2024

Anthony Greg Fajardo
Assistant City Manager
City of Fort Lauderdale
100 N. Andrews Avenue
Fort Lauderdale, FL 33301

Fort Lauderdale Police Headquarters
WJE Peer Review - Interim Findings Regarding Foundations
WJE No. 2024.4855.0

Dear Mr. Fajardo:

At the City of Fort Lauderdale's (City) request, Wiss, Janney, Elstner Associates, Inc. (WJE) has completed a limited peer review of the micropile foundation design prepared by the Structural Engineer of Record (SEOR), Thornton Tomasetti (TT) and the delegated micropile designer Keller - North America (Keller), for the new Fort Lauderdale Police Headquarters building, located at 300 West Broward Boulevard, Fort Lauderdale, Florida. This letter summarizes the scope and findings of our review.

PROJECT BACKGROUND

Description of the Structure

The Fort Lauderdale Police Headquarters is a 191,000-square-foot, three-story structure currently under construction. The building will include workspace for over 700 personnel, including training rooms, public meeting areas, and community space. The building's design was led by AECOM (the prime consultant), with TT as the SEOR. The general contractor for the Project is Moss and Associates (Moss). As noted above, Keller is the delegated micropile designer. Collectively, these companies are referred to as the Project Team.

The main portion of the building is rectangular in plan, with the length of the building oriented on a north-south axis. The building's north elevation steps northward at the third floor and roof, forming a cantilever projection. The facade of the building is clad with a combination of precast concrete fascia and full-height glass fenestration systems.

The structure's gravity framing consists of one-way concrete slabs spanning between precast joists (running in the east-west direction). These joists span to and are supported by continuous concrete soffit beams (running in the north-south direction), which bring the load to the cast-in-place concrete columns.

The lateral structural system for the building is comprised of cast-in-place concrete shear walls and/or concrete moment frames.

The foundations for the columns are spread footings bearing on soil improved through vibro-compaction.

Understanding of Issues to Date

WJE understands that after the precast concrete fascia panels were installed at the cantilevered roof overhang on the north side of the building, cracking and excessive deflection of the concrete slab and beams were reported. These deflections were significant enough that the glazing units spanning between the third floor and the roof no longer fit in the allotted space between the floors.

We understand that the SEOR has acknowledged to the City that some of the roof structural design members at the cantilever at the north elevation of the building were under-designed and did not fully account for the weight of the precast concrete fascia panels. After realizing this issue, the SEOR developed a repair that enlarged the cross-section and added a diagonal strut to some of the third-story columns on the north elevation, reportedly reducing the span of the cantilever beam by 3 feet. The SEOR also developed an additional repair design to address the deflection of the beams at the cantilever on the north elevation. This additional repair added to the height of the beams by creating "upturned" sections that project above the roof slab.

In addition to the repairs described above, the SEOR developed repairs for the column foundations on the north elevation of the building to provide additional load carrying capacity. Two repair approaches were proposed. The first approach was to enlarge the existing footings. The second approach was to add micropiles, and pile cap to support the columns instead of the existing spread footings. We understand that the City and Project Team are moving forward to implement the micropile solution.

PEER REVIEW SCOPE

The City engaged WJE to provide a peer review of the repairs at the north end of the building (Phase 1) and also to do a peer review of the structural design of the entire building (Phase 2). The current focus of WJE's work is Phase 1. As the Project Team would like to move forward with implementation of the micropile foundation solution we have developed interim findings regarding the micropile foundations and the pile cap.

As part of our review, we have reviewed pertinent portions of the following documents:

- Report of Limited Geotechnical Exploration, Report No. 19284.1 by Nutting Engineers (Nutting), dated January 20, 2021
- Structural drawings consisting primarily of sheets Titled, *PERMIT SET*, by TT, dated June 10, 2022
- Pile Cap calculations, *FOR REVIEW*, by TT, dated August 23, 2024
- Sheets HQ-S6-1-04 and 05, by TT, dated September 12, 2024
- Micropile design calculations, by Keller, dated September 24, 2024
- Micropile Drawings, by Keller, dated October 2, 2024

We also reviewed the loading of the first story columns transferred to the foundation provided by the SEOR in the Structural Design North Cantilever Supporting Calculations (dated April 25, 2024). Additionally, we developed our own structural analysis model to confirm the loads provided by the EOR for the first story columns are reasonable.

DESCRIPTION OF THE FOUNDATION SYSTEM

TT's original foundation design for the building consisted of shallow spread footings bearing on native soils improved via vibro-replacement to provide an allowable bearing capacity of at least 7,000 pounds per square foot (psf). On the north elevation, the foundations supporting building columns at gridlines G/2, H/2, J/2 and K/2 are proposed by TT to be changed from shallow foundations to deep foundations consisting of pile caps supported on micropiles extending into limestone. Building loads are transferred from the structure above into the new pile caps via column jackets and steel dowels.

FOUNDATION PEER REVIEW COMMENTS

The following discussion summarizes the results of the micropile and pile cap peer review. The SEOR and the delegated micropile design engineer (Keller) should determine if any foundation design changes are warranted as a result of the peer review comments.

- The permanent casing of the micropile is structural over its length due to the lack of a full-length center reinforcing bar. Keller's design for the casing assumes a 75-year service life for the building, and an annual rate of corrosion loss equal to 0.0015 inches per year, for a total loss of 0.11 inches from the casing wall thickness. We recommend:
  - The assumed 75-year design life of the building be confirmed by the City. If a longer service life is required, the design should be reviewed and modified, if necessary.
  - The assumed rate of corrosion is not representative of a severe corrosive environment, and may or may not be suitable for the site. We recommend the Geotechnical Engineer of Record (Nutting) be consulted to either validate Keller's assumed rate of corrosion or provide a recommended corrosion rate based on site-specific conditions.
- Keller used 160 kips (1 kip is 1,000 pounds) and 210 kips as the design axial compressive service loads for the micropiles. At this time, the basis of these loads is unclear. Assuming an ultimate level design axial compressive load (Pu) of 265 kips per TT's August 23 calculations, the 210 kip axial load equates to a ratio of ultimate load to service load of 1.26, which is likely conservative. However, the column jacket, which carries 100% of the 1,020 kip load from the structure (TT's August 23 calculations) above is not symmetrical with the pile layout but is shifted south. Therefore, the southernmost micropiles will see a nominally larger portion of the total load. We recommend TT revise their analysis to account for this effect and provide demand service and ultimate level axial loads to Keller for the design of the micropiles. For record purposes, and consistent with delegated design requirements under the Florida Building Code, the design loads should be shown on TT's drawings.
- Keller estimated elastic shortening of the micropiles under design loads to be on the order of ½ inch, in addition to the unknown magnitude of micropile settlement. Total micropile shortening/settlement at design loads will be better known after the sacrificial load test required by Keller's design is performed. Once this value is known, we recommend TT review the building's structural framing and foundations for their ability to tolerate these movements, and either confirm acceptability, or modify the framing as necessary.

- The micropile cap plate shown on Keller's October 2 drawings is 12 inches square, the edge of which directly abuts the edge of the footing under perfect conditions. There are no allowances for construction tolerances. We recommend the design of the foundation be modified to account for reasonable construction tolerances on both the size of the existing footing and micropile location. On other projects of which WJE is familiar, Keller has utilized a 3 inch plan location tolerance for micropiles. We recommend the pile cap design be modified to account for such, or similar reasonable construction tolerances.
- Both TT's and Keller's design of the micropiles assumes only axial load is carried by the micropiles. As detailed on HQ-S6-1-05, there are 4-#6 hooked vertical bars enclosed by 2-#6 U-bars doweled into the existing footing outside of each micropile. This reinforcing pattern may transfer unintentional moments into the micropiles. We recommend the design of the pile cap either be modified to reduce the potential to impart bending into the micropiles, or the micropiles be designed to resist such moments.
- Keller's punching shear capacity calculation for the pile cap does not account for eccentricity of shear between the center of load and center of resistance. The design of the pile cap as currently drawn is likely acceptable, but changes to accommodate construction tolerances, and/or the addition of bending moments could result in under-designed condition. We recommend TT and Keller re-evaluate the punching shear design at the top of the micropiles, accounting for the items noted above.

SUMMARY, RECOMMENDATIONS, AND NEXT STEPS

Based on the limited peer review of the design of the micropile foundation repairs for the columns at the north end of the building, WJE has identified conditions and portions of the design which should be reviewed by the SEOR (TT) and the delegated pile design contractor (Keller) for their consideration and action. As the licensed design professional in responsible charge for the structure, the SEOR should determine if any structural design revisions are necessary. Updated drawings, calculations, and other documentation should be provided for review, coordination, approval, and record purposes.

The peer review services provided by WJE have been intended to call attention to areas of ambiguity, possible deficiency, or other anomalies that were identified during a relatively limited review of available documents. The services provided by WJE should be viewed in a proper context and not be construed as replacing or otherwise altering the contractual responsibilities of the Project Team members as they relate to the design and construction of the micropiles and pile caps. Although we have endeavored to identify areas of concern, our scope of services has not included an exhaustive or minutely detailed analysis of each design, component, or system specified on the drawings. Accordingly, the responsibility for a proper design remains solely with the design professional whose seal appears on the drawings.

WJE will continue to evaluate the repair design on the north elevation of the building and will provide peer review comments for that portion of our Phase 1 scope in a separate letter.

