Investigation of the December 6, 2007 Fatal Parking Garage Collapse at Berkman Plaza 2, Jacksonville, FL

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REPORT

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REPORT

On December 6, 2007 at approximately 6:15 a.m. an incident occurred during the construction of a five-story concrete parking garage in downtown Jacksonville, FL. The location is 500 East Bay Street, Jacksonville, FL. One construction employee was killed and twenty-one injured. The incident occurred when the 6^{th} parking level was being cast with fresh concrete.

The Regional Administrator, Region IV, requested the Directorate of Construction (DOC), OSHA National Office, Washington, DC to provide engineering assistance to the Jacksonville Area Office. A structural engineer visited the incident site and examined the failed structure on December 11, 2007. The same structural engineer made a subsequent visit to the site on January 8, 2008.

Subsequently, DOC investigated the incident, analyzed the structure for the design loads and for the construction loads placed at the time that the 6^{th} level was being cast. This document includes the report and the conclusions reached.

The garage is a poured-in-place concrete structure measuring approximately 116 ft. x 252 ft. There was no basement in the garage and it consisted of six levels including the ground level, called the 1^{st} level. The roof was the 6^{th} level and was designed for parking as well. The parking garage was a part of a bigger project, a 23-story condominium tower, called Berkman Plaza II. Structurally, the parking garage is a separate structure from the tower. The structural design consisted of cast-in-place one way continuous post-tensioned slabs in the north-south direction and post-tensioned beams in the east-west direction. The columns were also cast in place. There were seven bays in the north-south direction and two in the east-west direction. The bays were unequal and, therefore, the thickness of the slabs varied. The lateral load-resisting system consisted of cast-in-place shear walls enclosing the stairwells and some additional shear walls on the north side. See Fig. 1 for a typical plan of the garage. For the purpose of this report, the prefix “G”, meaning garage, has been omitted for identifying column grid lines.

The following were the key participants in the project:

1. Architect: Pucciano & English of Atlanta, GA.
2. Structural Engineer of Record (SER): Structural Consulting Group, LLC also of Atlanta, GA.
3. General Contractor/Construction Manager: Choate Construction Company of Pooler, GA.
4. Formwork, Shoring Contractor: Southern Pan Services Company of Lithonia, GA.
5. Formwork Designer: Patent Construction Systems (Patent) of Tampa, FL and Universal Engineering Sciences (Universal).
6. Concrete sub contractor: A. A. Pittman & Sons Concrete Co., Inc. of Jacksonville, FL. This sub contractor was responsible for placing and finishing concrete for the slabs and beams but not the columns.
7. Concrete sub contractor: Southern Pan Services Company (Southern) of Lithonia, GA. was responsible for all vertical concrete, e.g., columns, shear walls.
8. Concrete provider: Florida Rock.
9. Reinforcing steel provider: Gerdau-Ameristeel of Jacksonville, FL.
10. Post-tensioning sub-contractor: PTE Strand Co., Inc.of Hialeah, FL.
11. Reinforcement placement sub contractor: Infinity Reinforcing of Palm Coast, FL.

The garage structure was placed under the threshold category by the Florida Building Code (FBC). Synergy Engineering (Synergy) was retained as the threshold inspector. Synergy had a contract for the condominium tower as well as for the garage. Among its responsibilities were to inspect the reinforcing steel, post-tensioning steel conforming to the contract drawings and approved shop drawings. Synergy also participated in the progress meetings held regularly at the site. It also had the responsibility for inspecting shores, reshores, and other formwork components. The site representative of Synergy was a registered professional engineer.

In addition to Synergy, Universal was another inspector at the site. Universal was retained by Southern to inspect the formwork, shoring and reshoring and advise them on such matters. Both Synergy and Universal prepared inspection reports.

