

INVESTIGATION OF THE FEBRUARY 14, 2011 PARTIAL COLLAPSE OF A PARKING STRUCTURE UNDER CONSTRUCTION IN SAN ANTONIO, TX

U. S. Department of Labor
Occupational Safety and Health Administration
Directorate of Construction

August 2011



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1. INTRODUCTION

On February 14, 2011, at approximately 11:40 a.m., a partially erected precast concrete frame suddenly collapsed, injuring two employees who were working on top of the elevated piers approximately 60' away. The frame that collapsed was approximately 65' long, 56' wide and 80' high, with a total weight of 1,780 kips. Within a few seconds, the adjacent partially erected precast concrete line frame to the south also collapsed. The line frame was approximately 114' in length and 113' in height with a weight of 671 kips. Both frames fell toward the east. The failed frames were a part of the larger University Health System West Parking Garage, located on the north side of the northern expansion joint. The main parking structure south of the expansion joint remained standing. The incident site is located at the southeast corner of Wurzbach Road and Medical Drive, San Antonio, Texas.

Personnel from the San Antonio Area Office (SAAO) of the Occupational Safety and Health Administration (OSHA) arrived at the scene within hours of the incident. The OSHA investigation began soon after the incident and included interviewing witnesses, taking photographs and requesting technical information from the general contractor. On the day of the incident, the OSHA Regional Administrator for Region VI asked the Directorate of Construction (DOC), in OSHA's National Office in Washington, DC, to provide engineering assistance in assessing the collapse and in determining the cause of the incident. An engineer from DOC visited the incident site February 17 - 18, 2011 to examine the collapse, take measurements of the recovered concrete pieces, and discuss the collapse with the general contractor, the concrete subcontractor, the precast concrete designer and employees at the site. Personnel from SAAO and DOC revisited the site May 2, 2011, to take additional photographs and measurements on the recovered pieces and to collect anchor bolts and nuts for laboratory examination and testing.

The DOC's investigation included:

- Examining all photographs of the incident site, the erection sequence of the precast concrete members and the surveillance video of the frame during the collapse to determine the general failure pattern and to identify the possible initial failure point.
- Reviewing the erection requirements and the precast concrete shop drawings to determine whether the precast concrete erector was following the design instructions and using adequate erection procedures and proper connection materials.
- Performing engineering calculations to determine the cause of the failure for the partially erected precast concrete frames based on the actual strength of the connection materials from the laboratory tests.
- Conducting engineering evaluations based on the findings of the above items to determine the root cause of the incident.

2. DESCRIPTION OF THE PROJECT

The 3,300-space West Parking Garage is part of a \$900 million Capital Improvement Program of the University Health System. The garage is located in the southeast corner of Medical Drive and Wurzbach Road (See Figure 1). The structural design of the garage is a hybrid; columns and beams are typically precast concrete, and floor slabs are typically cast-in-place (CIP) concrete and post-tensioned. The overall dimensions of the garage are 470' in the north-south direction and 338' in the east-west direction (See Figure 2). The major portion of the parking structure, south of the expansion joint between Gridlines N.9 and O, was eleven stories tall and was nearly completed and remained standing after the February 14 collapse.

A smaller portion of the garage, referred to as “North Prow”, was located on the north side of the expansion joint; it was under construction and collapsed on February 14, 2011. The North Prow, at Grids P/2 through R/5, was approximately 90' (N-S) by 172' (E-W). A four-lane access road was designed to pass under it, thus, the bottom level of the North Prow was built on top of CIP piers, approximately 16' above the grade. This level matched Level 4 of the garage and the top level corresponded to Level 10, for a total of seven structurally supported levels. There was a retaining wall between Grid Lines O

and P, the portion of the garage south of Grid Line O extended down to Level 1 (See Figure 2). All erected precast members in the North Prow area collapsed (See highlighted area in Figure 2); it included an approximately 65' long, 56' wide and 80' high two-bay frame (At Grids P/2 through R/3) and an approximately 114' long and 113' high line frame (At Grids O/2 through O/4).

Project Team

Based on the available information, the project team for the garage construction consisted of the following parties:

- R-S-C-R, Inc. was the structural engineer-of-record.
- Zachary-Vaughn-Layton (ZVL) was the Construction Manager and General Contractor.
- Urban Concrete Contractors, Ltd. (UCC) was the concrete subcontractor responsible for the CIP concrete and post-tensioning. UCC also fabricated all precast columns of the garage.
- Consulting Engineers Group, Inc. (CEG) was retained by UCC to design the precast and post-tensioned structural members of the garage.
- Precast Erectors, Inc. (PEI) was retained by UCC to erect the precast concrete members. In turn, PEI retained Consolidated Crane & Rigging for the crane services for the erection and Grout Tech, Inc. for the grouting services.

All of the above firms are local companies in San Antonio, TX.

3. REVIEW OF THE CONSTRUCTION DOCUMENTS

We reviewed the following construction documents:

- Architectural drawing, prepared by Perkins+Will, Garza/Bomberger & Associates and Rehler Vaughn & Koone, Inc., dated April through December 2010.
- Structural design drawings, prepared by R-S-C-R, Inc., dated March through July 2010.
- Precast erection drawings, prepared by Consulting Engineers Group, Inc., dated April 28, 2010, approved for Job Use on August 9, 2010.

- Shop fabrication drawings for selected precast columns and beams in the collapsed area, prepared by Consulting Engineers Group, Inc., on various dates.

Pertinent requirements in the above documents related to the erection, bracing and grouting of the precast concrete frames are summarized below. Copies of the specific pages of the above documents are also included in Appendix B.

Precast Erection Drawing, Sheet No. E1.00

- From the General Notes:
“G-9 TAKE[S] ALL MEASURES NECESSARY TO PROTECT THE SAFETY OF THE PUBLIC ALONG WITH THE SAFETY OF THE STRUCTURE DURING CONSTRUCTION, SUCH MEASURES SHALL INCLUDE BUT NOT BE LIMITED TO BRACING AND SHORING OF DEAD LOADS, CONSTRUCTION LOADS AND WIND LOADS. CORRECT AT OWN EXPENSE[S] ANY SUBSEQUENT STRUCTURAL DAMAGE OR OTHER OBJECTIONABLE CONDITIONS CAUSED BY YOUR OPERATIONS.”
- From the Erection Notes:
“E-2 DRY PACK BETWEEN COLUMN AND FOUNDATION SHALL HAVE A MINIMUM STRENGTH OF 9000 PSI. DRY PACKING SHALL BE DONE IMMEDIATELY AFTER ERECTION OF COLUMNS. NO MORE THAN 2 LEVELS MAY BE ERECTED BEFORE COLUMNS ARE FULLY GROUTED.”
“E-4 STABILITY OF STRUCTURE SHALL BE MAINTAINED AT ALL TIMES UNTIL ALL CONNECTIONS ARE COMPLETED.”
- From the Precast Concrete Notes:
“P-2 THE PRECAST ERECTOR IS COMPLETELY AND SOLELY RESPONSIBLE FOR THE MEANS AND METHODS OF THE ERECTION OF ALL PRECAST PRODUCT SHOWN IN THESE DRAWINGS INCLUDING, BUT NOT LIMITED TO, THE ERECTION SEQUENCING AND THE DESIGN AND DETAILING OF ANY AND ALL TEMPORARY GUYING AND BRACING FOR THE PRECAST MEMBERS AND STRUCTURE, UNLESS NOTED OTHERWISE HEREIN.”

“P-5 CONNECTIONS ARE TO [BE] COMPLETED AS ERECTION PROGRESSES UNLESS ADEQUATE MEASURES [ARE] TAKEN BY THE PRECAST ERECTOR. PRECAST ERECTOR SHALL BE SOLELY RESPONSIBLE FOR COMPLETE ERECTION OF PRECAST CONCRETE ELEMENTS, INCLUDING BRACING, LEVELING, WELDING, BOLTING, ETC. ALL FABRICATION AND ERECTION SHALL COMPLY WITH APPROPRIATE PCI TOLERANCES.”

“P-6 COLUMN ANCHOR BOLTS ARE DESIGNED TO SUPPORT UNBRACED COLUMNS DURING ERECTION, UNLESS NOTED OTHERWISE, COLUMN BASE PLATES SHALL BE FULLY GROUTED AS SOON AS PRACTICAL AFTER THE INSTALLATION OF THE COLUMN, AT THE LATEST, GROUTING SHALL BE COMPLETED BY THE END OF THE DAY THE COLUMN IS SET.”

Connection Details between Precast Columns

A typical connection between the 48" diameter CIP column (pier) and the lower precast column is shown in Details F11 and F15 on Sheet E2.02 of the precast erection drawings. A typical connection between the precast columns is presented in Details 105 and 106 on Sheet E6.06. Both the lower and upper column connections included base plates and anchor bolts, as well as proprietary grouted reinforcement couplers (“NMB Connectors”) used to splice the No. 11 vertical reinforcement in the column. There were typically three anchor bolts provided at each connection, with diameters of 1-1/2" and 1" at the lower and upper connections, respectively.

The following specific requirements were listed in Detail F15 of Sheet No. E2.02:

- *“9,000 PSI NON-SHRINK GROUT MIX TO DRY PACK CONSISTENCY [SHALL BE] INSTALL[ED] UNDER BASE PLATE PRIOR TO ERECTING PRECAST BEAMS.”*
- *“FILL # 11 NMB SLEEVE COMPLETELY WITH HIGH STRENGTH GROUT PER NMB SPECIFICATIONS.”*
- *“PROVIDE 3"Ø X 1'-0" CORRUGATED SLEEVES[S] AROUND #11 BARS TO ALLOW FOR ALIGNMENT AT ERECTION. GROUT SOLID AFTER ALIGNMENT.”*

- *“(6) #11 X 8'-0" (USE TEMPLATE TOP & BOTTOM FOR PLUMBNESS)
[DURING PLACEMENT OF CONCRETE IN THE CIP PIER.]”*

The following specific requirements were listed in Detail 106:

- *“9,000 PSI NON-SHRINK GROUT MIX TO DRY PACK CONSISTENCY [SHALL BE] INSTALL[ED] UNDER BASE PLATE PRIOR TO ERECTING PRECAST BEAMS.”*
- *“FILL # 11 NMB SLEEVE COMPLETELY WITH HIGH STRENGTH GROUT PER NMB SPECIFICATIONS JUST PRIOR TO ERECTING UPPER COLUMN (TYPICAL).”*

Anchor Bolts, Plate Assembly AB001

The fabrication drawing for the anchor bolts to be embedded in the CIP columns, Plate Assembly AB001, indicates that the anchor rod was to be *“A193-B7 HIGH STRENGTH THREADED ROD OR F1554 Grade 105”*, 2' 4" long and 1-1/2" in diameter. The top-most nut was to be a *“HIGH STRENGTH NUT (A563)”*, and the other nuts were to be *“STANDARD HEX NUTS.”* Plate washers are indicated at the nuts above and below the base plate of the precast column and between two nuts near the lower end of the anchor bolt embedded in the CIP pier. The plate washers were to be 3-1/2" square by 1/4" thick. The top-most nut was specified as 1-1/2" thick, consistent with a heavy hex nut. The height of the nuts below the base plate, intended for positioning and leveling only, was specified as 1".