CLOSING

Thank you for the opportunity to assist the City with this project. Please contact us with any questions.

Sincerely,

WISS, JANNEY, ELSTNER ASSOCIATES, INC.

DONALD C. CARROLL
LICENSE
No. 80566
STATE OF
FLORIDA
PROFESSIONAL ENGINEER
11/7/24

Brent Chancellor, PhD, PE
Associate Principal

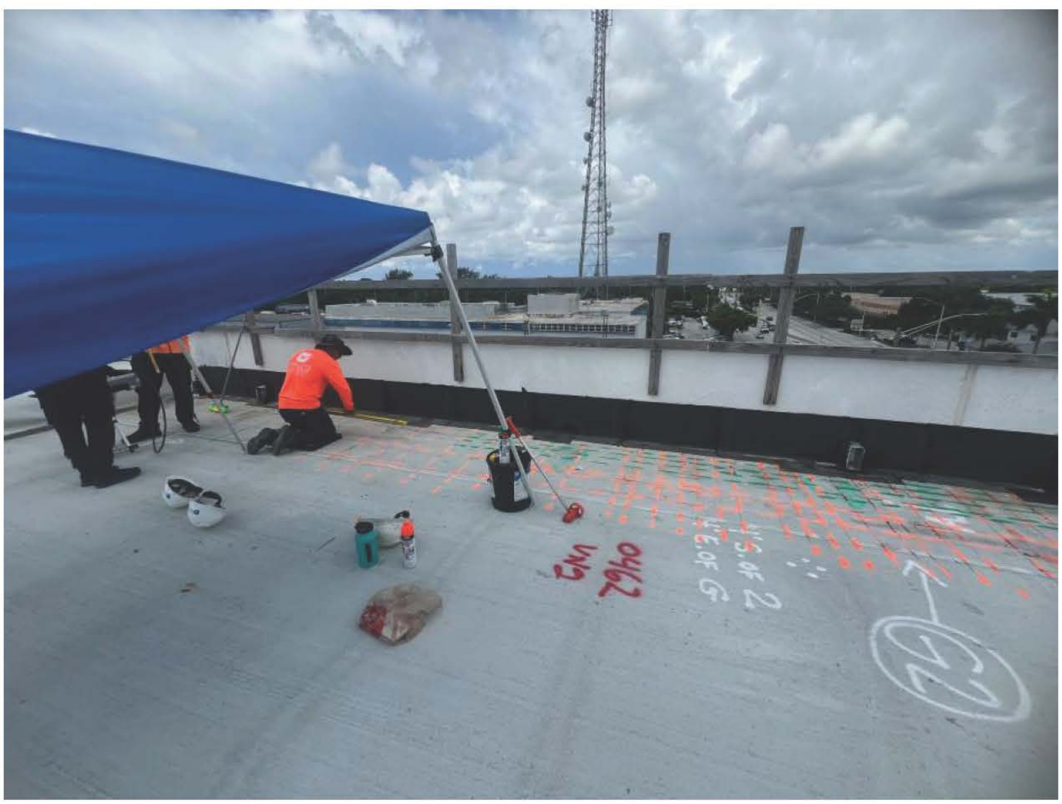
Donald Carroll
Associate Principal, PE, Geotechnical Engineering Unit Manager

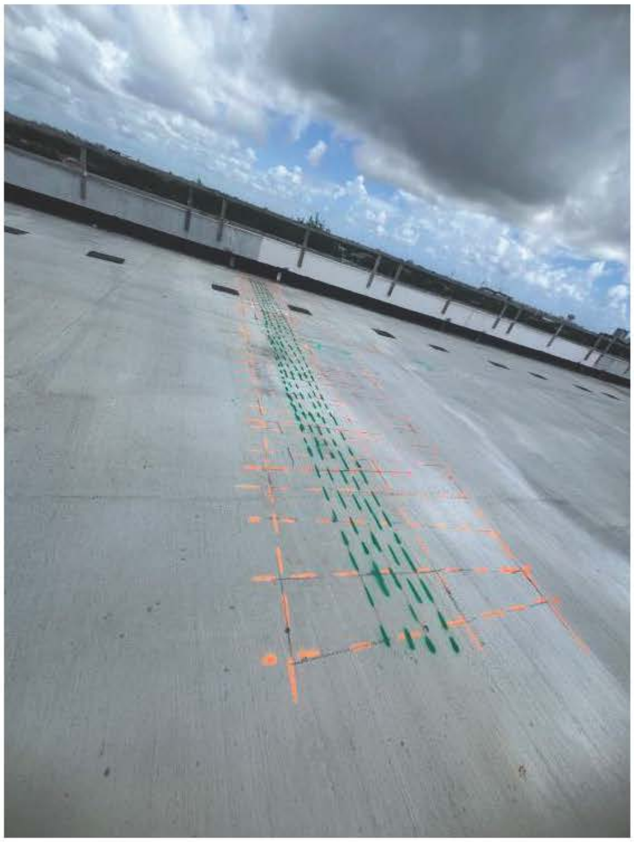
Matthew Fadden, PhD, PE
Associate Principal

This document has been digitally signed and sealed by Donald Carroll 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.