The construction began in the early part of 2007 with pile foundations for the garage. The 1st level was a slab on grade. Casting of the elevated slabs began in June of 2007. Each level was divided in two parts called A and B for casting identification. Up to the time of the incident, five levels were already poured and the casting of the sixth level, part A, was in progress at the time of the incident. On December 6, 2007, concrete casting began in earnest in the early hours, e.g., 12:30 a.m., from the west side near column line E between column grid lines 2 and 3 proceeding north. First the crew poured concrete in the beam formwork up to the underside of the slab and then placed concrete for the slab. Concrete for the slab was successively placed without any reported problems. After having cast concrete in the bay bounded by column grid lines A & C and 2 & 3, the crew turned east and began placing concrete between column grid lines 3 and 4, then proceeding south towards column grid line E. They had completed casting concrete up to approximately 10-15 ft. south of column grid line C when the incident occurred.

The collapse was massive as it encompassed all the elevated slabs from columns grid lines A to G and column grid lines 2 to 4. The slabs fell generally on the top of each other with the columns crushed in between. The shores and re-shores were also crushed between the collapsing slabs and beams. See Fig.s 5 thru 18 for the extent of the collapse. Two bays on the south side, however, remained standing with slabs north of column grid line G hanging towards the north, still connected by rebars and post-tensioning cables, see Fig. 17. The failure included the shear wall on column grid line A and the shear walls enclosing stair G1 near column grid line 2.

Shores for the 6th level began to be erected on or about November 14, 2007. At the time of the collapse, the 6th level was shored down to the 5th level. Reshores were provided between the 5th & the 4th level, and between the 4th and the 3rd level. There were no reshores under the 3rd level as they had been removed earlier on or about November 19, 2007. Therefore, on the day of the incident, the loads of the wet concrete and other construction loads from the 6th level were supported on the 5th, 4th and the 3rd levels of the garage. This was the first time that concrete was being cast on elevated slabs without reshores extending down to the 1st level.

Southern retained Patent to design the formwork and to prepare formwork layout drawings including shoring and reshoring. Patent prepared the drawings showing the layout of the formwork, shores and reshores. The first three drawings bore a signature dated May 4, 2007 and the last five drawings had the same signature dated June 12, 2007. On drawings No. 7 & 8, number 8607K038, re-shores were indicated extending down to the 1st level. It required that at the time the 6th level was cast, all levels below the 6th level must be shored/reshored. See, Fig. 3.

During the interview with OSHA, Patent stated that it was their standard policy to ask the contractors to extend the reshores down to the ground level, regardless of the height of the structure and the number of floors. Patent, however, stated that if the contractor did not wish to place re-shores down to the ground level, the contractor had the option to retain an engineer to advise him whether fewer levels of reshores could be used.

There are conflicting reports about why Southern removed the reshores under the 3rd level despite the fact that the Patent drawing showed the reshores extending down to the 1st level. When OSHA asked Synergy why, as a threshold inspector, it would permit placement of concrete on the 6th level without the reshores under the 3rd level, it responded that the SER, in response to its e-mail seeking clarification of where re-shores were required, advised that reshores were only required under a certain slab requiring repairs, and at no other place. Synergy, therefore, did not raise the issue with the contractor of the lack of reshores under the 3rd level. See, attachment D, showing copies of the e-mails. It was discovered earlier that 61 top #5 mild steel reinforcement bars, 46-feet long, were inadvertently not placed in the ramp, from 2nd to 3rd level slab bounded by column grid lines D & E, and 2 & 3. To correct the structural deficiency created by the lack of rebars, SER recommended certain repairs to the slab and asked that the slab in question continue to be reshored until repairs were completed. OSHA asked SER about the e-mail. SER stated that his response was not meant to address the necessity for or lack of reshores anywhere in the garage except in the areas needing repairs. SER further explained that methods and means of construction are solely the responsibility of the contractor, and the contractor should determine whether shoring and reshoring are required.