Horizontal Bracings, Sheet No. E4.01B

Sheet No. E4.01B of the precast erection drawings specified that seven levels of horizontal bracings were to be installed at the columns between Gridlines O and P (See Figure 3). Sheet No. E6.15, Detail 283 also specified that the type of the bracings were to be adjustable in length and 6" in diameter pipe braces (B-5).

4. DESCRIPTION OF THE COLLAPSE

An engineer from DOC visited the incident site on February 17-18 and May 2, 2011, to examine the collapse and discuss the installation sequence of the precast members with the GC (ZVL), the concrete subcontractor (UCC) and the precast designer (CEG). The installation sequence and collapse in the North Prow area are described below.

Installation Sequence of the Precast Members

A mobile crane was used to hoist and install precast columns and beams. The crane was stationed on the east side of the garage and the erection progressed northward. Thus, the erection of the garage was generally proceeding from south to north. When the crane was parked at a particular location, the local erection would be from west to east and from the bottom level to the top level. At the time of the collapse, seven levels of precast members had been erected on top of six CIP piers, which extended approximately 16' above the grade. However, no slabs or slab formwork were in place at the time of the collapse.

Pattern of the Collapse

The lower and upper precast columns had already been erected before the collapse at the top of the six piers at Grids P/2 through R/3. A total of 12 columns fell at the time of the collapse. Each column had a specific identification on its base plate. The location where the collapsed columns landed is shown in Figure 4. It was therefore determined that 11 out of the 12 columns fell toward the east. Column C-143, the upper column at Grid R/2, was the only exception as it fell toward the south. Column C-140 was the upper column at Grid O/2 on the south side of the retaining wall as it fell toward the north. Thus, the general direction of the collapse was toward the east.

Based on the horizontal fall distance from the base of the lower column to its supporting piers in Figure 4, it appeared that Grid Line 2 columns fell near their supporting piers. In fact, the bottom of the lower column at Grid R/2 was still attached to its supporting pier. However, Grid Line 3 columns fell farther east from their supporting piers. Thus, we believe the three columns on Grid Line 3 led the collapse and the three columns on Grid Line 2 were pulled by the Grid Line 3 columns to collapse.

We also reviewed the surveillance video from a security camera located at the northeast corner of the garage. It captured the collapse of the precast frames from a distance. Four still photographs taken from the video are presented in Figure 5. We made the following observations:

- Initial movement of the frame appeared to occur on Grid Lines P and Q with the Q Line slightly ahead of the P Line (Figure 5 - A and B).
- The precast members at Grid R/2 appeared to fall after those on Grid Lines Q and P, and Grid R/3 (Figure 5 - C).
- It appeared that each of the six columns fell as an individual unit, i.e., the connection between the lower and upper columns remained intact during the fall (Figure 5 – A, B and C).
- The columns and beams along Grid Line 0 appeared to fall a few seconds after the collapse of the frame at Grids P/2 through R/3 (Figure 5 – D).

Column to Pier Connections

From the above description, the initial breaking point for the six precast columns appeared to be at the connection between the top of the CIP pier and the bottom of the lower column. Figures 6 through 17 present the conditions of these connections after the collapse. Our general observations of these connections are described below:

- Three anchor bolts, 1-1/2" in diameter, were installed on each pier as specified in the precast erection drawings (Figures 6, 8, 13 and 15).
- Four or six #11 rebars were also installed at each pier as specified.
- Approximately 2" high plastic shim packs were present, but not at the proper location as specified in the erection drawings (Figures 6, 8 and 10).
- 9,000 psi non-shrink grout had not been placed between the top of the CIP piers and the bottom of the precast column as required by the precast erection drawings (Figures 6, 8 and 10).
- High strength grout had not been injected into the NMB sleeves, a proprietary metal coupling to splice #11 rebars, as required by the precast erection drawings (Figures 6, 8, 10 and 16).

- Grout had not been placed in the corrugated sleeve around the #11 rebars extending from the CIP pier as required by the precast erection drawings (Figures 6, 8 and 16).
- 9,000 psi non-shrink grout had not been placed in the three pockets on top of the base plate (Figures 7, 10, 13, 15 and 17).
- The holes in some base plates for the anchor bolts had been enlarged at several locations by flame-cutting (Figures 7, 9 and 17).
- The size of the anchor bolt assembly was field measured. The anchor bolt was 1.48" in diameter, very close to the specified 1½". The thickness of the top nut was 1.25", instead of the specified 1½". The thickness of the leveling nut below the base plate was 0.79", instead of 1". The washer was 3.5" X 3.5" as specified, and 0.22" in thickness, very close to the specified ¼". In addition, the leveling nuts appeared to be zinc-coated. There were no markings on the nuts to identify their grades.

The conditions of the six column-to-pier connections after the collapse are summarized in Figure 18 and described below:

- Since the entire six-column frame fell toward the east direction, the west bolts and south bolts failed in tension, while the east bolts failed in compression or bending.
- Along Grid Line 2, for Piers P2, Q2 and R2, the six bolts in tension, the thread of the five top nuts were stripped off from their respective bolts. For the sixth bolt at Pier P2, the top nut was partially sheared off of its cross sectional area through an oversized hole in the base plate. In regard to the three east bolts, two fractured within the threaded length of the bolt above their leveling nuts and the third bolt at Pier R2 was still attached to the Column R2.
- Along Grid Line 3, for Piers P3 and Q3, the west bolts fractured above the leveling nuts; the south bolts and east bolts were pulled and compressed in the crushed concrete, respectively, and fell together with the base plate (Figures 13 and 15). For Pier R3, the thread of the top nut was stripped from the west bolt, the south bolt fractured below the leveling nut and the east bolt fractured above the leveling nut.
- The thread of the leveling nut below the base plate at the east anchor bolt was stripped and the nut was pushed downward to the non-threaded portion of the bolt at Columns P2, Q2, P3, Q3 and R3 (Figures 6, 9, 13, 15 and 16). The east leveling nut

had an approximately 3" downward displacement at Columns Q2, P3 and Q3. We did not have an opportunity to check the condition of the east leveling nut at Pier R2.

In addition, the witness statements indicated that immediately prior to the collapse, *"concrete fell to the ground about the size of a basket ball, then a shower of little pieces of concrete between the large pieces and the smaller pieces and noticed columns leaning over."* Thus, we believe that the anchor bolt assembly was critical to determining the casual of the collapse. Thus, in the May 2 visit, we collected the south anchorage assembly of Column Q3 (Figure 15) and an intact anchor bolt from the incident site and sent them to the OSHA Salt Lake Technical Center for laboratory examination and testing.

Column to Beam Connections

During erection, the temporary column-beam connection consisted of a coil bolt assembly from the side of a column connected to a steel plate projected out from the top face of the beam (See Figure 19). Figure 20 presented the failure condition of this connection. We made the following field measurements:

- The connection component from the beam top was a 3" wide and ½" thick steel plate projecting 4.0" above the concrete face. This steel plate was located 12" from the end of the beam. The size of the hole in the steel plate was 1.5" in diameter.
- From the column side, a ¾" diameter coil bolt assembly projected horizontally out from the side of the column; the thickness of the nut was 0.7", the C (corner to corner) dimension of the nut was 1.38" and the F (face to face) dimension of the nut was 1.21". Thus, the size of the nut was less than the diameter of the connecting hole in the steel plate. Washers were used to bridge the bearing area of the nuts to the oversized hole. The size of the washer was 2" O.D., 0.8" I.D. and 0.12" in thickness.

Based on the above measurements and Figure 20, the hole in the steel plate of the precast beam was oversized and the size and the thickness of the washers could not provide adequate stiffness to transfer the applied load from the connecting coil bolt. As a result,

it made this connection ineffective to provide any flexural resistance during the collapse of the precast frame.

5. ANALYSIS AND DISCUSSION

Summary of the Laboratory Test Results

On May 2, 2011, the OSHA San Antonio Area Office collected one intact anchor bolt and one anchorage assembly (an anchor bolt, four nuts and three washers) from the incident site, and sent them to the OSHA Salt Lake Technical Center (SLTC) for laboratory examination and testing. The SLTC report is included in Appendix B, and the test results are summarized below:

- The length of the anchor bolt was estimated to be 2' 4", with a nominal diameter measured to be 1.5". The upper thread length was measured to be 8.5" and the lower thread length was 5.5".
- Both anchor bolts met the specification of ASTM F1554, Grade 105, with a yield strength of 105 ksi and a tensile strength of 125 to 150 ksi (Reference 9).
- Based on SLTC Rockwell B (HRB) hardness values and other observations, all four nuts of the anchorage assembly could only meet the specifications of ASTM A563, Grade O, with a proof load stress of 69 ksi (Reference 8).
- The thickness of the top (#4) nut was specified to be a 1½" thick, 1½" diameter high strength hex nut. The measured thickness was 1.2760", not 1½". The top nut did not meet the standard for a heavy hex nut as per ANSI B18.2.2.
- The #3 nut was a leveling nut below the base plate with a measured thickness of 0.85". Since it was zinc-coated, the proof load stress was reduced to 52 ksi, as per ASTM A563 – 07a, Table 3 (Reference 8).
- The two bottom (#2 and #1) nuts were measured to be around 1.28" to 1.34" thick.

Estimation of the Tensile Strength of the Bolts

From the Strength Design of Anchorage to Concrete (Reference 4),

$$N_s = A_{se} \times f_{ut}$$

where A_{se} is the effective area of the threaded anchor = 1.41 in².

f_{ut} is the maximum tensile strength = 125 to 150 ksi.

$$N_s = 1.41 \text{ in}^2 \times (125 \text{ to } 150 \text{ ksi}) = 176 \text{ to } 212 \text{ kips.}$$

Thus, the tensile strength of the anchor bolt was estimated to be between 176 and 212 kips.

Estimation of the Shear Strength of the Nuts

Top Nut in the Design Condition

From ASTM F1554 (Reference 9), the recommended nut should be an A563 Grade D with a proof load stress of 135 ksi. Drawing AB001 specified the thickness of the top nut to be 1.5". The shear area of root of nut threads (A_{ts}) (Reference 6) is:

$$A_{ts} = \pi \times 1.5" \times 0.75 \times L_e$$

where L_e is the height of the nut = 1.5".

$$A_{ts} = 3.14 \times 1.5" \times 0.75 \times 1.5" = 5.30 \text{ in}^2.$$

$$F = S_U \times A_{ts}$$

where S_U is the ultimate shear strength of the nut (50 to 60% of the proof load stress, Reference 7).

$$F = (0.5 \text{ to } 0.6) \times 135 \text{ ksi} \times 5.30 \text{ in}^2 = 358 \text{ to } 429 \text{ kips.}$$

Thus, the shear strength of the top nut in the design condition was estimated to be between 358 and 429 kips. The strength of the top nut was higher than the strength of the bolt. The failure of the anchor bolt assembly would thus be due to the fracture of the bolt within its threaded length. This is a normal mode of failure. Thus, the grade and the size of the top nut in the design condition was adequate.