FILE NAMED *PICTURES.PDF* CONTAINING PHOTOGRAPHS OF ROOF RECTIFICATIONS

Thursday 8/29

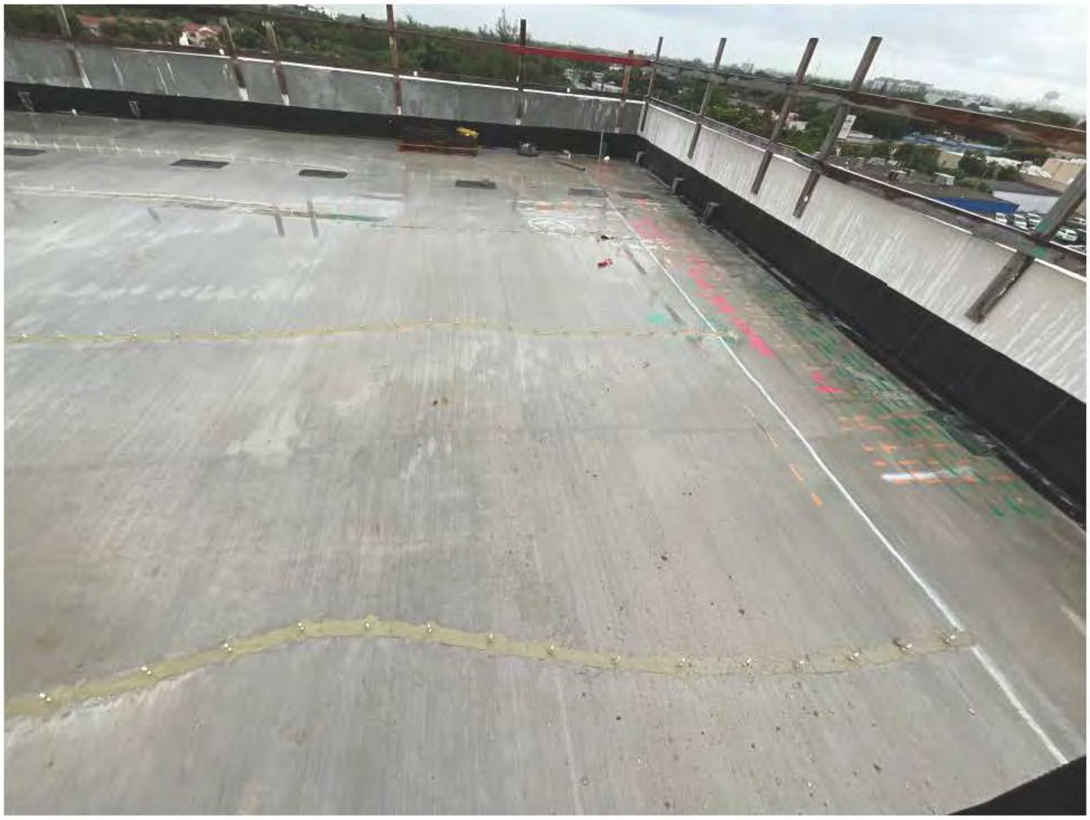





0465

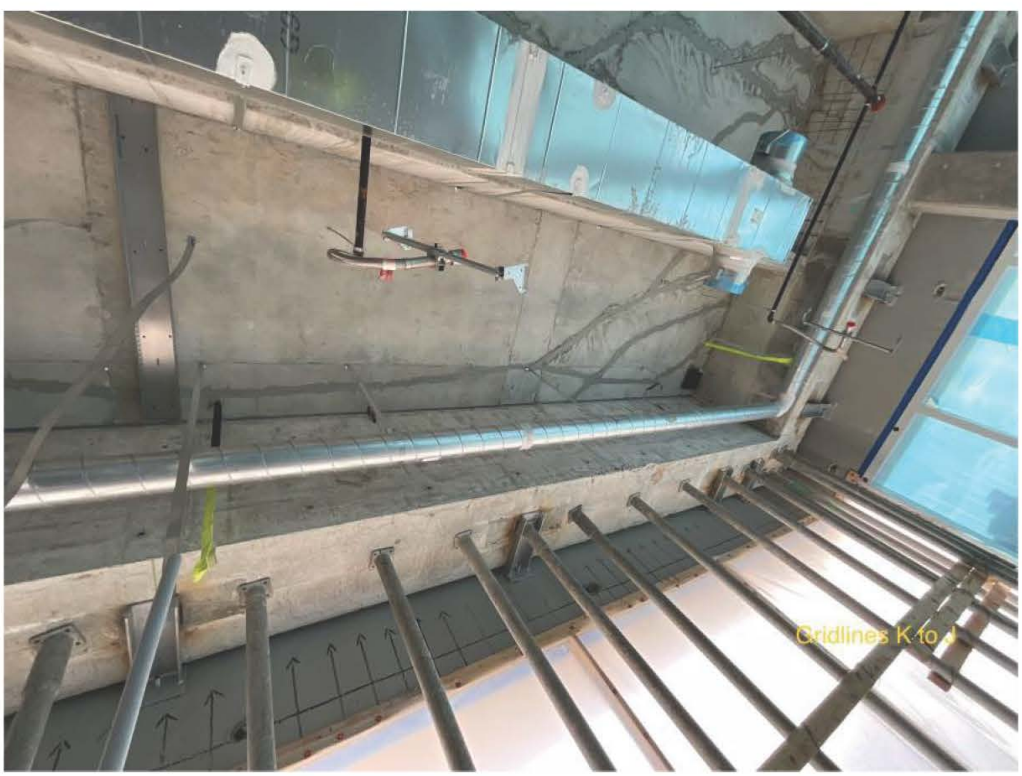

K2

Friday 8/30




K2

Saturday 8/31

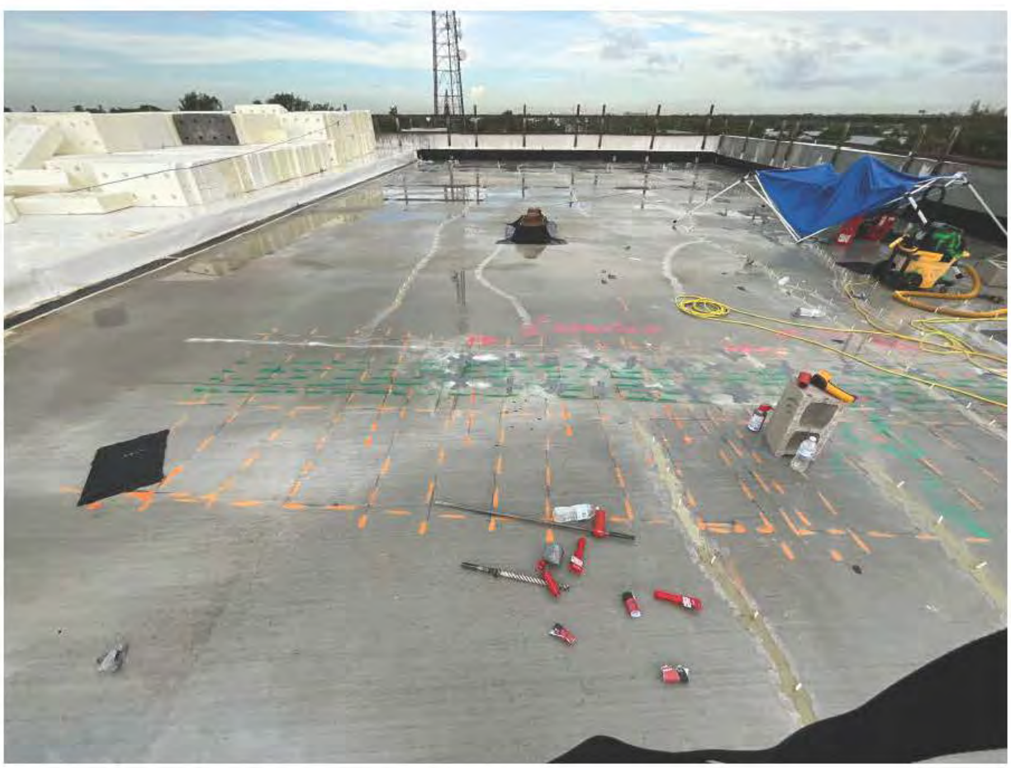


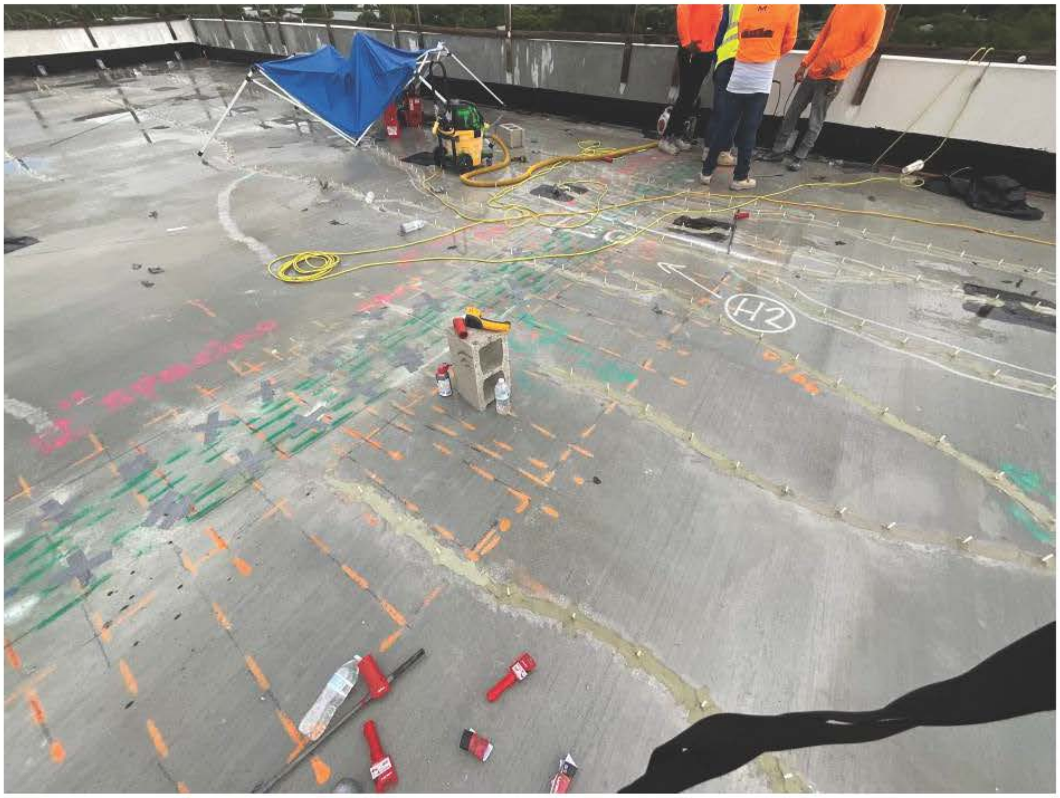

Gridlines K to J


Gridlines K to J


Gridlines K to J

Tuesday 9/3




H2


16


Gridlines J to H


Gridlines J to H

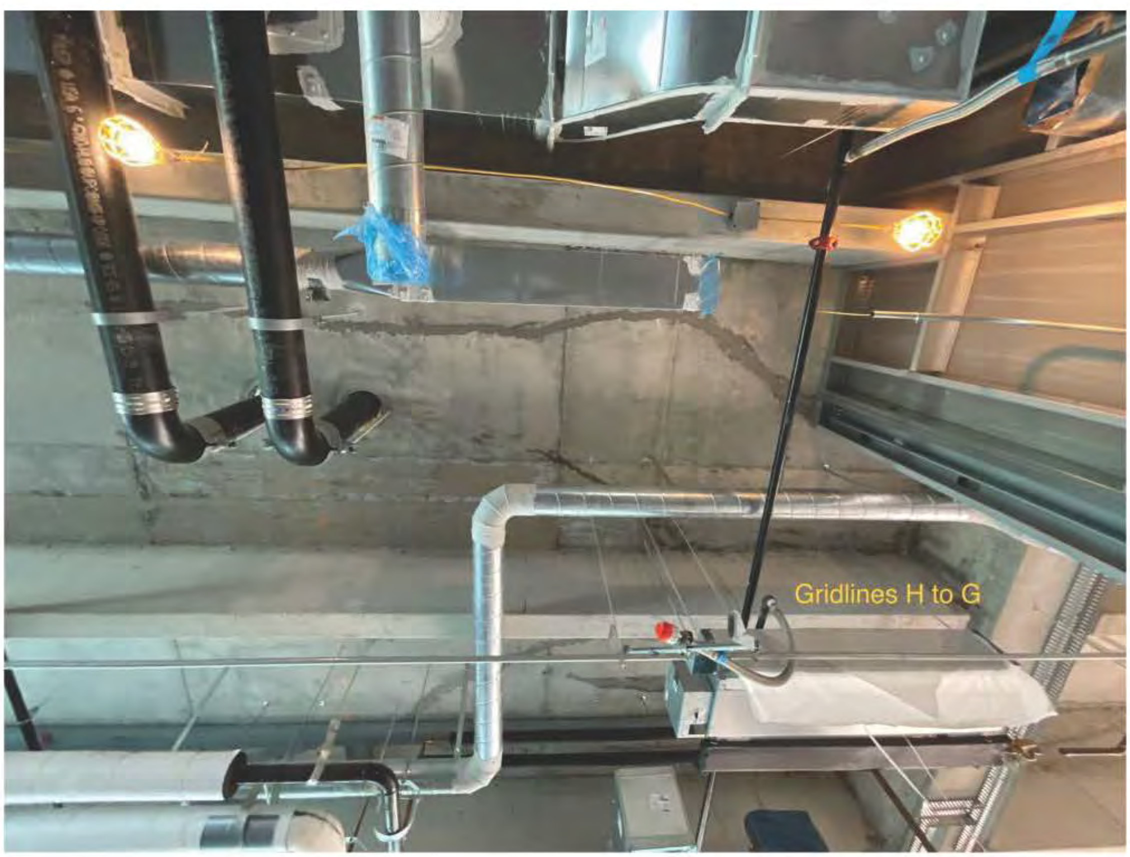

Gridlines J to H

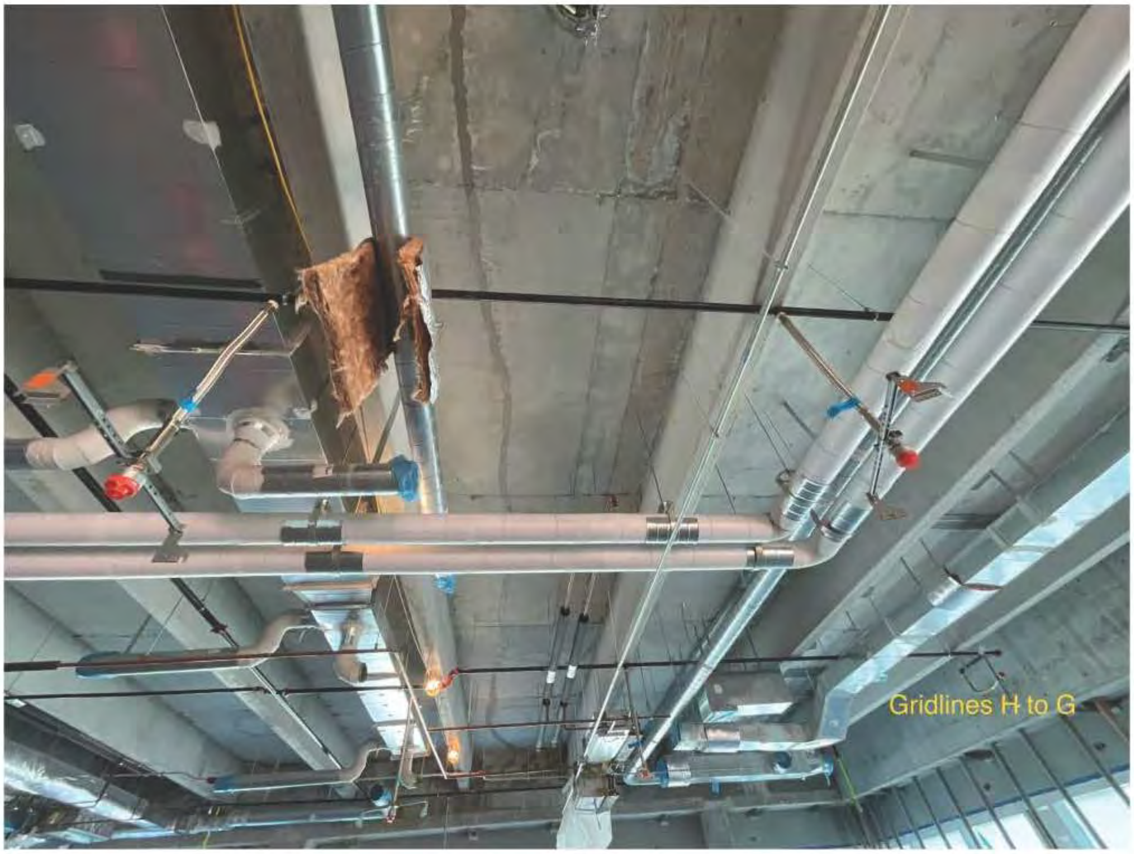

Gridlines J to H

Wednesday 9/4


U bend 12" spacing


Gridlines H to G

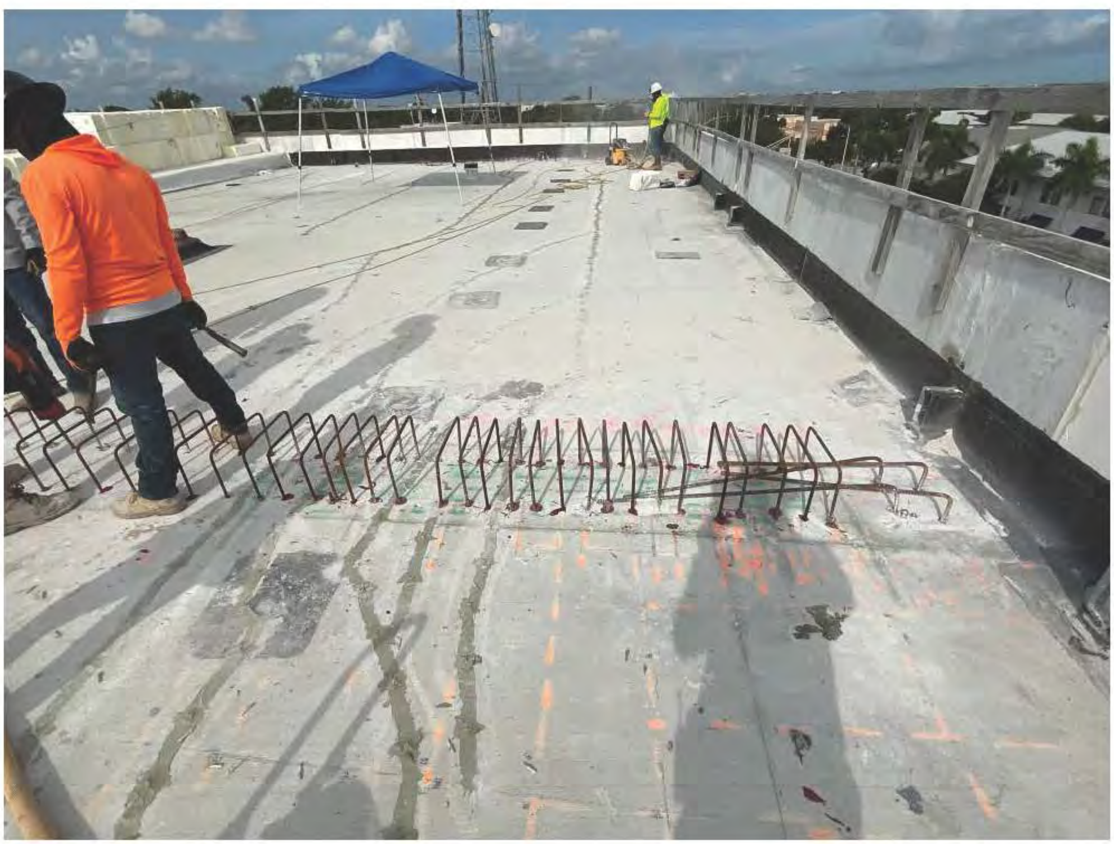

Gridlines H to G

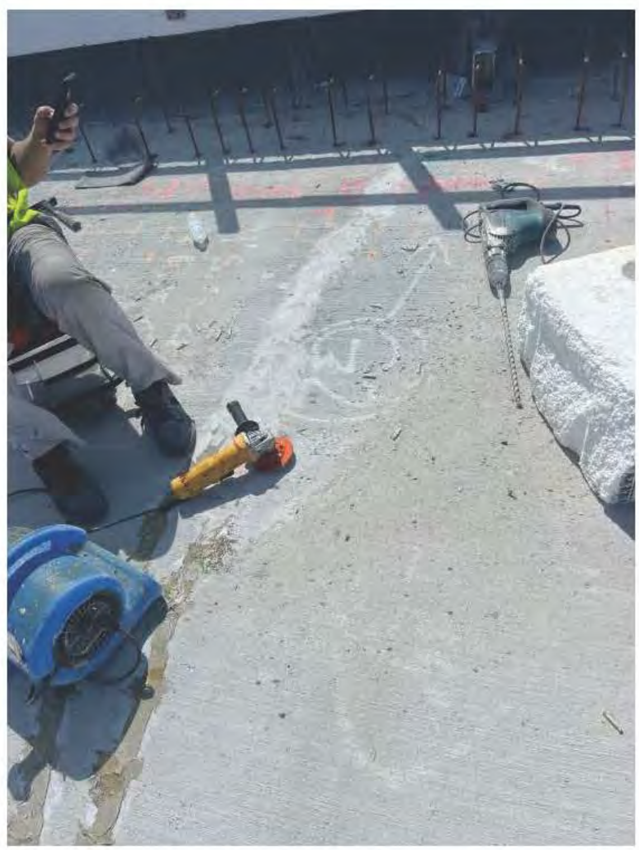

Gridlines H to G


Gridlines H to G

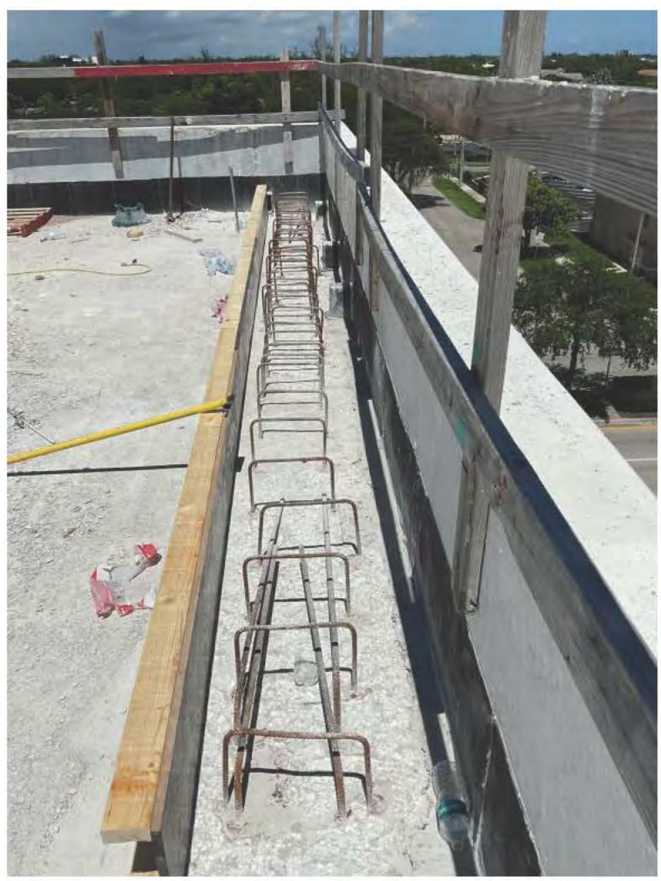
Thursday 9/5

Friday 9/6

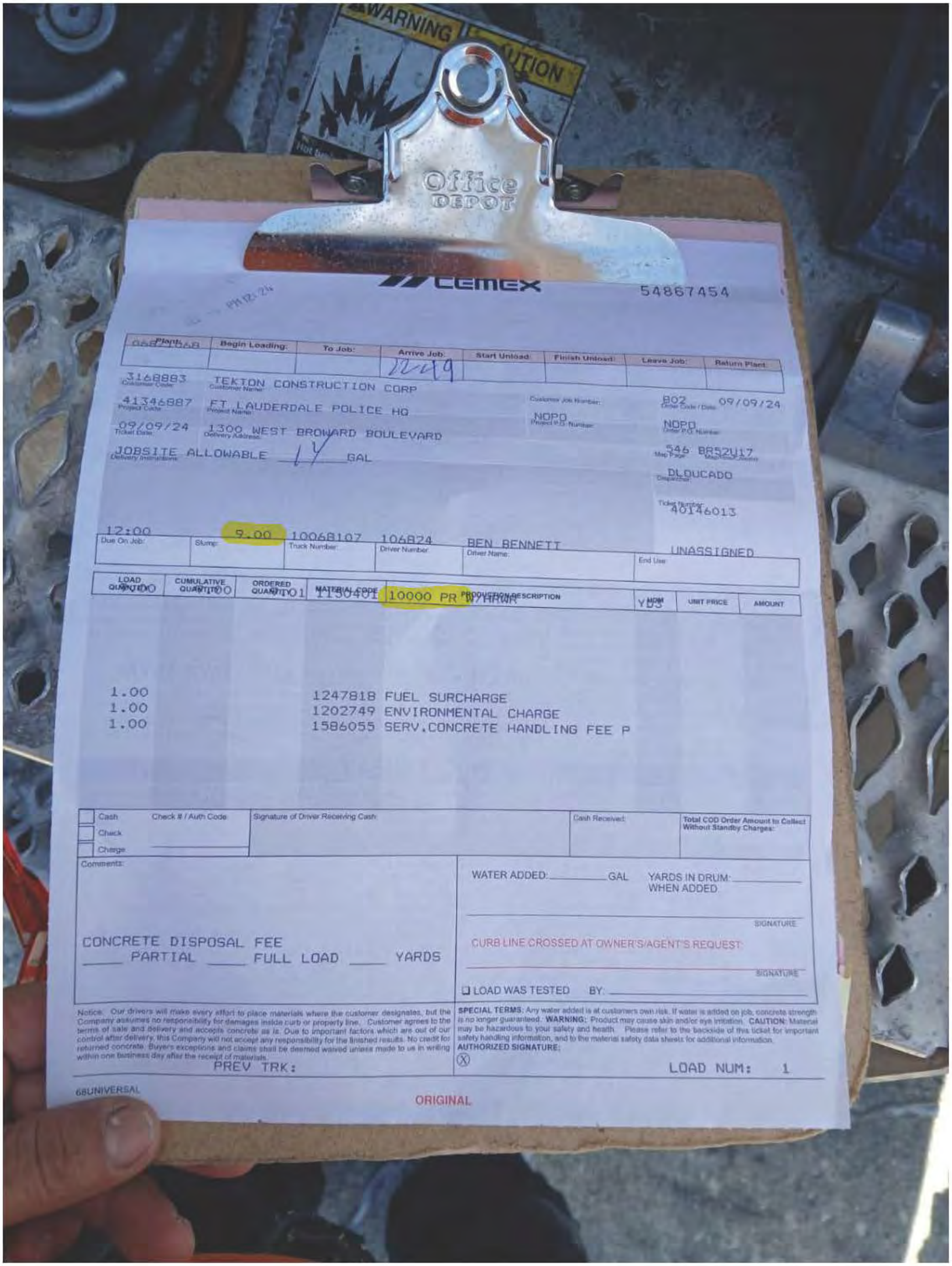




Saturday 9/7



Monday 9/9



CEMEX 54867454

Plant:	Begin Loading:	To Job:	Arrive Job:	Start Unload:	Finish Unload:	Leave Job:	Return Plant:
			12:49				

3168883 (Customer Code) TEKTON CONSTRUCTION CORP (Customer Name)