The construction of the parking garage included many minor and major issues. It was reported to OSHA by a number of sources that the difficulties were compounded by the fact that the SER was not forthcoming in resolving the questions, and had a nonchalant and dispassionate attitude towards the structure he designed. SER denied this during an interview with OSHA. The majority of the issues arose at the beam-column joints from the congestion created by a large number of post-tensioning cables, top and bottom mild reinforcements of the beam, and longitudinal reinforcements and dowels in the column. See Fig. 4 for the number of reinforcements at the dead end of a post-tensioned beam. Honeycombing and voids were reported at the beam-column joints. For example, the 5th level beam on column line G between grid lines 1 & 3, the 2nd level beam on column line G between grid line 3 & 4, and the 5th level beam on grid line E between grid lined 2 & 3 could not be post-tensioned due to honeycombing at their ends.

Another set of issues arose from the cracks observed at the interior and exterior beam-column joints and in the slabs, see attachment E. For example, it was reported that cracks developed at multiple levels at the columns C-2, C-3, D-3, D-4, E-4, F-4, G-4 and H-4. There were also cracks at the slab framing into the shear walls enclosing the stair. For example slabs had cracks near the stair G1, G2 and G3 at the 3rd, 4th and 5th levels. There were also reported to be cracks in the 3rd level slab. An eyewitness reported during an OSHA informal interview that a crack extended diagonally across the post-tensioning cables through the entire depth of 20” thick slab on the 3rd level. Others reported cracks of a lesser severity and not through the entire depth of the slab. The cracks were brought to the SER’s attention. He responded that the cracks at the beam column joints and at the slab wall junctions were occurring due to the restraints against movement. He suggested that certain areas of slab be reshored and that the cracks should be kept under observation. When asked by OSHA about the cracks in the 20” thick slab away from the shear walls, SER expressed a lack of knowledge of these cracks. The cracks in the 20” thick slab were never fully resolved.

STRUCTURAL ANALYSES and DISCUSSION

The purpose of the structural analyses was to:

1. Determine whether the garage structure was properly designed in accordance with the industry standards.

2. Determine whether the third level could have supported the loads imposed upon it at the time of the incident without any reshores under the third level, and if the contractor had assumed, as is customary, that the structural design was sound and reliable.

3. Determine the cause of the collapse.

The following drawings were reviewed.

1. Structural drawings SG 0.1, SG 1.1, SG 1.2, SG 2.1 thru 2.5, SG 3.1 thru 3.5. SG 0.1 was signed on December 16, 2006. The rest were signed on September 5, 2006.
2. Architectural drawings , G-1 thru G-11 with various dates.
3. Formwork and shoring/re-shoring drawings 8607K038 (eight drawings)
4. Southern Pan Services Company drawings SG 3.1 thru 3.5, G 6 thru 8.
5. PTE Strand Co., strand lay-out drawings PT-01, PTP021, PTP020, PTP 030, PTP 040 and PTP 060
6. Gerdau Ameristeel re-bar detail drawings: R-05, RC-02 thru RC-10, RSG-1 thru RSG-12.

The structural analyses were generally limited to the area of the collapse. The following information provided in the general notes of the structural drawings was pertinent to this investigation:

1. Florida Building Code (FBC) was used to design the structure.
2. Design of the garage was based upon a live load of 50 psf, as indicated by the SER. (There is no mention of any live load reduction in the documents. It was, therefore, assumed that the FBC-permitted reduction was used, see attachment A).
3. 5,000 psi was indicated to be the concrete strength at 28 days for slabs, beams and columns. However, for our evaluation, a 6,000 psi concrete strength was assumed for the beams and slabs, based upon the testing laboratory documents, and 5,000 psi for the columns.

FBC and all other industry codes provide a “margin of safety” in the design of all structures by increasing the actual loads by factors called “Load factors” and by reducing the capacities of materials by “Phi (ϕ) Factors”. A combination of the two factors provides a desired factor of safety and is well recognized and practiced in the industry, and has served well, see attachment B. For the purpose of this report, evaluations were done with and without these factors to arrive at the code- prescribed design strength, and at the “failure” loads with no margin of safety.