Top Nut in the As-built Condition

From the laboratory test results, the top nut in the as-built condition was A563 Grade O with a thickness of 1.28". :

$$A_{ts} = 3.14 \times 1.5" \times 0.75 \times 1.28" = 4.52 \text{ in}^2.$$

$$F = S_U \times A_{ts} = (0.5 \text{ to } 0.6) \times 69 \text{ ksi} \times 4.52 \text{ in}^2 = 156 \text{ to } 187 \text{ kips.}$$

Thus, the shear strength of the top nut in the as-built condition was estimated to be between 156 and 187 kips. The average strength of the nut (172 kips) was lower than the

average strength of the bolt (194 kips), thus, it was observed that the majority of the nut threads were stripped off during the collapse. However, since the maximum shear strength of the nut (187 kips) was slightly higher than the minimum tensile strength of the bolt (176 kips), a few fractures of the bolt at the threaded lengths were also observed.

Leveling Nut in the As-built Condition

From the laboratory test results, the leveling nut below the base plate had a measured thickness of 0.85". Since it was zinc-coated, the proof load stress was 52 ksi. The shear strength of the nut was calculated as follows.

$$A_{ts} = 3.14 \times 1.5" \times 0.75 \times 0.85" = 3.00 \text{ in}^2.$$

$$F = S_U \times A_{ts} = (0.5 \text{ to } 0.6) \times 52 \text{ ksi} \times 3.00 \text{ in}^2 = 78 - 94 \text{ kips}.$$

The shear strength of the leveling nut in the as-built condition was estimated to be between 78 and 94 kips.

Estimation of the Concrete Breaking Strength of the Anchorage System

Four #4 ties at 4" centers were installed around the three anchor bolts and six #11 rebars (See Figure 21). The embedment length of the anchor bolts and rebars were 1'-7" and 6'-9", respectively. In addition, twenty-seven #9 rebars at approximately 5½" centers with #3 ties at 6" centers were installed inside the perimeter of the CIP pier (See Structural Drawing WG-201). Based on ACI 318-08, Appendix D, Paragraph D.4.2.1, when sufficient anchorage reinforcements are provided in the concrete, calculations of the concrete breakout strength are not required. The design of the anchorage system is governed by the strength of the anchor bolt or nut. It should be noted that for the four anchorage assemblies still attached to the base plates after the collapse (Figures 13 and 15), they fell with the crushed concrete, and were not pulled out from the intact concrete.

Estimation of the Column Loads at the Time of the Collapse

According to the GC and the Concrete subcontractor (UCC), all precast columns and beams were erected prior to the collapse at Grids P/2 through R/3. On this basis, the column loads for each column were calculated and presented in Figure 22. The weight of

precast members was taken directly from the fabrication drawings. The total column load on each column is listed in Figure 22 for the following discussion.

Discussion of the Collapse

The precast erector did not install the required non-shrink grout below the base plate and high strength grout to the #11 NMB sleeves. All column loads were therefore supported by the three leveling nuts below the steel plate. In comparing the maximum shear strength of the three leveling nuts at 282 kips ($= 3 \times 94$ kips) to the column loads in Figure 22, the loads in Columns Q3, Q2, R3 and R2 exceeded the above value and the leveling bolts started yielding and then the columns started tilting. As the loads on the other two Columns P3 and P2 were less than 282 kips, they remained stable. The initial tilting of the above four columns were limited in the east-west direction, due to the resistance provided by the precast beams erected in the north-south direction connecting the two stable columns (P3 and P2). A review of the location of the four columns which failed (Figure 4) indicated that initially columns Q3, Q2 and P3 were leaning toward the east direction, while P2 was leaning toward the west direction.

As the tilting continued, the load on Column Q3 caused an overturning moment, which would be proportional to the increase of the tilting angle. When the tilting angle reached approximately 3° , the overturning moment of Column Q3 caused the complete fracture of the west anchor bolt, initiating the collapse. During the collapse, the east side of the base plate cut into the top of the CIP pier and crushed the concrete, pushed the east leveling nut down approximately 3" from its original position and compressed the embedded portion of the east anchor bolt further downward. As a result, both the east and the south anchor bolts and the precast column fell along with the crushed concrete pieces to the ground. As shown in Figure 15, both the east and south bolts were still attached to the base plate of Column Q3 after the collapse.

There were shim packs below the base plate of the precast column which would support only a fraction of the compressive load after the threads of the leveling nuts were stripped. The shim pack, however, would not stop the tilting of the column due to its

relatively small size, low stiffness and the location where the shims were placed. In addition, when both Columns Q3 and Q2 tilted in the same direction, the column-to-beam connections failed to provide any significant flexural resistance.

After the collapse of the north precast frame at Grids P/2 through R/3, the line frame at Grids O/2 through O/4, south of the retaining wall and north of the expansion joints, also collapsed within a few seconds. The second collapse was primarily due to the forces pulling the two horizontal braces of the north frame during its collapse and the vibrations caused by the first collapse. The line frame collapsed in the northeast direction.

Discussion of the Wind Effect

At the time of the collapse, the wind gust was approximately 11 miles per hour from the south (Reference 10). The collapsed frame was at the north end of a nearly completed parking structure and the general direction of the collapse was toward the east. Thus, the wind was not a causal factor in the collapse.

Discussion of the Root Cause of the Collapse

Proper Erection Procedures for the Precast Columns

The proper sequence for the erection of the precast columns is described below. The actual method of erection deviated in some critical ways as discussed above.

- Install the leveling nut at each of the three anchor bolts at 6" below the top, and place a washer on the top of each leveling nut. Place two pieces of shim packs on top of the CIP pier.
- Lift the precast column by the crane in a vertical position and lower it until the column base plate rests on top of the washer of the three leveling nuts. In this position, the top portion of the three anchor bolts should be in the open pockets above the steel base plate and each of the six #11 rebars should be inside of the NMB sleeves.

- Adjust the leveling nuts to plumb the column, while the crane still supports the column. Install three washers and top nuts to the anchor bolt in the pockets above the base plate and tighten the top nuts.
- Slack the lifting sling of the crane to gradually transfer the column weight to the three leveling nuts. Readjust the leveling nuts to plumb the column and retighten the top nuts, if required.
- Disconnect the lifting sling from the erected precast column. In this condition, if there are no diagonal bracings provided, any lateral loads on the column, such as wind load must be resisted by the three anchor bolts.
- Prior to erecting precast beams, install 9,000 psi non-shrink grout under the base plate, in the corrugated sleeve around the #11 rebars and in the three pockets on top of the base plate.
- Prior to erecting the upper column, fill the #11 NMB sleeves completely with high strength grout.

Root Cause of the Incident

From the discussion in the above sections, it is apparent that the precast erector did not comply with the required procedures to erect the precast columns. If the precast erector had installed the non-shrink grout below the base plate and grouted the NMB sleeves, the column loads would have been supported by the CIP piers instead of the leveling nuts. The primary cause of the collapse was the lack of grout underneath the column base plates and the lack of sleeve grout around the #11 rebars.

6. CONCLUSIONS

Based on our investigation of the collapsed precast concrete frame, at Grid P/2 through R/3, we conclude that:

1. The partial collapse of the garage under construction occurred due to flawed construction in that the contractor failed to provide proper support for the precast column base plates due to a lack of grout underneath the base plates. As a result,

the loads were transferred to the leveling nuts, stripping their threads. Uneven displacement of the nuts caused the columns to tilt, resulting in the collapse. Although two packs of plastic shims were placed under the column base plates, they were rendered ineffective after the columns were plumbed and the leveling nuts were tightened.

2. The bays that collapsed were essentially in an unstable condition as they were neither braced nor had any flexural capacity at the base because the dowel bars were not grouted to achieve continuity.
3. The Precast Erectors, Inc. did not erect the framing in accordance with the contract documents. Concerning the erection of the framing, the following deficiencies were found after the collapse.
 - The leveling nuts below the base plate of the lower precast column were 0.85" thick, ASTM A563, Grade O, Zinc-coated instead of 1" thick ASTM A563, Grade D material as required. The proof load stress of the leveling nuts was 52 ksi instead of 150 ksi as required.
 - The top nuts above the steel plate (base plate) of the lower precast column were required to be 1½" thick ASTM A563, Grade D material. However, the nuts installed were approximately 1¼" thick ASTM A563, Grade O material. The proof load stress of the nuts was 69 ksi instead of 150 ksi.
 - The west anchor bolt was misaligned when Pier P2 was cast which resulted in an enlarged hole in the column base plate during the erection of the frame. As a result, during the collapse the top nut sheared off due to the oversized hole in the base plate and the anchor bolt did not provide the required strength. Similar enlarged holes were also observed in other base plates in the collapsed area.
 - Between Grid Lines O/2 to P/2 and O/3 to P/3, seven levels of horizontal pipe braces (B-5, adjustable 6" in diameter) along each grid line were required. However, only one level of brace per grid line was installed.

4. The wind was not a causal factor in the collapse.

7. REFERENCES

1. Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary, by American Concrete Institute.
2. Steel Construction Manual, by American Institute of Steel Construction, Inc., Thirteenth Edition, First Print, 2005.
3. ANSI/ASCE 360-05, Specification for Structural Steel Buildings, by American Institute of Steel Construction, Inc., Thirteenth Edition, 2005.
4. Strength Design of Anchorage to Concrete, by Ronald A. Cook, Portland Cement Association, 1999.
5. Steel Design Guide 1, Base Plate and Anchor Rod Design, by James M. Fisher and Lawrence A. Kloiber, American Institute of Steel Construction, Inc., Second Edition, First Print, 2006.
6. An Introduction to the Design and Behavior of Bolted Joints, by John H. Bickford, CRC Press, Taylor & Francis Group, Third Edition, 1995.
7. Introduction to the Design and Behavior of Bolted Joints (Non-Gasketed Joints), by John H. Bickford, CRC Press, Taylor & Francis Group, Fourth Edition, 2008.
8. ASTM Designation: A563 – 07a Standard Specification for Carbon and Alloy Steel Nuts, by American Society for Testing and Materials, 2007.
9. ASTM Designation: F1554 – 07a Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength, by American Society for Testing and Materials, 2007.
10. University Health System, West Parking Garage, Investigation of Structural Collapse, Preliminary Report of Findings, by Matthew P. Carlton and B. Lee Lawrence, Wiss, Janney, Elstner Associates, Inc., for Espey & Associates, P.C., June 16, 2011.



Figure 1. Project Location Plan (Modified from Google Maps).