41346887 (Project Code) FT LAUDERDALE POLICE HQ (Project Name) — Customer Job Number: — 802 09/09/24 (Order Code / Date)

09/09/24 (Ticket Date) 1300 WEST BROWARD BOULEVARD (Delivery Address) — NOPO (Project P.O. Number) — NOPO (Order P.O. Number)

JOBSITE ALLOWABLE 14 GAL (Delivery Instructions) — 546 BR52U17 (Map Page / Map Row/Column)

DLOUCADO (Dispatcher)

Ticket Number: 40146013

Due On Job:	Slump:	Truck Number:	Driver Number:	Driver Name:	End Use:
12:00	9.00	10068107	106824	BEN BENNETT	UNASSIGNED

LOAD QUANTITY	CUMULATIVE QUANTITY	ORDERED QUANTITY	MATERIAL CODE	PRODUCT/DESCRIPTION	UOM	UNIT PRICE	AMOUNT
9.00	9.00	99.01	1130401	10000 PR W/HRWR	YDS		
1.00			1247818	FUEL SURCHARGE			
1.00			1202749	ENVIRONMENTAL CHARGE			
1.00			1586055	SERV.CONCRETE HANDLING FEE P			

Cash / Check / Charge — Check # / Auth Code — Signature of Driver Receiving Cash: — Cash Received: — Total COD Order Amount to Collect Without Standby Charges:

Comments:

CONCRETE DISPOSAL FEE
____ PARTIAL ____ FULL LOAD ____ YARDS

WATER ADDED: ______ GAL YARDS IN DRUM WHEN ADDED: ______

SIGNATURE

CURB LINE CROSSED AT OWNER'S/AGENT'S REQUEST