The load factors considered in the evaluation of the design were (1.4 x DL) or (1.2 x DL + 1.6 x LL), which ever provided a higher value. For the strength design, the ϕ factor for flexure and shear was used as per ACI 318-02 code. For the evaluation of the structure, a live load of 40 psf was used as permitted by the FBC, instead of 50 psf as indicated by the SER in his general notes. However, if the contractor was to have determined whether the third level could support the loads imposed upon it at the time of the incident, a live load capacity of 50 psf could have been used. The contractor could have safely assumed that the 3rd level had a live load capacity of 50 psf, as this information was readily available on the structural drawings.

Evaluation of Slab:

The design consisted of one-way continuous post-tensioned slab in the north-south direction supported by post-tensioned shallow and wide beams in the east-west direction. In addition to the post-tensioning cables, the slab was reinforced with mild steel for positive and negative flexural moments including temperature reinforcements. The slab design was generally typical for all levels. The thickness of the slabs varied with their span lengths, as shown below:

Column line from	Span length	Slab thickness
GA to GC	60’-6”	20”
GC to GD	38’-10”	16”
GD to GE	25’-4”	8”
GE to GF	26’-10”	8”
GF to GG	26’-10”	8”
GG to GH	26’-10”	12”
GH to GI	47’-4”	14”

The slab design was found to be adequate for the live load of 40 psf without any live load reduction. The amounts of post-tensioning cables and mild steel were generally proper. The thicknesses of the slab also met the general ACI guidelines and undue deflections could not have been expected. The slab was also deemed satisfactory for a live load of 50 psf.

Evaluation of beams:

Flexure

The schedule of beams taken from the structural drawings is shown in Figure 2. For location of beams, see figure 1. For our evaluation, 6,000 psi was considered to be the strength of the concrete although the contract documents specified 5,000 psi as the concrete strength. Testing laboratory documents indicated that 6,000 psi was the required strength for beams and slabs (see Table 4). There were five different beams provided, SB-1 thru SB-5. The most critical beam of significance to this investigation was SB-5 that was the most heavily loaded as it supported wider spans of the slabs.

For our analyses, considerable thought was given to determine whether the SB-5 beam should be treated as a simple beam or with continuity with the column at the east end, and with the beam/column at the west end. It was quickly realized that fixity at either of the ends of the beam would be problematic due to a number of reasons. At the east end, there was a slender 14" x 28" column oriented about the minor axis with the beam. Further, the 60" beam was much wider than the column, thus only a few top reinforcements could develop their full strength in the column. The drawings called for 6 # 8 continuous top and bottom bars, of which only three could fall within the confines of the column. With a 90 degree hook, a minimum development length of 15 ½" was required for a concrete strength of 6,000 psi. The column was only 14" wide, and with the minimum amount of outside cover, it would not have been possible to develop full strength of the bars. The post-tensioning cables were placed at the center of gravity (c.g.), of the T-beam and thus could not be expected to provide continuity of the beam with the column.

On the west side, it was similarly problematic to consider the beam to be continuous. First, the post-tensioning cables were dead-ended on column grid line 3 at the c.g. of the beam. The top mild reinforcements did not continue to the adjoining span. The column was, however, 28" wide instead of 14". At least 3 #8 bars could be developed in the column with proper development lengths. It was calculated that 333 ft-kips of partial fixity could be obtained which is only 4.66% of the total simple positive moment and therefore, could be ignored.

The beam was evaluated for four load cases with 6,000 psi concrete:

1. Load case 1: Unfactored dead load of the beam and the slab.
2. Load case 2: Unfactored service loads consisting of dead load of the slab/beam and the reduced live load, as permitted by FBC, based upon a basic live load of 40 psf.
3. Load case 3: Factored dead load and factored reduced live load, as permitted by FBC.
4. Load case 4: Unfactored dead load and other unfactored loads of the wet concrete and construction loads coming from the higher levels at the time that the 6th level was being cast at the time of the incident.