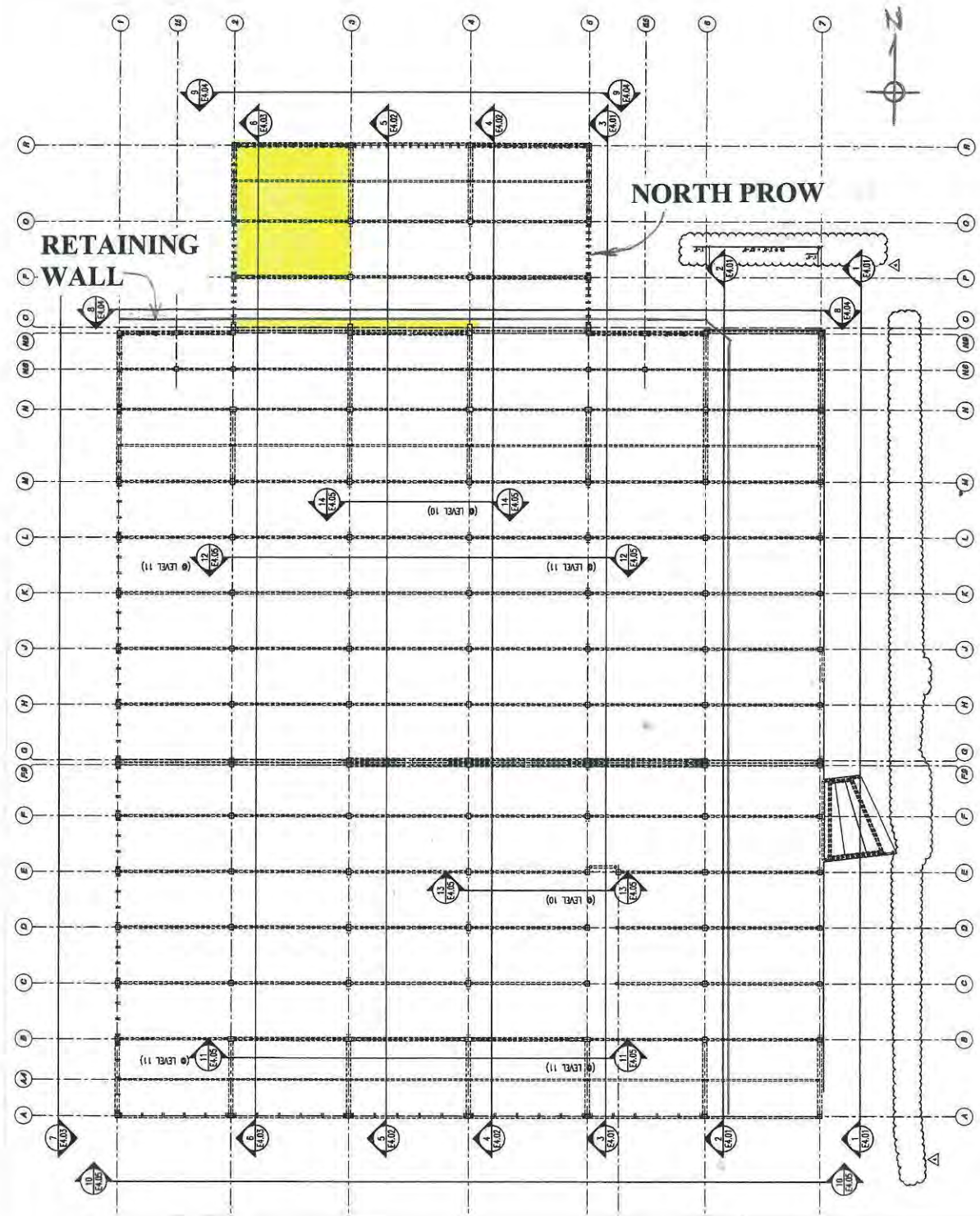


Figure 2. Project Site Plan (Modified from the Precast Erection Drawing Sheet No. E1.00).
Note that the collapsed areas are highlighted.

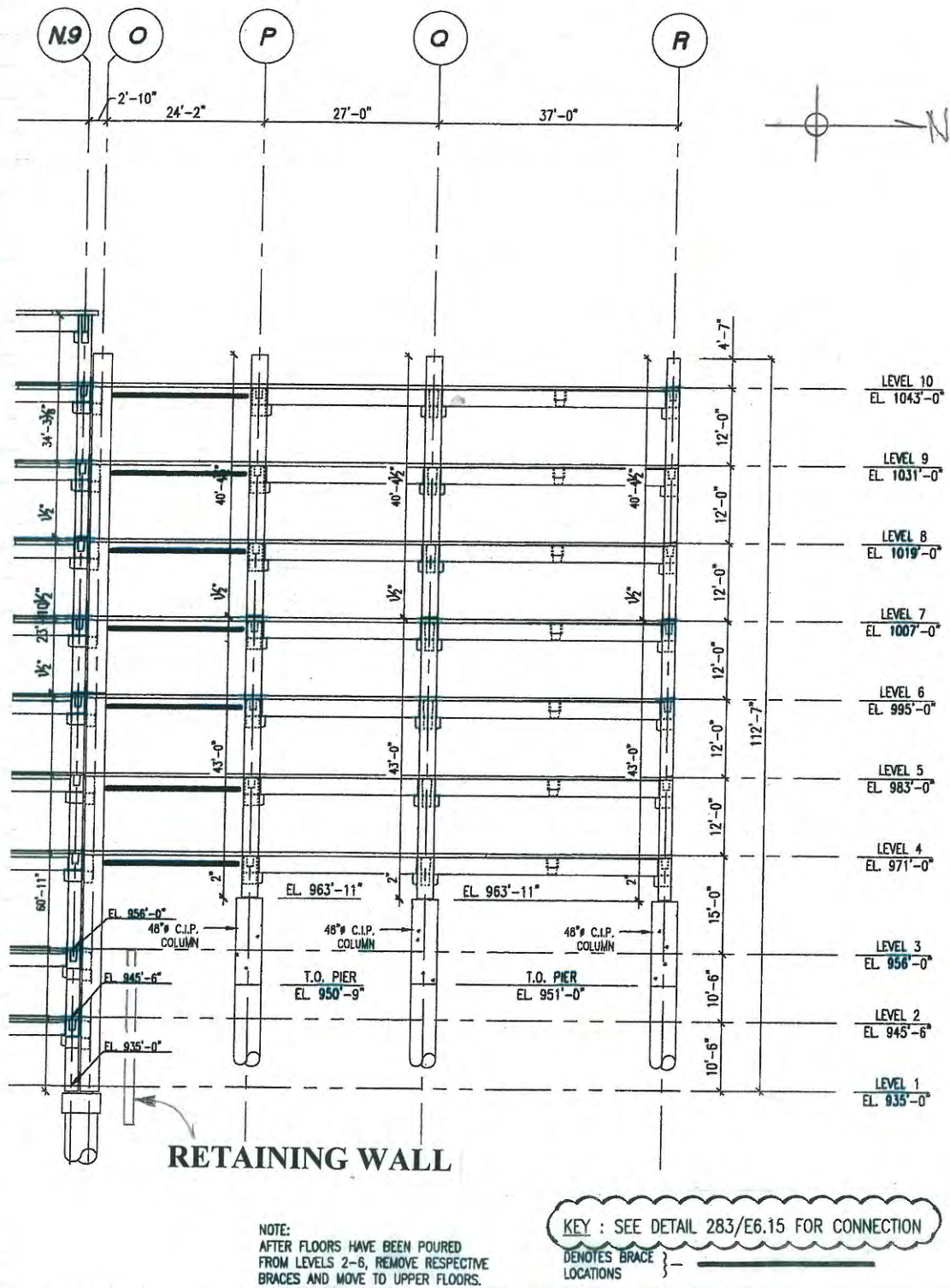


Figure 3. Elevation View of the Parking Structure at the North Prox Area (Modified from the Precast Erection Drawing Sheet No. E4.01B).



Figure 4. Landing Location of Precast Columns [Modified from WJE Preliminary Report (Reference 10)].



Figure 5. Still Photographs from the Video Record (Modified from Project Security Camera, February 14, 2011). Note that A is the upper left photo, B is the upper right, C is the lower left photo and D is the lower right photo.



Figure 6. Failed Conditions on Top of CIP Pier P2.



Figure 7. Failed Conditions at the Bottom of Column P2 (Mark No. C-028).



Figure 8. Failed Conditions on Top of CIP Pier Q2.



Figure 9. Failed Conditions at the Bottom of Column Q2 (Mark No. C-029).



Figure 10. Failed Conditions on Top of CIP Pier R2.



Figure 11. Failed Conditions at the Bottom of Column R2 (Mark No. C-030).
Note that this is the only column that still connected to the pier.



Figure 12. Failed Conditions on Top of CIP Pier P3.



Figure 13. Failed Conditions at the Bottom of Column P3 (Mark No. C-049).
Note that the entire south and east anchorage assemblies were still attached to the base plate.



Figure 14. Failed Conditions on Top of CIP Pier Q3.



Figure 15. Failed Conditions at the Bottom of Column Q3 (Mark No. C-050).
Note that the entire south and east anchorage assemblies were still attached to the base plate.



Figure 16. Failed Conditions on Top of CIP Pier R3.



Figure 17. Failed Conditions at the Bottom of Column R3 (Mark No. C-118).

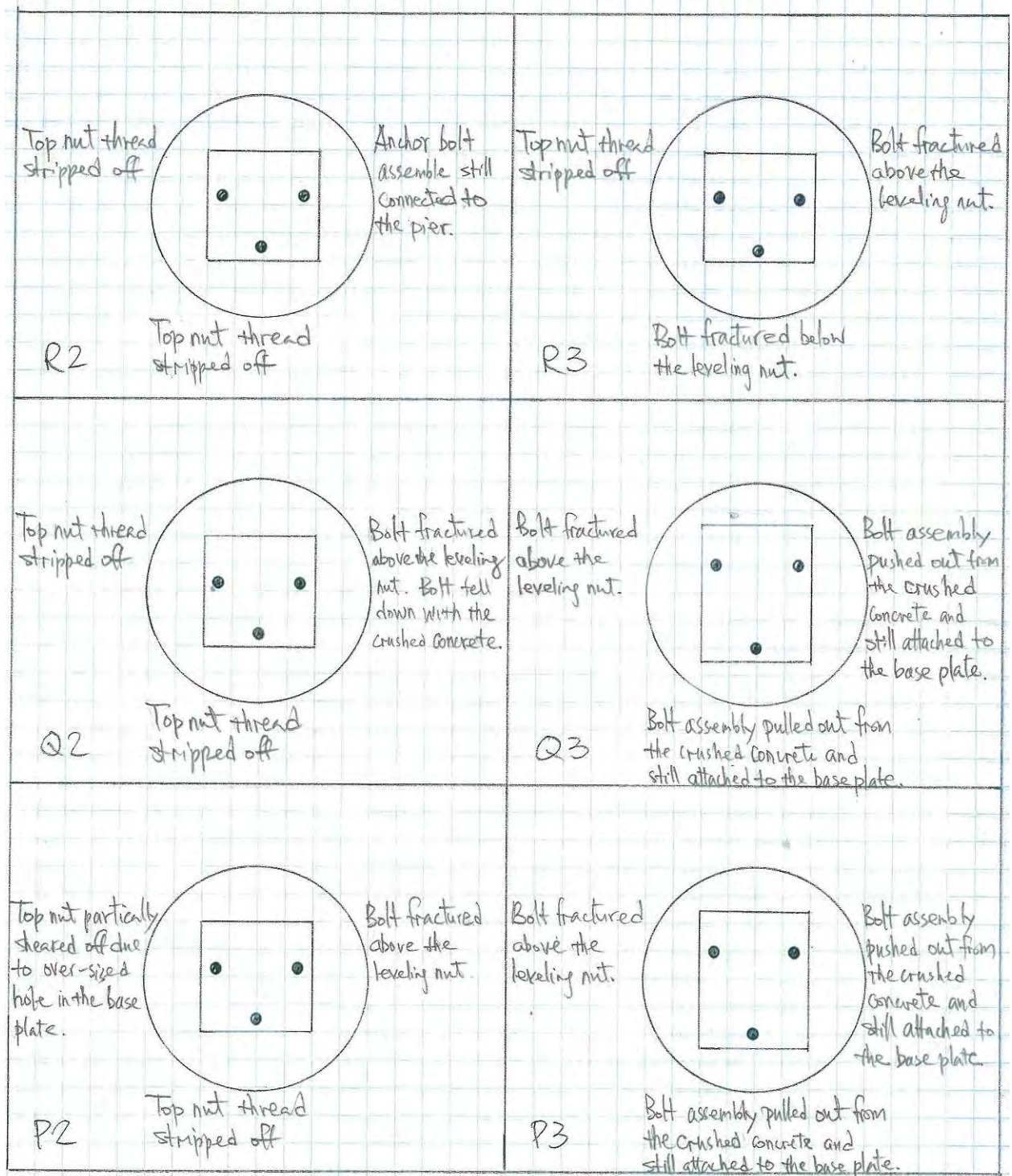


Figure 18. Summary of the Failure Conditions of Column to Pier Connections Grid P/2 through R/3.