SIGNATURE

☐ LOAD WAS TESTED BY: ______

Notice: Our drivers will make every effort to place materials where the customer designates, but the Company assumes no responsibility for damages inside curb or property line. Customer agrees to the terms of sale and delivery and accepts concrete as is. Due to important factors which are out of our control after delivery, this Company will not accept any responsibility for the finished results. No credit for returned concrete. Buyers exceptions and claims shall be deemed waived unless made to us in writing within one business day after the receipt of materials.

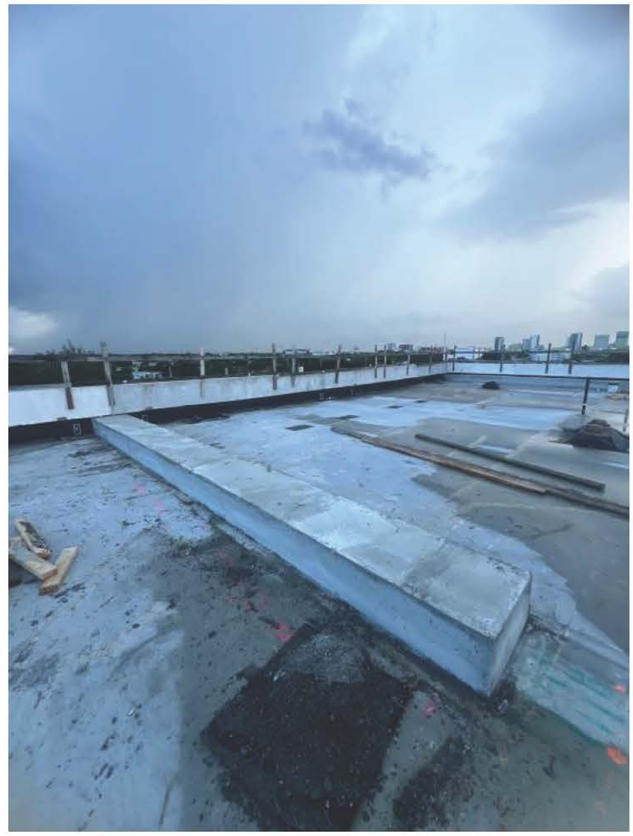
SPECIAL TERMS: Any water added is at customers own risk. If water is added on job, concrete strength is no longer guaranteed. WARNING: Product may cause skin and/or eye irritation. CAUTION: Material may be hazardous to your safety and health. Please refer to the backside of this ticket for important safety handling information, and to the material safety data sheets for additional information.
AUTHORIZED SIGNATURE:
Ⓧ