Loads imposed upon the beams were derived based upon the tributary area. The beam had 78 strands in addition to 6 #8 rebars top and bottom. It was determined that, based upon concrete strength of 6,000 psi, the beam had a positive flexural strength of approximately 5,370 and 5,967 ft-kips with and without ϕ factor, respectively. Under the load case No.1, the actual demand to support the unfactored dead loads of the slab and the beam was 5,013 ft-kips. Under load case No. 2, the actual demand was 5,496 ft-kips below the design strength without the ϕ factor. However, under load case No. 3, the actual demand was 7,018 ft-kips, 31% higher than the design strength, indicating deficient design by the SER. Under load case No. 4 that represents the loads at the time of the incident, the actual demand was 7,150 ft-kips, higher than the design strength of 5,967 ft-kips, even when load factors and ϕ factors are not considered.

From the flexural aspect, the beam design was deficient under code prescribed load and ϕ factors. The beam was, however, able to support its own dead load with little factor of safety when the shores were removed. At the completion of the project, it is believed that the beam would have been able to support the load without the load and the ϕ factors.

At the time of the incident, case No.4, the actual demand was 7,150 ft. kips which could be reduced to 6,820 ft. kips, considering a fixity of 333 ft. kips at each ends of the beam. Even with consideration of the partial fixity, the actual demand was 14% higher than the design strength without load and ϕ factors. However, the actual demand could even be lower because the beam SB6 located between column grid lines 4A and 4C supported a part of the 3rd level loads coming from the 6th level, as this beam remained shored during the casting of the 6th level. This reduction in demand was not accounted for in the computation.

A failure due to flexure generally does not take place in a catastrophic manner as it provides visible deformation and noticeable sag before leading to the ultimate collapse. No such observations were reported by employees but future observations of the failed elements, after the current recovery is completed, could lead to re-evaluation.

Shear

Under load case Nos. 1, 2 and 4, our analysis indicated that the designed shear stirrups at a spacing of 12" o.c. were marginal, see Table 1. As mentioned earlier, the evaluation of these cases was done without considering load and ϕ factors. When load and ϕ factors were considered, the spacing of shear stirrups in all load cases were found to be deficient. In load case No. 3, the required spacing was 8" o.c., as per applicable codes instead of 12" as shown on the contract drawings.

The shear stirrups were significantly under-designed for the factored dead and live loads and did not meet the code requirements. At the completion of the project, it is believed that failure would not occur due to deficient shear design based upon unfactored dead and live loads but then, the margin of safety would be minimal. It is further believed that the deficient shear design did not contribute to the collapse as shown in Table 1, load case No.4.

Evaluation of Columns:

21 columns were evaluated for different load cases. A load combination of (1.4 x DL) or (1.2 x DL + 1.4 x LL) was used to arrive at the governing load. The following load cases were considered:

1. Load Case No.1: Unfactored dead load of slab, beam and columns.
2. Load Case No. 2: Unfactored dead and unfactored reduced live loads.
3. Load Case No. 3: Factored dead and factored reduced live loads (Basic live load of 40 psf).
4. Load Case No. 4: Unfactored dead loads and unfactored construction loads from the 6th level at the time of the incident.

The required capacities were compared with available strengths with and without the ϕ factor, see Table 2.

With ϕ factor, Design strength $\boldsymbol{\phi P_{n,\,max} = 0.80\phi[0.85 f_c' (A_g - A_{st}) + f_y A_{st}]}$
Without ϕ factor, Design strength $\boldsymbol{P_{n,\,max} = 0.80[0.85 f_c' (A_g - A_{st}) + f_y A_{st}]}$

Of the 21 columns, eight columns C2, C3, C4, D3, E3, F3, G3 and H4 were considered critical for the above four load cases, see Table 2.

Of the eight columns, all except H4 were determined to be deficient as per the prescribed codes, based upon the 5,000 psi concrete, the strength specified by the SER. However, if 6,000 psi concrete was considered, only four columns, C2, C3, C4 and D3 would be deemed to be deficient. Available records, see Table 3, indicated that the required strength was only 5,000 psi for all columns. Further, if the ϕ factor is not considered in the evaluation of the column design strength, all columns had the capacity to support the load even at 5,000 psi concrete strength, with the exception of C4.