Figure 19. Typical Column to Beam Connection.
Note that this photograph was taken at a connection away from the collapse.



Figure 20. Failed Conditions of the Column to Beam Connection.

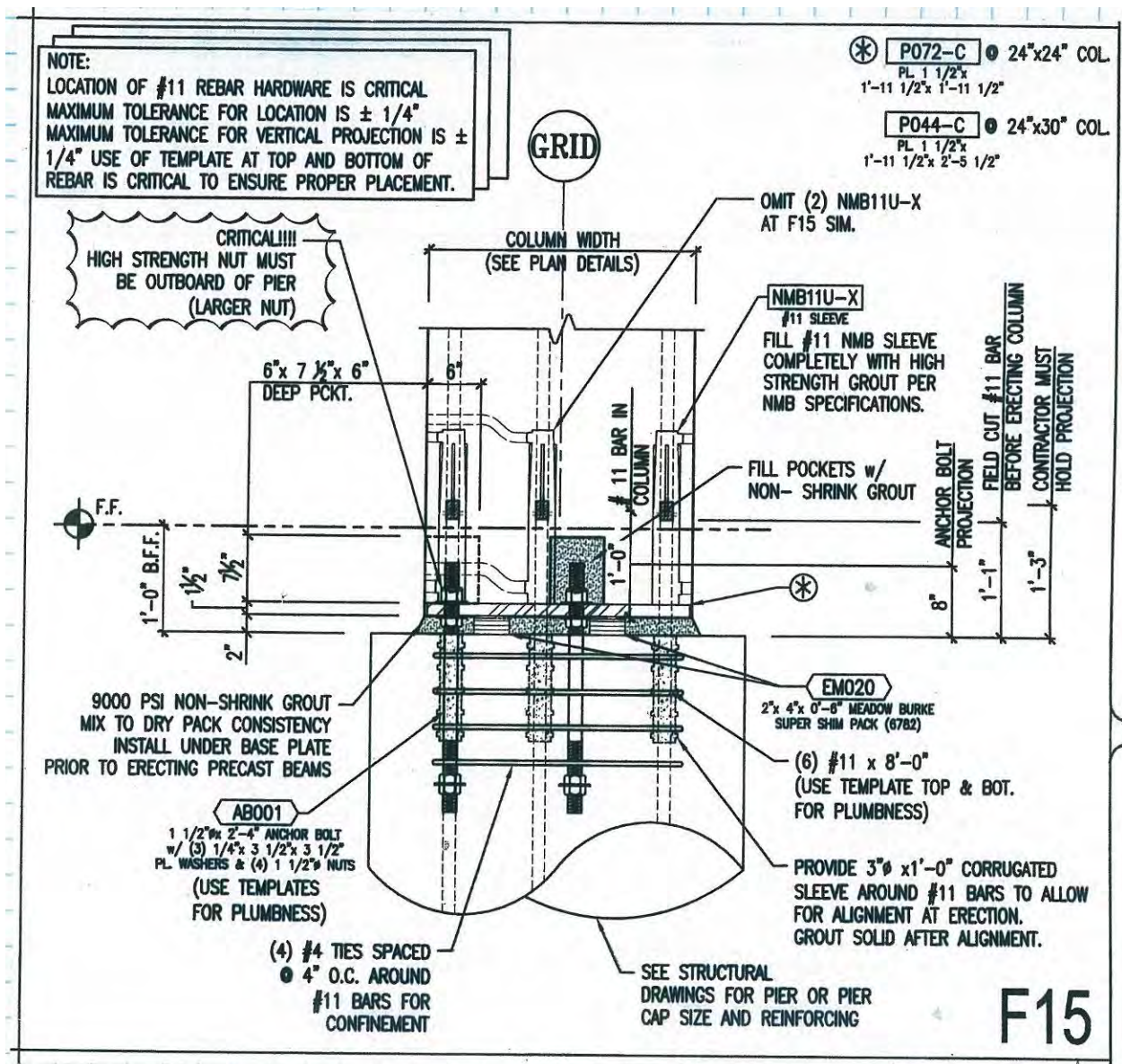


Figure 21. Reinforcements Provided around the Anchorage System in the CIP Piers (Modified from the Precast Erection Drawing Sheet No. E2.02).

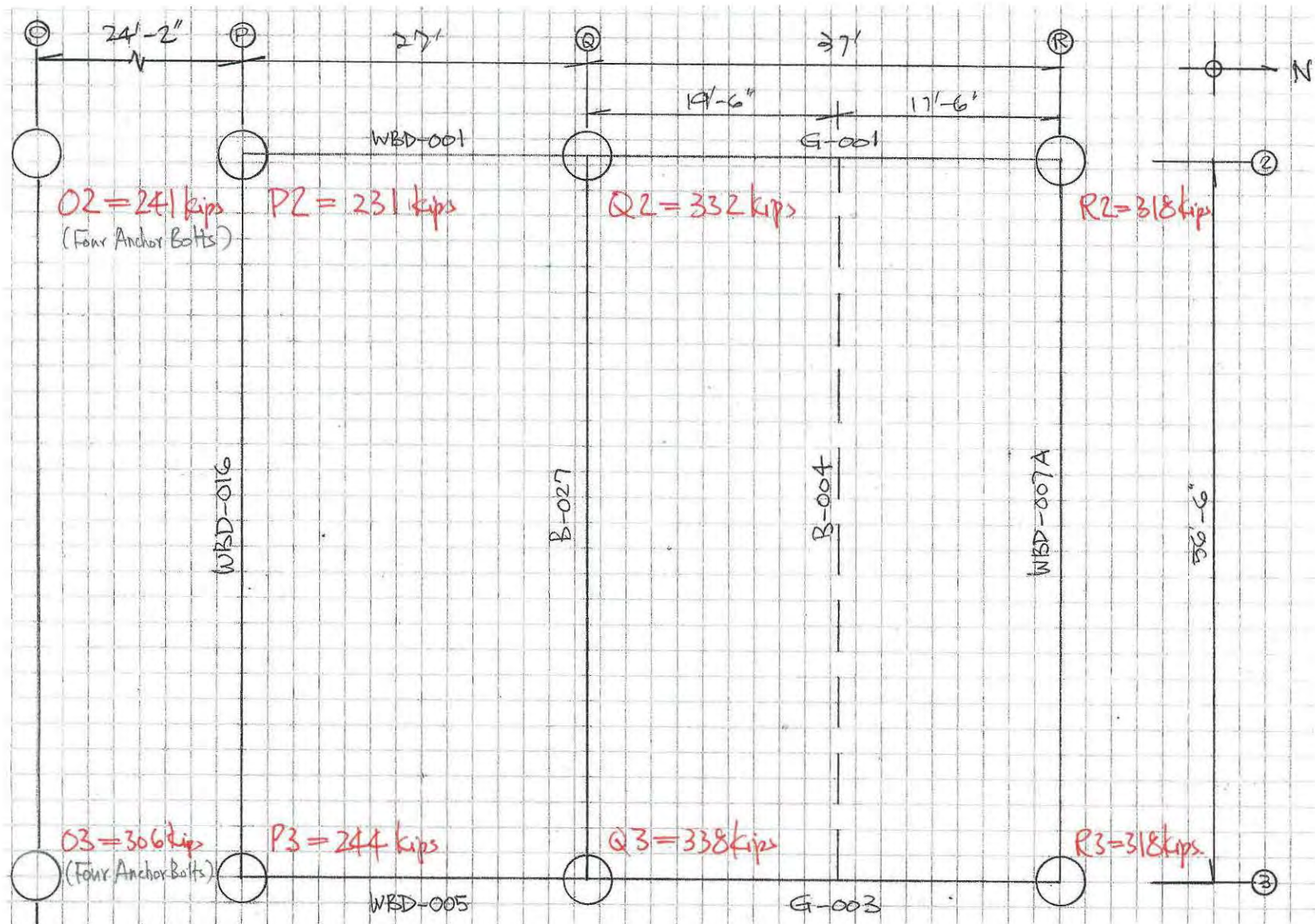


Figure 22. Estimation of the Column Loads at the Time of the Collapse.

APPENDIX A

LABORATORY TEST RESULTS



Report Date: 29 June 2011
Company: Precast Erectors, Inc.
Inspection number: 314305590 (L01065)
Compliance Officer: McAfee (Scott Jin)
Area Office: 0625500
Region: 6
SLTC Staff: Daniel T. Crane

Metallurgical analysis of samples submitted from the collapse of a parking structure Inspection Number 314305590

Two lengths of 1 ½ inch diameter steel threaded anchor bolt, four large nuts and three square washers were submitted for directed analysis to the Salt Lake Technical Center (SLTC). It was requested that we determine compliance with a submitted specification supplied in a mechanical drawing denoted AB001¹. These were received as follows: one anchor bolt with no nuts or washers attached and one anchor bolt with all four nuts and all three washers attached. This is as shown in Photo #1. The case was assigned the laboratory tracking number L01065.



Photo 1: Anchor bolts and fittings as received at SLTC

¹ A copy of AB001 is included with this report for reference.

For reference, the anchor bolt with no nuts or washers was designated anchor bolt #1 and the anchor bolt with the nuts and washers was designated anchor bolt #2. Measurements of tensile strength, steel chemistry and hardness were requested by Scott Jin of OSHA's National Office. This report contains those results.

Figure #1 identifies the nuts and washers for correlation with the analytical results.