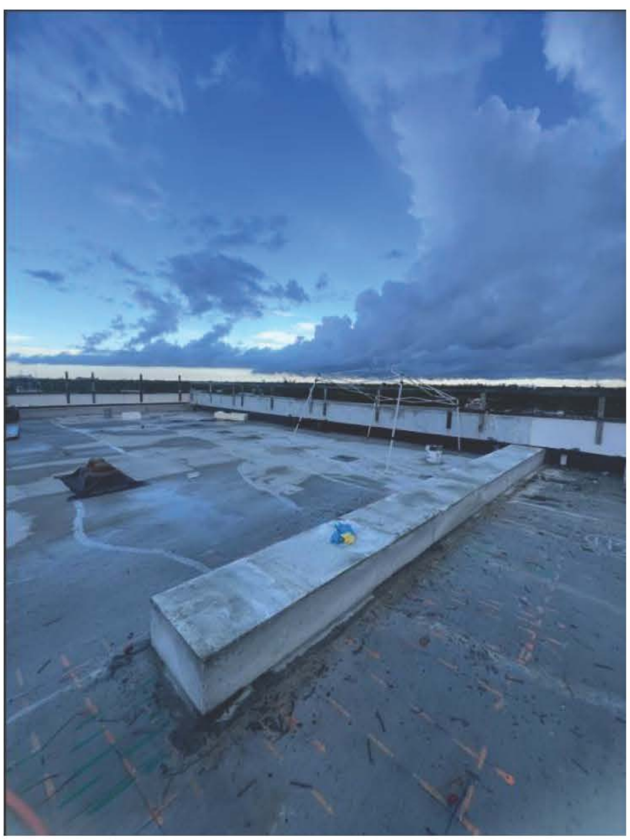
PREV TRK: LOAD NUM: 1

68UNIVERSAL

ORIGINAL

Tuesday 9/10





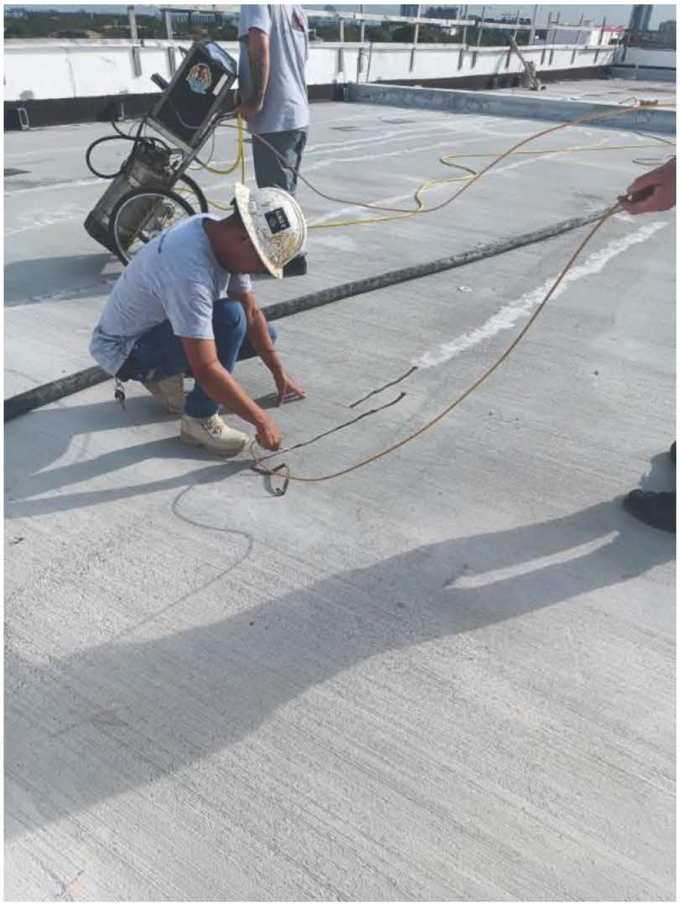
Friday 9/13

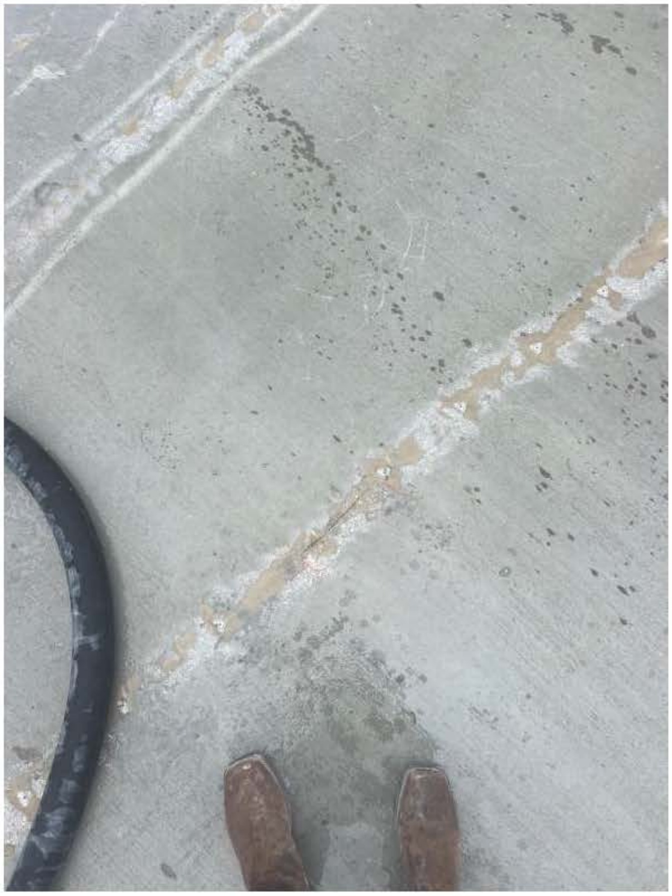
Tuesday 9/17


STANLEY


STANLEY

Friday 10/11

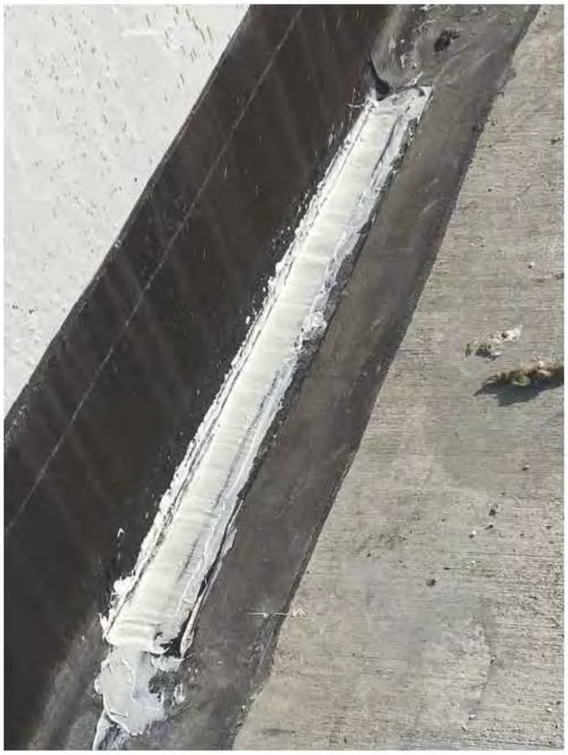


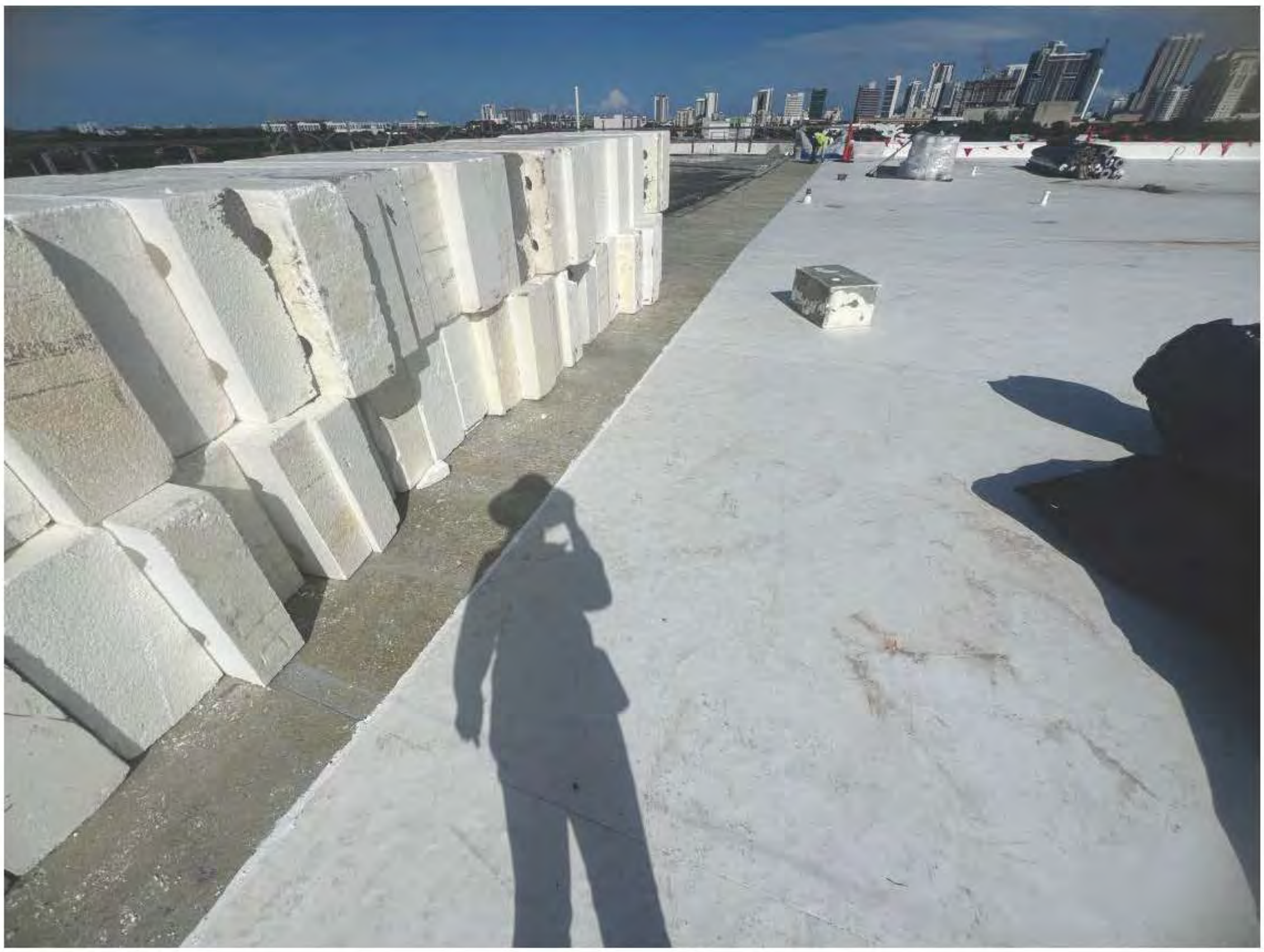




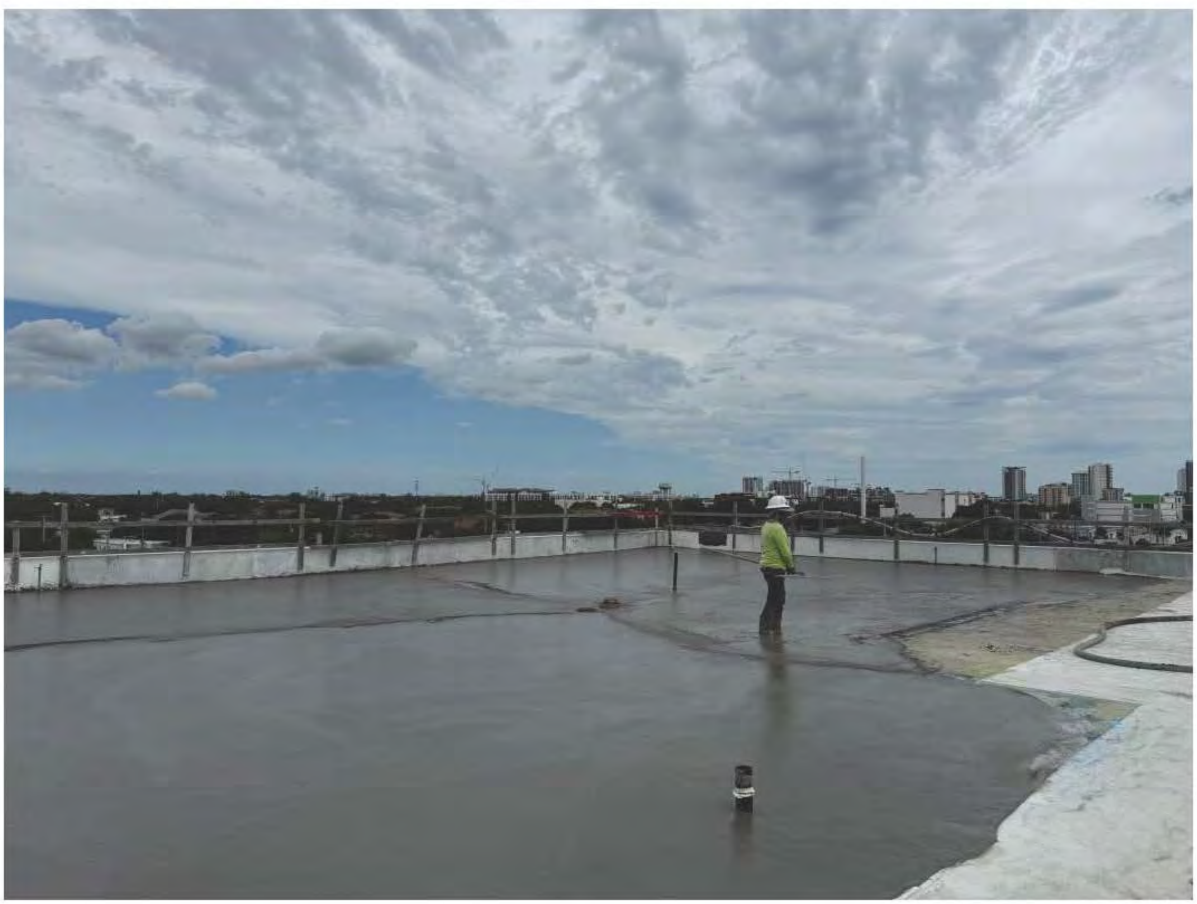


Monday 10/14





Tuesday 10/15



PENHALL TECHNOLOGIES STRUCTURAL SCANNING REPORT

Ft. Lauderdale Police HQ

Structural Reinforcement Field Report for Lakdas/ Yohalem Engineering

1300 W Broward Blvd. Ft Lauderdale

Company Overview

Thank you for allowing Penhall to assist with this project. Penhall has been the go-to concrete solutions partner for a wide-range of industries since 1957. In 2001, Penhall Company began to offer concrete scanning to better meet the growing needs of our customers. Using GPR (ground penetrating radar) to locate pipes, conduits, utility lines, rebar, voids within concrete, and other subsurface hazards ensures projects are kept on budget, on time, and most importantly, safe. With over 50 expertly trained analysts and the use of state-of-the-art equipment, Penhall is an industry leader in structural scanning, Digital X-Ray, and private utility locating using ground penetrating radar and electromagnetic techniques.

Penhall is a member of ABC (Association of Builders and Contractors), AGC (Association of General Contractors), UConn (United Contractors), CSDA (Concrete Sawing & Drilling Association) and AIA (American Institute of Architects).

Penhall Technologies proudly offers industry leading advancements in x-ray imaging with our digital x-ray solution. Unlike traditional x-ray services which use highly toxic isotopes, our x-ray machine can be turned on and off, eliminating the risk associated with constant exposure to harmful radiation. We also capture images digitally, which increases our productivity compared to using antiquated film methods.