The column C4 was considered the most critical. The size of the column C4 was increased below the third level due to architectural reasons. Therefore, loads from the third level and above were only considered for the C4 column. For load case No.1, C4 was barely able to support the dead loads even when the ϕ factor was not considered. When the ϕ factor is considered, the design strength was 971 kips compared with the demand of 1,545 kips. This is the most serious design flaw in the structure. For load case No. 2, the column could not support the loads with or without the ϕ factor at 5,000 psi concrete strength. Only at 6,000 psi concrete without the ϕ factor, the column could barely support the loads. For load case No. 3, the column was determined to be grossly under-designed. For the load case No. 4, it was computed that approximately 1,641 kips were placed on the column at the time of the incident between the third and the second floor. The column could not support this load even when the ϕ factor is omitted at 5,000 psi concrete strength. Only when the concrete strength is considered to be 6,000 psi, and when the ϕ factor is ignored, then the column is able to support the load.

Actual concrete strengths of the columns have been tabulated in Table No. 3. With the exception of two columns, most of the concrete breaking strengths at 28 days were noted to be 6,000 psi or higher. In two cases, however, the concrete strengths were approximately 5,700 psi. For the C4 column, there were three laboratory breaking strength reports available: 302 sampled on July 23, 2007; 435A sampled on October 12, 2007; and 436A sampled on October 15, 2007. Report 302 indicates a strength of 7,230 pounds at 7 days. Reports 435A and 436A indicate strengths of 5,770 and 6,480 pounds respectively, at 28 days. The sampling of the concrete for Report No. 302 was taken when concrete was placed between the 2nd and the 3rd levels. Therefore, if indeed the actual concrete strength was above 6,000 psi, and the margin of safety was disregarded, it is considered unlikely that the failure could have occurred at the loads placed on the C4 column at the time of the incident.

Discussion:

We will now consider whether it would have been appropriate for the contractor to have assumed that the 3rd level slab and beam would be able to support the loads during casting of the 6th level, assuming that the structural design was correct and reliable. Contractor had a right to assume that the structural design is sound and meets the applicable codes. It is concluded that it would be erroneous for the contractor to load the 3rd level during casting of the 6th level without performing an evaluation of the capacity of the slab, beam and column with due regard to the design parameters and applicable building codes. Only a person knowledgeable in structural design could perform such an evaluation. Regrettably, no such evaluation was performed.

We then considered that had the contractor performed such a proper evaluation, what conclusion would he have reached. Our analysis indicated that the contractor could have reached the conclusion that reshores might not be required under the 3rd level, if the design of the structure was properly performed. This conclusion would lead to little margin of safety, and failure could occur with any incidental increase of construction load.

SER had indicated that the design of the garage was based upon a live load of 50 psf. It was also mentioned on the structural drawings that the design was performed in accordance with FBC that would have permitted a reduced live load of 30 psf (60% of 50 psf) for the beam. The 20" slab would therefore be designed for its dead load of 250 psf and a live load of 30 psf. Similarly, the 16" slab would be designed for its dead load of 200 psf and a live load of 30 psf. The ultimate load capacity of the 20" slab would therefore be 1.4 x 250 = 350 psf, and that of 16" slab would be 1.2 x 200 + 1.6 x 30 = 288 psf. Applying the phi factor of 0.9, the ultimate strength capacities at the time of failure would be increased to 389 psf and 320 psf for 20" and 16" slabs respectively. Therefore, the 20" slab had a "reserve" capacity of 139 psf (389-250=139) and the 16" slab had a "reserve capacity of 120 psf (320-200=120).

The superimposed loads from the 6th level during its casting would be:
Dead load of concrete = 250 psf for 20" slab; 200 psf for 16" slab
Construction load = 50 psf, see Attachment C
Forms and shores = 6.5 psf, see Attachment C

All loads except the forms and shores will be shared equally by the 5th, 4th and 3rd levels. The 20" and 16" slabs at the 3rd level would therefore be subjected to load of 106.5 psf and 90 psf respectively below their "failure loads" at the time of casting of the 6th level.