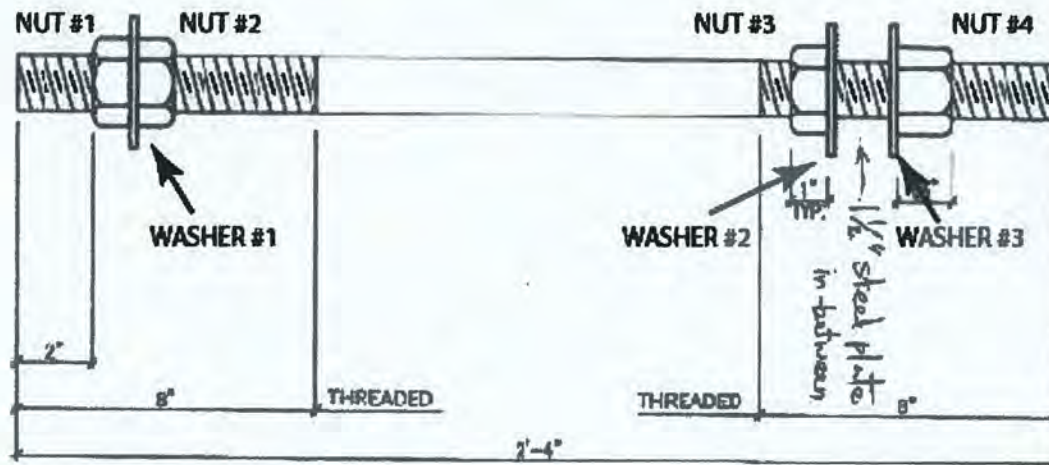


Figure 1: Identification of Nuts and washers on a figure extracted from mechanical drawing AB001 supplied to SLTC by Scott Jin. Note that this is the mechanical drawing. The assembly differed from this in that the threaded end on the left was 5.75 inches on anchor bolt #1 and 5.424 inches on anchor bolt #2. Also, Nut #4 on anchor bolt #2 was 1.276 inches thick and not 1.500 as indicated in AB001.

The anchor bolts were sent to a contract laboratory, American Metallurgical Services, for tensile testing and steel chemistry. Sections of each anchor bolt were removed for tensile testing prior to verification of the overall length, but both bolts appear to have been nominally 28 inches long, consistent with the specification.

When the rods were retrieved from the contract laboratory, the balance of the SLTC examination was performed.

Verification of dimensions

Measurements of the items were performed using calipers and a steel rule as shown in the photos of exemplars below.

Anchor bolt #1

- Length estimated at 28"
- Diameter measured at 1.502"
- Short thread length 5.750"
- Long thread length 8.375"
- Both threads UNS6
- Tensile strength 139,000 psi
- TS is consistent with F1554 Grade 105



Photo 2: Example of bolt diameter measurement

No color or grade marks were seen on the rod.
Rod was corroded
No visible elongation of the threads was observed.

Anchor bolt #2

Length estimated at 28"
Diameter measured at 1.498"
Short thread length 5.424"
Long thread length 8.500"
Both threads UNS6
Tensile strength 152,000 psi
TS is consistent with F1554 Grade 105
No color or grade marks were seen on the rod.
No visible elongation of the threads was observed.

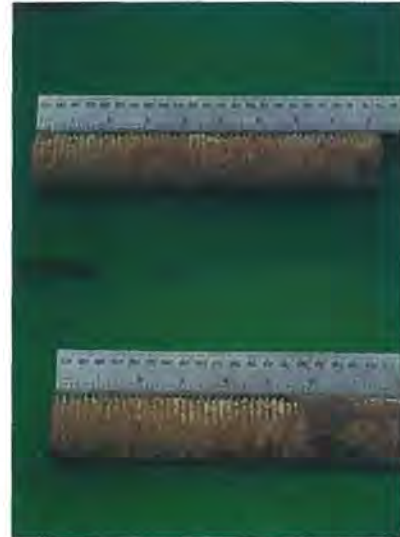


Photo 3: Example of measurement of the thread length

CHEMICAL ANALYSIS:										
Sample	%	C	Si	S	P	Mn	Ni	Cr	Mo	Fe
A		.40	.26	.020	.016	.97	.096	.88	.15	matrix
The above meets 4140 low alloy chemical requirements.										
Sample	%	C	Si	S	P	Mn	Ni	Cr	Mo	Fe
B		.38	.27	.025	.012	.93	.076	.89	.16	matrix
The above meets 4140 low alloy chemical requirements.										

Figure 2 Sample B is Rod #1, Sample A is rod #2 (results from American Metallurgical Services)

TENSILES:									
Sample	in. Dia.	in. ² Area	(lbs.) Tensile Load	(psi) Tensile Strength	(lbs.) Yield Load	(psi) Yield Strength	2" Elong.	RA	
A	.502	.1979	30,035	152,000	26,645	135,000	17%	50%	
B	.501	.1971	27,345	139,000	23,958	122,000	18%	52%	

Figure 3: Sample B is Rod #1, Sample A is rod #2 (results from American Metallurgical Services)

Nut measurements

Nut #1

1 ½" Hex Nut

Flat diameter: 2.200" nom

Across corners diameter: 2.530" nom

Thickness: 1.344" nom

UNC 6

No apparent stripping

Hardness HRB 58

No grade markings present on the nut

Consistent with standard hex nut (A563) B18.2.2. type O

Nut #2

1 ½" Hex Nut

Flat diameter: 2.200" nom

Across corners diameter: 2.520" nom

Thickness: 1.296" nom

UNC 6

No apparent stripping

Hardness **HRB 54**

The hardness of this nut was below the minimum specified (HRB =55) for any listed hex nut in ASTM A563

No grade markings present on the nut

Nut #3

1 ½" Zn coated Hex Jam Nut

Flat diameter: 2.261" nom

Across corners diameter: 2.554" nom

Thickness²: 0.853" nom

UNC 6 probable, but undetermined (The nut was not removed from the rod)

Apparent internal thread damage. Outer thread separated from nut body at thread root on side nearest end of the rod

Hardness HRB 60

No grade markings present on the nut

Consistent with standard hex nut (A563) B18.2.2. Type O



Photo 4: Nut # 3 -- Note the threads separated from the nut at the root of the thread where the rod enters the nut.

² A re-measurement of Nut # 3 was requested and is included in the next section. This measurement reflects that value.

Nut #4

1 ½" Hex Nut

Flat diameter: 2.222" nom

Across corners diameter: 2.543" nom

Thickness: 1.276" nom

UNC 6

No apparent stripping

Hardness HRB59

No grade markings present on the nut

Consistent with standard nut (A563) B18.2.2. type O

Square Washer #1

Heavily bent, but measured to be approximately 3.54" square with a 1.62" diameter hole and a nominal thickness of 0.25"

Square Washer #2

3.50" x 3.54"

Hole diameter 1.73"

Thickness 0.25" nom

Square Washer #3

3.51" x 3.56"

Hole diameter 1.73"

Thickness 0.25" nom

The nuts met the requirements of standard hex nuts for Grade 0 A563. None of the nuts met the criteria set for A563 Heavy Hex Nuts. Note that the proof load stress for Grade 0 nuts is 69,000 psi and the proof load stress for Grade A nuts is 90,000 psi.

Of particular note is that Hex Nut 4 was specified to be a 1 ½ thick, 1 ½ High Strength Hex nut. It was 1.2760" thick and not 1 ½". It does not meet the standard of a Heavy Hex nut per (A563) B18.2.2

Also, the drawing (AB001) indicated that the end of the rod on which nuts 1 and 2 are threaded should be threaded for 8 inches. The threading was measured to be between 5.424" and 5.750".

Requested re-measurement of Nut #3

After the preliminary report, Scott Jin requested a re-measurement of nut #3. Here are the measurements as requested. The thickness of the nut was measured at each face as close to the threads as possible. Included here is a photo of the measurement at face #1. There is a little dirt on the nut and a camber near the edge which accounts for some uncertainty in the measurement.

Precast Erectors -- 314305590	
6/20/2011	
Thickness measurements for Nut #3	
Face	Thickness (inches)
1	0.8545
2	0.8585
3	0.8535
4	0.8560
5	0.8515
6	0.8420
Average	0.8527
STDEV	0.0075

Figure 4: Re-measurement of Nut #3



Photo 5: Example of measurement of the thickness of nut #3 at face #1

Hardness testing by ZWICK 2.5N

The hardness values are Rockwell B (HRB)

Nut # 1

Series n = 3	HRB	h Pre-load μm	h FR Pre-load μm	h μm
x	58.43	16.589	99.723	83.13
s	2.86	0.525	6.012	5.71
v	4.89	3.16	6.03	6.87

Nut # 2

Series n = 3	HRB	h Pre-load μm	h FR Pre-load μm	h μm
x	54.11	22.320	114.107	91.79
s	0.79	2.467	1.676	1.57
v	1.45	11.05	1.47	1.71

Nut #3

Series n = 3	HRB	h Pre-load μm	h FR Pre-load μm	h μm
x	59.64	16.504	97.223	80.72
s	2.80	2.666	7.597	5.59
v	4.69	16.15	7.81	6.93

Nut #4

Series n = 3	HRB	h Pre-load μm	h FR Pre-load μm	h μm
x	59.32	17.371	98.731	81.36
s	0.40	0.769	0.336	0.80
v	0.67	4.43	0.34	0.98

Nuts 1, 3 and 4 meet the minimum hardness for Grade O standard Hex nuts (Rockwell minimum is B55). The hardness of Hex nut 2 HRB 54, which is below the minimum allowed hardness for any grade in A563 18.2.2

The minimum for Grade A is HRB68. None of nuts had hardness exceeding HRB 68.

Conclusion

The Anchor Bolts were consistent with ASTM F1554 Grade 105 bolt standard, but were not threaded according to drawing AB001, in that the threads on one end were only nominally 5.5 inches where it was specified to be 8 inches.

Hex nuts #1 and #4 were A563 standard 1 ½ inch hex nuts. Hex nut #3 was a Zn-coated standard 1 ½ inch jam nut with a thickness of 0.85 inch (nom). The specification was 1 inch thick. ASTM

Drawing AB001 is attached below for reference.)

Figure 5: Drawing AB001 provided to SLTC by Scott Jin

APPENDIX B

KEY CONSTRUCTION DOCUMENTS

GENERAL NOTES:

THESE GENERAL NOTES ARE TO BE USED IN CONJUNCTION WITH THE STRUCTURAL DRAWINGS, NOTES AND SPECIFICATIONS PROVIDED BY THE ENGINEER-OF-RECORD.

G-1 THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS AND CONDITIONS AT THE JOB SITE PRIOR TO STARTING CONSTRUCTION AND REPORT ANY DISCREPANCIES OR INCONSISTENCIES TO THE ARCHITECT/ENGINEER.

G-2 ALL PHASES OF WORK SHALL CONFORM TO THE MINIMUM STANDARDS OF THE LATEST EDITION OF THE INTERNATIONAL BUILDING CODE. ALL ASTM SPECIFICATIONS NOTED ON THESE DRAWINGS SHALL BE OF THE LATEST REVISION.

G-3 SEE ARCHITECTURAL DRAWINGS FOR FLOOR ELEVATIONS, SLOPE, AND LOCATION OF DEPRESSED AND ELEVATED FLOOR AREAS. THE CONTRACTOR SHALL COMPARE THE STRUCTURAL SECTIONS WITH THE ARCHITECTURAL SECTIONS AND REPORT ANY DISCREPANCIES TO THE ARCHITECT PRIOR TO PERFORMING THE WORK.

G-4 COMBINING ALL CONSTRUCTION DOCUMENTS WITH THE STRUCTURAL DOCUMENTS DEFINES THE TOTAL PROJECT. THE STRUCTURAL DOCUMENTS REPRESENT THE FINISHED STRUCTURE AND DO NOT INDICATE THE MEANS OR METHODS OF CONSTRUCTION. VERIFY ALL FIELD CONDITIONS THAT AFFECT NEW CONSTRUCTION BEFORE STARTING CONSTRUCTION.