Penhall radiographers who perform concrete x-ray services undergo specialized training and certification. Due to our service industry, and our position as the nation's leader in concrete solutions, our technicians are completely knowledgeable of common and not-so-common on-the-job hazards, and the safety procedures to be followed.

Our concrete scanning, digital x-ray imaging, and utility locating services allow our customers to identify subsurface objects or hazards before they excavate or cut into concrete. This yields a variety of advantages:

- Reduce your safety risk, keeping everyone on the job site safe
- Reduce financial exposure
- Save time, knowing exactly where to cut or drill to avoid hazards and costly delays

Project Information	
Date:	5/22/2024
Start time:	8am
Customer Name:	Pravin N
Site Address:	1300 W Broward Blvd. Ft Lauderdale
Project Name:	Ft. Lauderdale Police HQ
Site Contact:	Pravin N
GPR Analyst:	Jonathan Gongora
Weather:	Sunny/ night time rain

Scope Details	
Scan Purpose:	To locate number of rebars and their depths
Number of Locations:	18
Location Dimensions:	10'x 2'
Surface Conditions:	Good/wet
Marking Methods:	Crayon
Equipment Used:	Mini Xt with LineTrac
Dielectric Used:	7.68
KeV Used:	N/A

Project Summary
Customer needed rebar marked to the surface with depth to verify spacing and rebar presence.

Location:	Column 1, beam, 3rd Floor
Comments:	All horizontal rebar depths were labeled. From left to right it looks like the rebar starts to move closer to the surface. Starting from 8" deep to 7.5" deep.

Location:	Behind column 1, 3rd Floor
Comments:	5 rebars were spotted. Depths ranging from 1" to 6".