It must be mentioned here that at the completion of the structure, the 3rd level would never experience a load as large as it was subjected to during the 6th level casting because the garage floors are designed for a light live load of 30 psf as per FBC. When the 6th level was being cast, the 3rd level was subjected to a load 300% greater than the live load.

Conclusions:

1. At the time of the incident when the concrete on the 6^{th} level was being poured, there were no re-shores below the 3^{rd} level except a few under the perimeter beams. The re-shores under the 3^{rd} level in the collapsed area were removed by the shoring subcontractor without a determination made by a person knowledgeable in structural design that the 3^{rd} level would be able to support the loads of the wet concrete and construction loads from the 6^{th} level, and form and shore loads. The contractor violated OSHA's 1926.701(a) standard. If the contractor had not removed the reshores, the incident would not have occurred despite the flawed structural design.

2. If a proper structural determination was done on the premise that the design by the structural engineer of record was sound and reliable, the contractor could have concluded that the re-shores under the 3^{rd} level might not be required during casting of the 6^{th} level. However, there would be no margin of safety.

3. The structural engineer of record (SER) designed the structure with critical flaws. His design did not meet the industry standards, and was extensively deficient.

4. The structural engineer of record designed the columns and beams that could not support the code-prescribed loads. The SER could not provide any computations justifying his design, even when multiple opportunities to do so were afforded him by OSHA. The design of the one-way post-tensioned slabs was found to be adequate.

5. The garage structure at its final completion would not have collapsed under the code prescribed live load, even with flawed design but would have had little margin of safety, a gross violation of the industry practice.

6. The shoring subcontractor disregarded the shoring plans prepared by its subcontractor which indicated that the 3^{rd} level should be reshored down to the 1^{st} level. There were no

other shoring plans available at the site for the employees to rely upon and refer to. Thus, OSHA standard 1926.703(a)(2) was violated.

7. In the areas of the parking garage that were still standing after the incident, several aluminum stringers were observed to have been placed in the flat position instead of the upright position. This compromised the load-carrying capacity of the beams. However, this did not contribute to the collapse.

8. The threshold inspector failed in his duty to report to the appropriate party the absence of reshores below the 3rd level at the time that the 6th level was being cast. If the threshold inspector had reported the absence of reshores, this incident would not have occurred.

9. The threshold inspector failed to notice that top continuous rebars were missing in the ramp slab from the second to the third level. The slab was poured without the top bars. Thus, the threshold inspector failed in his duties. However, this did not contribute to the collapse, as the slab was continued to be shored.

10. Given the presence of cracks in the 3rd level slab, the threshold inspector and the general contractor failed to resolve the potential adverse impact of the cracks on the load-carrying capacity of the slab. They thereby permitted the subcontractor to proceed with casting of the higher levels without resolving the issue with the engineer of record or with any other engineer with expertise in structural engineering

11. The absence of reshores was in plain sight of everyone involved. If the shoring plans that called for re-shores under the 3rd level were disregarded by the shoring subcontractor, the general contractor and the threshold inspector both failed to ask for new plans.

TABLE 1

SUMMARY OF DESIGN FORCES OF THIRD FLOOR BEAM SB-5 (f_c’ = 6,000 psi)