G-5 THE CONTRACTOR IS RESPONSIBLE FOR SAFETY PRECAUTIONS AND PROGRAMS IN CONNECTION WITH THE WORK THAT CONFORMS WITH REGULATIONS OF THE OCCUPATIONAL SAFETY AND HEALTH ADMINISTRATION (OSHA) SAFETY AND HEALTH STANDARDS FOR THE CONSTRUCTION INDUSTRY.

G-6 CONSTRUCTION MATERIAL SHALL BE SPREAD OUT IF PLACED ON FRAMED FLOORS OR ROOF. LOAD SHALL NOT EXCEED THE DESIGN LIVE PER SQUARE FOOT.

G-7 ESTABLISH AND VERIFY ALL OPENINGS AND INSERTS FOR ARCHITECTURAL, MECHANICAL, ELECTRICAL AND PLUMBING WITH APPROPRIATE TRADES, DRAWINGS AND SUBCONTRACTORS PRIOR TO CONSTRUCTION. DO NOT PENETRATE ANY STRUCTURAL ELEMENTS (BEAMS, COLUMNS, WALLS, SLABS, STEEL DECKS, ETC.) WITHOUT PRIOR WRITTEN APPROVAL OF STRUCTURAL ENGINEER.

G-8 NOTES AND DETAILS ON DRAWINGS SHALL TAKE PRECEDENCE OVER GENERAL STRUCTURAL NOTES AND TYPICAL DETAILS. WHERE NO SPECIFIC DETAILS ARE SHOWN, CONSTRUCTION SHALL CONFORM TO SIMILAR WORK ON THE PROJECT.

G-9 TAKE ALL MEASURES NECESSARY TO PROTECT THE SAFETY OF THE PUBLIC ALONG WITH THE SAFETY OF THE STRUCTURE DURING CONSTRUCTION. SUCH MEASURES SHALL INCLUDE BUT NOT BE LIMITED TO BRACING AND SHORING OF DEAD LOADS, CONSTRUCTION LOADS AND WIND LOADS. CORRECT AT OWN EXPENSE ANY SUBSEQUENT STRUCTURAL DAMAGE OR OTHER OBJECTIONABLE CONDITIONS CAUSED BY YOUR OPERATIONS.

CODE AND DESIGN SPECIFICATIONS:

BUILDING CODE: 2009 INTERNATIONAL BUILDING CODE (IBC)

LOADS	: ASCE7-08, "MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES"
STRUCTURAL CONCRETE	: ACI 301-08, "STANDARD SPECIFICATIONS FOR STRUCTURAL CONCRETE"
	: ACI 318-08, "BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE"
STRUCTURAL STEEL	: ANSI/AISC 360-05, "SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS"
	: "AISC STEEL CONSTRUCTION MANUAL" 13TH EDITION
MASONRY	: ACI 530-08, "BUILDING CODE REQUIREMENTS FOR MASONRY STRUCTURES"
PRECAST CONCRETE	: PCI MNL 120-08, "DESIGN HANDBOOK FOR PRECAST AND PRESTRESSED CONCRETE"
POST-TENSIONING	: PTI SIXTH EDITION, "POST-TENSIONING MANUAL"
WELDING	: AWS D1.1 AND AWS D1.3, "STRUCTURAL WELDING CODE", LATEST EDITION.

DESIGN LOADS:

THE ELEVATED DECK SYSTEM CONSISTING OF PRECAST BEAMS AND COLUMNS AND POST-TENSIONED SLAB IS DESIGNED FOR THE FOLLOWING LOADS. ANY ADDITIONAL LOADS IMPLIED ON THE STRUCTURE HAVE TO BE APPROVED IN WRITING BY CONSULTING ENGINEER'S GROUP, INC. BEFORE PLACEMENT OF SUCH LOAD AND A 48 HOUR PRIOR NOTICE IS REQUIRED.

DEAD LOAD

SELFWEIGHT OF STRUCTURE..... AS APPLICABLE

SUPERIMPOSED DEAD LOAD..... 10 PSF

MECHANICAL UNITS..... NONE (ANY UNITS WEIGHING UPTO 40 PSF [TOTAL WEIGHT/BASE AREA] ARE PRE-APPROVED)

LIVE LOADS

TYPICAL LEVELS

PARKING LOAD 40 PSF (REDUCIBLE TO COLUMNS PER CODE)

CONCENTRATED WHEEL LOAD..... 3000 LBS (PLACED PER CODE)

STAIRS & LOBBY AREAS..... 100 PSF

ROOF LEVEL

STAIR & ELEVATOR ROOF..... 20 PSF

THE VEHICLE BARRIER IN PARKING GARAGES IS DESIGNED TO RESIST A HORIZONTAL LOAD OF 6,000 LBS AS PER CODE

ERECTION NOTES:

E-1 ACCESS TO AND ON THE JOB SITE SHALL BE MAINTAINED BY THE GENERAL CONTRACTOR AT ALL TIMES, SO THAT ERECTION AND DELIVERY EQUIPMENT CAN MOVE UNDER THEIR OWN POWER AND WORK UNINTERRUPTED.

E-2 DRY PACK BETWEEN COLUMN AND FOUNDATION SHALL HAVE A MINIMUM STRENGTH OF 9000 PSI. DRY PACKING SHALL BE DONE IMMEDIATELY AFTER ERECTION OF COLUMNS. NO MORE THAN 2 LEVELS MAY BE ERECTED BEFORE COLUMNS ARE FULLY GROUTED.

E-3 ALL PIECES SHALL BE ERECTED TO PCI TOLERANCES. BEARING PADS SHALL BE PLACED SQUARE AND FLUSH WITH THE ENDS OF BEAMS. WHERE NECESSARY, THE PRECAST ERECTOR SHALL SHIM BEAM BEARING CONDITIONS TO BRING OFFSETS WITHIN TOLERANCES. SHIM HEIGHTS GREATER THAN 2" SHALL BE APPROVED BY CONSULTING ENGINEERS GROUP.

E-4 STABILITY OF STRUCTURE SHALL BE MAINTAINED AT ALL TIMES UNTIL ALL CONNECTIONS ARE COMPLETED.

CONCRETE FOR ELEVATED SLABS:

C-1 ALL REINFORCING BARS SHALL CONFORM TO ASTM A-615, GRADE 60 EXCEPT WHERE NOTED. ALL REINFORCING BARS WELDED TO A STEEL SECTION SHOULD BE OF WELDABLE ASTM A-706, GRADE 60. ALL WELDED WIRE FABRIC SHALL CONFORM TO ASTM A-82 AND A-185 SUPPLIED IN FLAT SHEETS. REINFORCING STEEL SHALL BE CONTINUOUS WITH SPLICES LAPPED AT LEAST 40 DIAMETERS. STIRRUPS AND TIES SHALL BE GRADE 60 FOR BARS #3 AND SMALLER.

C-2 FABRICATE BENT BARS ACCORDING TO ACI 315. INSTALL REINFORCING WITH CLEARANCE FOR CONCRETE COVERAGE AROUND REINFORCING STEEL ACCORDING TO ACI 318. SUBMIT FOR REVIEW FABRICATION AND PLACEMENT SHOP DRAWINGS INDICATING BAR SIZES, SPACINGS, LENGTHS, LAPS, LOCATIONS, AND QUANTITIES OF REINFORCING STEEL, BENDING AND CUTTING SCHEDULES, AND SUPPORTING AND SPACING DEVICES. NOTIFY THE CITY BUILDING OFFICIAL, THE SPECIAL INSPECTOR AND CONSULTING ENGINEERS GROUP, INC. AT LEAST 48 HOURS IN ADVANCE TO REVIEW THE ELEVATED SLAB CONSTRUCTION BEFORE CONCRETE PLACEMENT.

C-3 CONCRETE SHALL DEVELOP A 28-DAY COMPRESSIVE STRESS (F'C) OF AT LEAST 5,000 PSI FOR ELEVATED SLABS, MIX CONCRETE ACCORDING TO ACI 301. USE A MAXIMUM AGGREGATE SIZE OF 1 1/8", OR ACCORDING TO ACI 318. MAXIMUM AGGREGATE SIZE BETWEEN BARS SHALL ALSO PERTAIN TO BETWEEN THE FORMS AND BARS.

C-4 THE PROPORTIONS OF MATERIALS AND USE OF ADMIXTURES INFLUENCE THE CONCRETE STRENGTH ALONG WITH THE MEANS AND METHODS OF CONSTRUCTION. THE CONTRACTOR IS RESPONSIBLE TO DETERMINE THAT THE CONCRETE IS SUITABLE FOR ITS INTENDED PURPOSE. THE ENGINEER RECOMMENDS THE CONTRACTOR CONSIDER THE FOLLOWING IN DETERMINING THE CONCRETE FOR THIS PROJECT: CEMENT SHALL BE TYPE 1 (GRAY). FLY ASH SHALL BE BORAL MATERIALS, CLASS C. IF FLY ASH IS USED, DO NOT EXCEED 20% OF THE TOTAL FLY ASH AND CEMENT USED BY WEIGHT. INCLUDE A POLYMERIC COMPOUND WATER-REDUCING ADMIXTURE THAT COMPLIES WITH ASTM C494. MIX SHALL RESULT IN A FINISHED CONCRETE PRODUCT WITH MOISTURE CONTENTS NECESSARY TO PROPERLY CURE THE CONCRETE.

C-5 BEFORE PLACEMENT OF ANY CONCRETE, SUBMIT CONCRETE MIX DESIGN(S) TO BE USED ON THE PROJECT. CONCRETE SHALL BE IN STRICT ACCORDANCE WITH THE MIX DESIGN. THE MAXIMUM AGE OF CONSECUTIVE TEST REPORTS SHALL BE ONE YEAR WHEN USED TO PROVIDE HISTORICAL DATA FOR THE EXPERIENCE METHOD OF THE MIX PROPORTION SELECTION.

C-6 PLACE AND CURE CONCRETE ACCORDING TO ACI 302.1R. FINISH ACCORDING TO ACI 117 TOLERANCES. IF CONCRETE IS PUMPED, PROVIDE HOSES OR OTHER SUITABLE MEANS TO SUPPORT THE HOSE SO THAT IT DOES NOT RIDE ON THE TENDONS OR ON THE MILD REINFORCING STEEL.

C-7 NOTIFY CERTIFIED TECHNICIANS ACCORDING TO ACI 301 TO MONITOR AND TEST CONCRETE ACCORDING TO ACI 311.5R. BATCH PLANT INSPECTION IS NOT REQUIRED. TEST ACCORDING TO FREQUENCY REQUIREMENTS IN ACI 318, SECTION 5.6.2.1. THE MINIMUM SAMPLING FREQUENCY SHALL BE ONCE EACH DAY OR ONCE FOR EACH 150 CU.YD PLACED OR EACH 5000 SQ.FT OF SLAB SURFACE AREA; WHICHEVER IS LEAST. TEST NUMBER OF TEST SPECIMENS ACCORDING TO ACI 311.5R, SECTION 2.4.13. REJECT OR ACCEPT CONCRETE BASED ON THE RESULTS OF TESTS. REPORT ALL TESTING PROMPTLY.

C-8 SUBMIT A WRITTEN STATEMENT WITH A COPY DIRECTLY TO THE STRUCTURAL ENGINEER AT THE COMPLETION OF THE PART OF THE PROJECT SUMMARIZING THE TESTS/INSPECTIONS PERFORMED AND THE COMPLIANCE OF THE TEST RESULTS/ITEMS INSPECTED WITH THE SPECIFIED REQUIREMENTS.