Location:	Front of column 1, 3rd Floor
Comments:	Briefly 5 rebars were spotted horizontally. Depths ranging 1”-3.5”

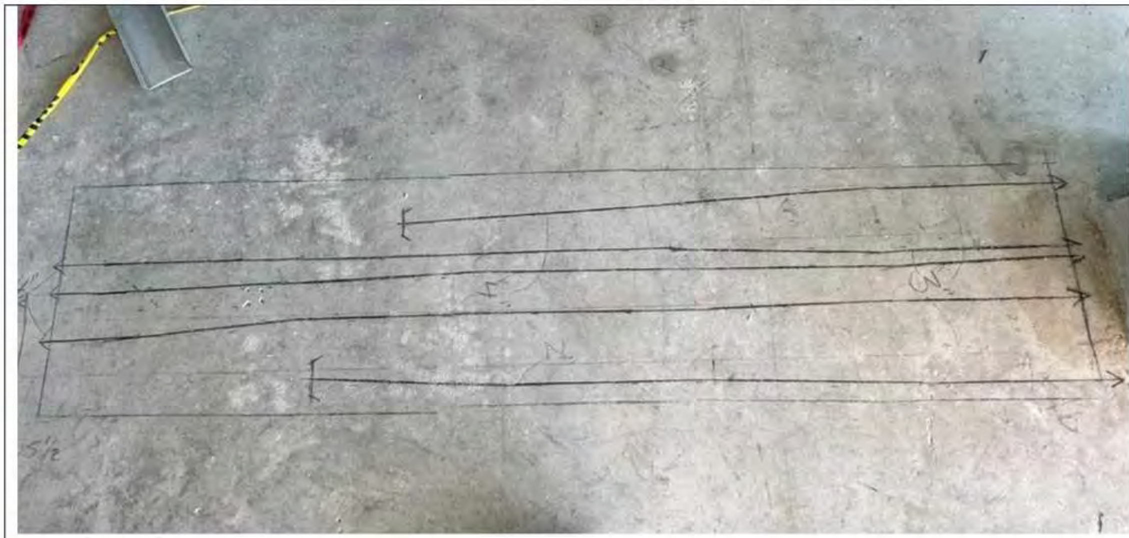
Location:	Column 2, beam, 3rd Floor
Comments:	The beam above column 2 was about 24" thick. The Mini XT was about to see about 10" deep. Vertical rebar was recorded to 7"-8" deep

Location:	Behind column 2, 3rd Floor
Comments:	In this area a congestion area was found running along horizontally. No specific amount of rebars was able to be counted. Congested area was found to be 3.5" deep.

Location:	Column 2, 3rd Floor
Comments:	In this area a congestion area was found running along horizontally. No specific amount of rebars was able to be counted. Congested area was found to be 2" deep.

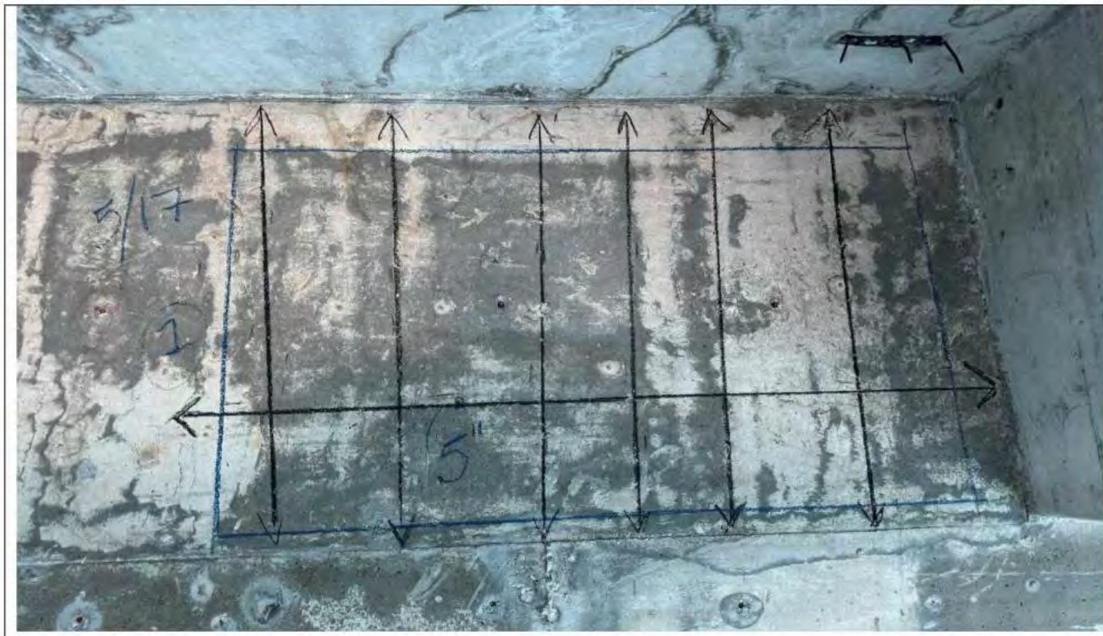
Location:	Column 3, 3rd Floor
Comments:	The beam above column 2 was about 24" thick. The Mini XT was about to see about 10" deep. Vertical rebar was recorded to 6"-7 deep

PENHALL TECHNOLOGIES



Location:	Behind Column 3, 3rd Floor
Comments:	5 horizontal rebars were found in this area with depths ranging between 2”-4’

Location:	Column 3, 3rd Floor
Comments:	In this area a congestion area was found running along horizontally. No specific amount of rebars was able to be counted. Congested area was found to be 2” deep.



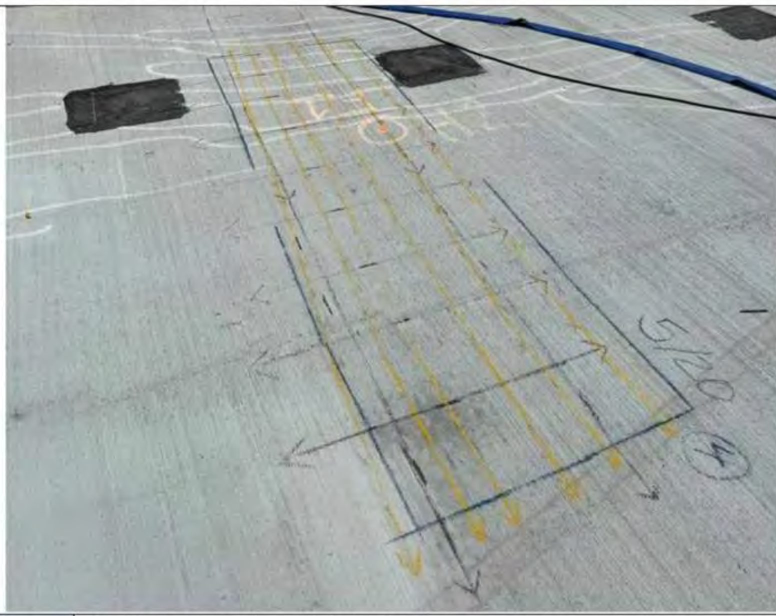
Location:	Column 4, 3rd Floor
Comments:	The beam above column 2 was about 24" thick. The Mini XT was about to see about 10" deep. Vertical rebar was recorded to 6"-7" deep. A horizonal rebar was found about 5" deep. More rebars may be present past the depth of 10"

Location:	Behind column 4, 3rd Floor
Comments:	About 5 horizonal rebars can be seen ranging at depths of 1.5" to 2.5" deep.

Location:	In front of column 4, 3rd Floor
Comments:	About 5 horizonal rebars can be seen ranging at depths of 1" to 5.5" deep.

Location:	Roof area over column 1 in front of column
Comments:	4 horizonal rebars can be seen ranging at depths of 2.5"-3" deep.

Location:	Roof area over column 1, behind column
Comments:	4 horizonal rebars can be seen ranging at depths of 3”



Location:	Roof area over column 2, behind column
Comments:	6 horizonal rebars can be seen ranging at depths of 2.5” to 5” deep.

Location:	Roof area over column 2, in front of column
Comments:	About 7 horizonal rebars can be seen ranging at depths of 2.5" to 4.75" deep.

Location:	Roof area over column 3, behind column
Comments:	6 horizonal rebars can be seen ranging at depths of 2.5 to 4” deep.

Location:	Roof area over column 3, in front of column
Comments:	About 10 horizonal rebars can be seen ranging at depths of 1.25" to 4.5" deep.

Location:	Roof area over column 4, in front of column
Comments:	About 6 horizonal rebars can be seen ranging at depths of 2.5" to 5" deep.

Additional Comments

Appendix 1: Marking Protocol.

Structure Scanning/X-Ray

Structure Scanning	
Red Mark	Identifies suspected conduit, post tension cable, and unknown hazards. Penhall recommends avoiding center line by 3" on both sides.
Black Mark	Identifies possible rebar or wire-mesh. Penhall recommends avoiding these marks by 2" on both sides.
Blue Mark	Designates the scan area. If cutting or coring needs to occur outside the blue box, the area needs to be scanned again.
Other Mark	

FOR FINDINGS THAT ARE MARKED IN BLACK, WE ADVISE AGAINST CUTTING WITHIN 2" OF CENTER LINE.

FOR FINDINGS THAT ARE MARKED IN RED, WE ADVISE AGAINST CUTTING WITHIN 3" OF CENTER LINE.

Subsurface Utility Designation

Subsurface Utility Designation	
Red Mark	Energized utility lines.
Orange Mark	Communication and Data lines.
Blue Mark	Water lines, including fire and domestic.
Yellow Mark	Gas, steam, petroleum, general fuels.
Green Mark	Sanitary sewer and drain lines
Purple Mark	Irrigation and non-potable water lines.
Pink Mark	Unknown utility and temporary survey marks.
White Mark	Proposed excavation or bore area.

PLEASE FOLLOW ALL LOCAL, STATE, AND FEDERAL LAWS ASSOCIATED WITH DIGGING NEAR UTILITIES. WE ADVISE AGAINST USING MECHANIZED EQUIPMENT WITHIN 36" HORIZONTALLY OF UTILITY MARKINGS.