C-9 ALL CONSTRUCTION JOINTS SHALL BE 3/16" WIDE SAW CUT JOINTS. JOINTS SHALL BE CUT 4 TO 8 HOURS AFTER CONCRETE IS SET AND SHALL BE A MINIMUM OF AT LEAST 1/4 OF THE SLAB THICKNESS IN DEPTH. REINFORCEMENT SHALL BE CONTINUOUS THROUGH SAWED JOINTS. FILL SAW CUTS WITH EUCO 700 SEMI-RIGID INDUSTRIAL FLOOR JOINT FILLER AS MANUFACTURED BY THE EUCLID CHEMICAL COMPANY. FOLLOW MANUFACTURERS RECOMMENDATIONS AND DIRECTIONS FOR APPLICATION OF PRODUCT.

PRECAST CONCRETE :

P-1 FABRICATOR SHALL KEEP RECORDS OF STRESSING FORCES, ELONGATIONS, CONCRETE CYLINDER BREAKS, AND SLUMP OF CONCRETE FOR EACH DAYS POUR EACH TYPE OF UNIT AND SEND COPIES TO THE ARCHITECT.

P-2 THE PRECAST ERECTOR IS COMPLETELY AND SOLELY RESPONSIBLE FOR THE MEANS AND METHODS OF THE ERECTION OF ALL PRECAST PRODUCT SHOWN IN THESE DRAWINGS INCLUDING, BUT NOT LIMITED TO, THE ERECTION SEQUENCING AND THE DESIGN AND DETAILING OF ANY AND ALL TEMPORARY GUYING AND BRACING FOR THE PRECAST MEMBERS AND STRUCTURE, UNLESS NOTED OTHERWISE HEREIN.

P-3 PRIOR TO BEGINNING ERECTION, CONTRACTOR/ ERECTOR SHALL FIELD SURVEY THE LOCATIONS OF ALL ANCHOR BOLTS, DOWELS, SLEEVES, INSERTS, AND EMBEDDED HARDWARE TO VERIFY THAT THEIR LOCATION IS WITHIN PCI TOLERANCES. SUCH SURVEY SHALL CONTINUE AS ERECTION PROGRESSES TO ENSURE COMPLIANCE WITH ERECTION TOLERANCES. ANY DISCREPANCIES SHALL BE IMMEDIATELY REPORTED TO CONSULTING ENGINEERS GROUP BEFORE ERECTION.

P-4 ANY DEVIATION FROM PRECAST CONCRETE DESIGN OR DETAIL SHOWN HEREIN SHALL BE APPROVED, IN WRITING, BY THE ARCHITECT. CONNECTION DETAILS ARE NOT EXCLUSIVE AND MAY BE ALTERED BY PRECASTER TO SUIT HIS STANDARD OF SUGGESTED DETAILS, PROVIDED THAT THESE STANDARDS SATISFY THE STRENGTH REQUIREMENTS OF THE PARTICULAR CONNECTION AND RECEIVE THE STRUCTURAL ENGINEER'S APPROVAL PRIOR TO CASTING.

P-5 CONNECTIONS ARE TO COMPLETED AS ERECTION PROGRESSES UNLESS ADEQUATE MEASURES AND TAKEN BY THE PRECAST ERECTOR. PRECAST ERECTOR SHALL BE SOLELY RESPONSIBLE FOR COMPLETE ERECTION OF PRECAST CONCRETE ELEMENTS, INCLUDING BRACING, LEVELING, WELDING, BOLTING, ETC. ALL FABRICATION AND ERECTION SHALL COMPLY WITH APPROPRIATE PCI TOLERANCES.

P-6 COLUMN ANCHOR BOLTS ARE DESIGNED TO SUPPORT UNBRACED COLUMNS DURING ERECTION, UNLESS NOTED OTHERWISE. COLUMN BASE PLATES SHALL BE FULLY GROUTED AS SOON AS PRACTICAL AFTER THE INSTALLATION OF THE COLUMN. AT THE LATEST, GROUTING SHALL BE COMPLETED BY THE END OF THE DAY THE COLUMN IS SET.

P-7 PRECAST MANUFACTURER SHALL PROVIDE ONLY THOSE OPENINGS AND SAW CUTS SHOWN ON APPROVED SHOP DRAWINGS, ALL OTHER HOLES WILL BE CUT BY THE RESPECTIVE TRADES IN THE FIELD. HOLES WHICH WILL APPARENTLY CUT PRIMARY REINFORCING IN MEMBERS SHALL BE APPROVED BY THE PRECAST SPECIALTY ENGINEER PRIOR TO CUTTING IN THE FIELD. CUTTING THE PRESTRESSED REINFORCING IS NOT PERMITTED UNLESS APPROVED BY THE PRECAST SPECIALTY ENGINEER OR UNLESS SPECIFICALLY NOTED OTHERWISE.

P-8 PRECAST PRODUCTS WILL BE FABRICATED TO TOLERANCES SPECIFIED IN SECTION 5 OF PCI MANUAL - 116 "MANUAL FOR QUALITY CONTROL FOR PLANTS AND PRODUCTION OF PRECAST/PRESTRESSED CONCRETE PRODUCTS." PRECAST PLANT SHALL BE PCI CERTIFIED BUT IN THE ABSENCE OF A PCI PLANT CERTIFICATION, A QUALITY CONTROL MANUAL SHALL BE SUBMITTED BY THE PRECASTER TO THE ENGINEER BEFORE CASTING ANY PIECES.

P-9 ALL MEMBERS SHALL HAVE MINIMUM $f_c = 5000$ PSI (28 DAY). MINIMUM CONCRETE COVER FOR REINFORCEMENT FOR PRECAST CONCRETE SHALL BE AS STATED IN ACI 318, UNLESS DETAILED OTHERWISE.

P-10 WELDING IS TO BE PERFORMED PER AWS RECOMMENDED PRACTICE. WELDERS MUST BE AWS CERTIFIED. WELD ELECTRODES SHALL BE E70 UNLESS NOTED OTHERWISE.

P-11 CONNECTIONS REQUIRING PATCHING SHALL BE PATCHED WITH SUITABLE MATERIAL TO REASONABLY MATCH THE ADJACENT CONCRETE. ALL STEEL PLATES EXPOSED TO WEATHER SHALL BE GALVANIZED.

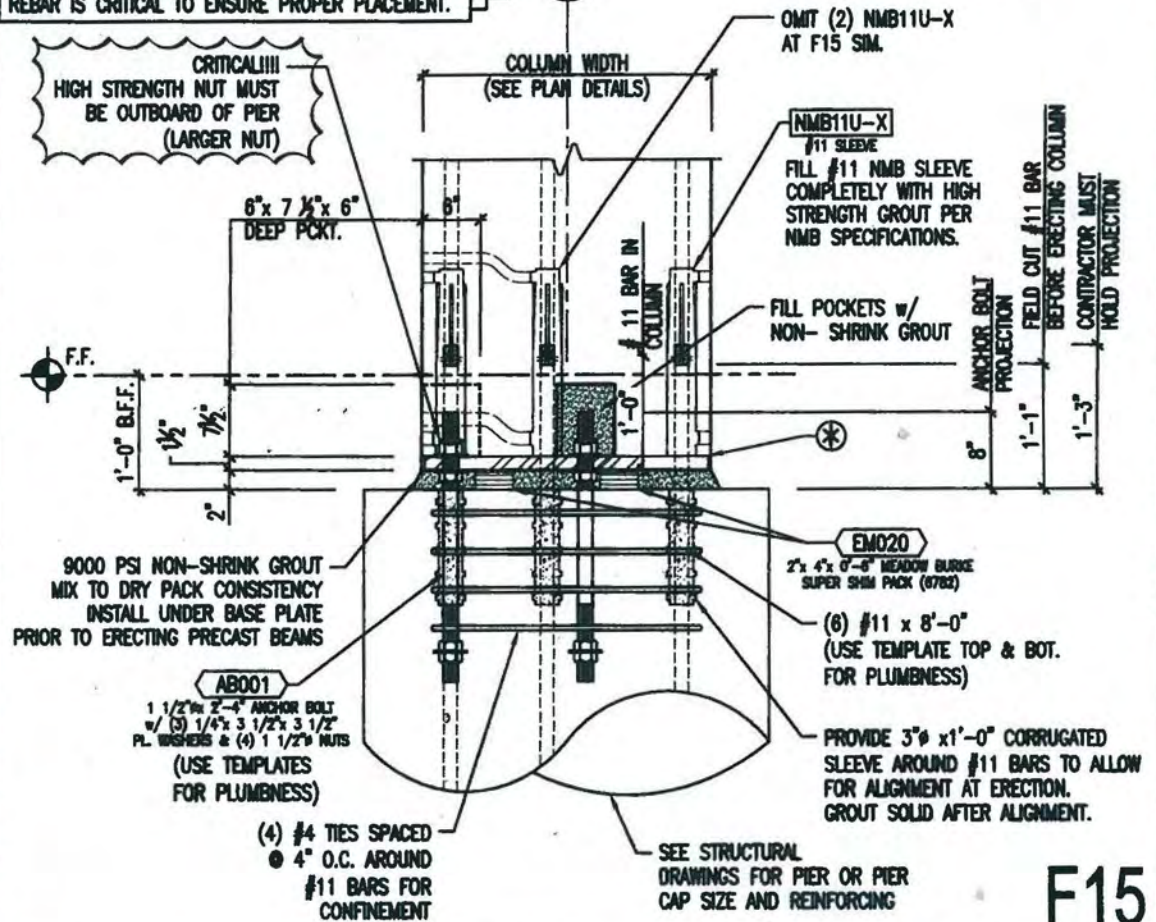
P-12 EXPANSION AND/OR ADHESIVE ANCHOR INSTALLATION AND HOLE PREPARATION SHALL BE PER MANUFACTURER SPECIFICATIONS. GROUT, WHERE SHOWN SHALL BE 6000 psi NON-SHRINK, NON-METALLIC SEALANT, WHERE SHOWN IN THESE DRAWINGS, SHALL BE THAT APPROVED BY THE ARCHITECT AND COORDINATED THROUGH THE CONSTRUCTION MANAGER. EPOXY, WHERE SHOWN IN THESE DRAWINGS, SHALL BE HILTI HY150 (OR APPROVED EQUIVALENT).



F11

NOTE:
LOCATION OF #11 REBAR HARDWARE IS CRITICAL
MAXIMUM TOLERANCE FOR LOCATION IS $\pm 1/4"$
MAXIMUM TOLERANCE FOR VERTICAL PROJECTION IS $\pm 1/4"$
USE OF TEMPLATE AT TOP AND BOTTOM OF
REBAR IS CRITICAL TO ENSURE PROPER PLACEMENT.

CRITICAL!!!
HIGH STRENGTH NUT MUST
BE OUTBOARD OF PIER
(LARGER NUT)



* P072-C @ 24"x24" COL.
PL. 1 1/2" x 1'-11 1/2" x 1'-11 1/2"

P044-C @ 24"x30" COL.
PL. 1 1/2" x 1'-11 1/2" x 2'-5 1/2"

F15