Loading stage	Loading during construction without load factor and ϕ factor ϕ = 1.0 for bending and 1.0 for shear									Loading for finished structure with load factor and ϕ factor ϕ = 0.9 for bending and 0.75 for shear		
Loading	Load case 1: Unfactored dead load (1.0 DL) of the third floor beam and slab			Load case 2: Service load Unfactored (DL + reduced LL)			Load case 4: Unfactored (DL + wet concrete and construction load from column line A to D at sixth floor) Load combination ={(1.0 DL) + 1/3{(wet concrete at 6^{th} floor + 50 psf)}			Load case 3: Factored {(DL + reduced LL)} or factored {(DL)} Load combination = {(1.2 DL) + (1.6 reduced LL)} or {(1.4 DL)} Loading (1.4 DL) Governs		
Magnitude	Actual demand	Design strength	Remarks	Actual demand	Design strength	Remarks	Actual demand	Design strength	Remarks	Actual demand	Design strength	Remarks
Flexural moment (Unit ft-kips)	5,013	5,967	Actual moment is less than its design strength. ∴ **O.K.**	5,496	5,967	Actual moment is less than its design strength. ∴ **O.K.**	7,150	5,967	Actual moment is 20 % beyond its design strength. ∴ **N.G.**	7,018	5,370	Actual moment is 31 % beyond design strength. ∴ **N.G.**
Design shear at h/2 From support face (Unit kips)	326	520	Actual shear is less than taken by concrete. ∴ **O.K**	357	520	Actual shear is less than taken by concrete. ∴ **O.K**	465	520	Actual shear is less than taken by concrete. ∴ **O.K**	456	390	Existing shear stirrups of # 4 at 12” (against required at 8”) on center is not enough to resist the shear. ∴ **N.G.**

TABLE 2

SUMMARY OF CRITICAL COLUMN DESIGN LOADS AND ITS DESIGN STRENGTH

(UNIT KIPS)

Loading	Garage column designation based on their grid line							
	C2[1]	**C3[1]**	**C4[2]**	**D3[2]**	**E3[1]**	**F3[1]**	**G3[1]**	**H4[2]**
Unfactored dead load of the slab, beam and columns	1,027	2,679	1,545	1,285	777	765	840	674
Service load (unfactored DL + unfactored reduced LL)	1,089	2,846	1,641	1,385	869	861	894	716
Ultimate load◙	1,438	3,751	2,163	1,799	1,088	1,071	1,176	944
Load during casting of sixth floor: unfactored (DL + sixth floor wet concrete load and construction load from A to D)	1,080■	2,841■	1,641■	1,283■	613	595	652	504
Design strength ϕP_n▲ (f_c’ = 5,000 psi)	1,049	2,196	971	1,049	1,049	1,049	1,049	971
(f_c’ = 6,000 psi)	1,220	2,536	1,142	1,220	1,220	1,220	1,220	1,142
Design strength ϕ =1.0 (f_c’ = 5,000 psi)	1,614	3,378	1,494	1,614	1,614	1,614	1,614	1,494
(f_c’ = 6,000 psi)	1,877	3,902	1,757	1,877	1,877	1,877	1,877	1,757
Ultimate load vs. Design strength ϕP_n▲ (f_c’ = 5,000 psi)	**N.G.**	**N.G.**	**N.G.**	**N.G.**	**N.G.**	**N.G.**	**N.G.**	**O.K.**
(f_c’ = 6,000 psi)	**N.G.**	**N.G.**	**N.G.**	**N.G.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**
Service load vs. Design strength (ϕ = 1.0) (f_c’ = 5,000 psi)	**O.K.**	**O.K.**	**N.G.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**
(f_c’ = 6,000 psi)	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**
Load at the time of incident vs. design strength (ϕ =1.0) (f_c’ = 5,000 psi)	**O.K.**	**O.K.**	**N.G.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**
(f_c’ = 6,000 psi)	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**	**O.K.**

Legend:

- **1** Load from sixth floor through second floor.
- **2** Load from sixth floor through third floor.
- ▲ Design strength $\boldsymbol{\phi P_{n,\,max} = 0.80\phi[0.85f_c\,'\,(A_g - A_{st}) + f\,y\,A_{st}]}$ (Ref. ACI 318-02, Eq. 10-2)
- ■ Load is based on weight of wet concrete load and construction load on sixth floor from column line A to column line D only.
- ◙ Ultimate load is based on 1.4 D.L. which is greater out of loading combination of (1.2 DL + !.6 L.L.) or (1.4 DL)

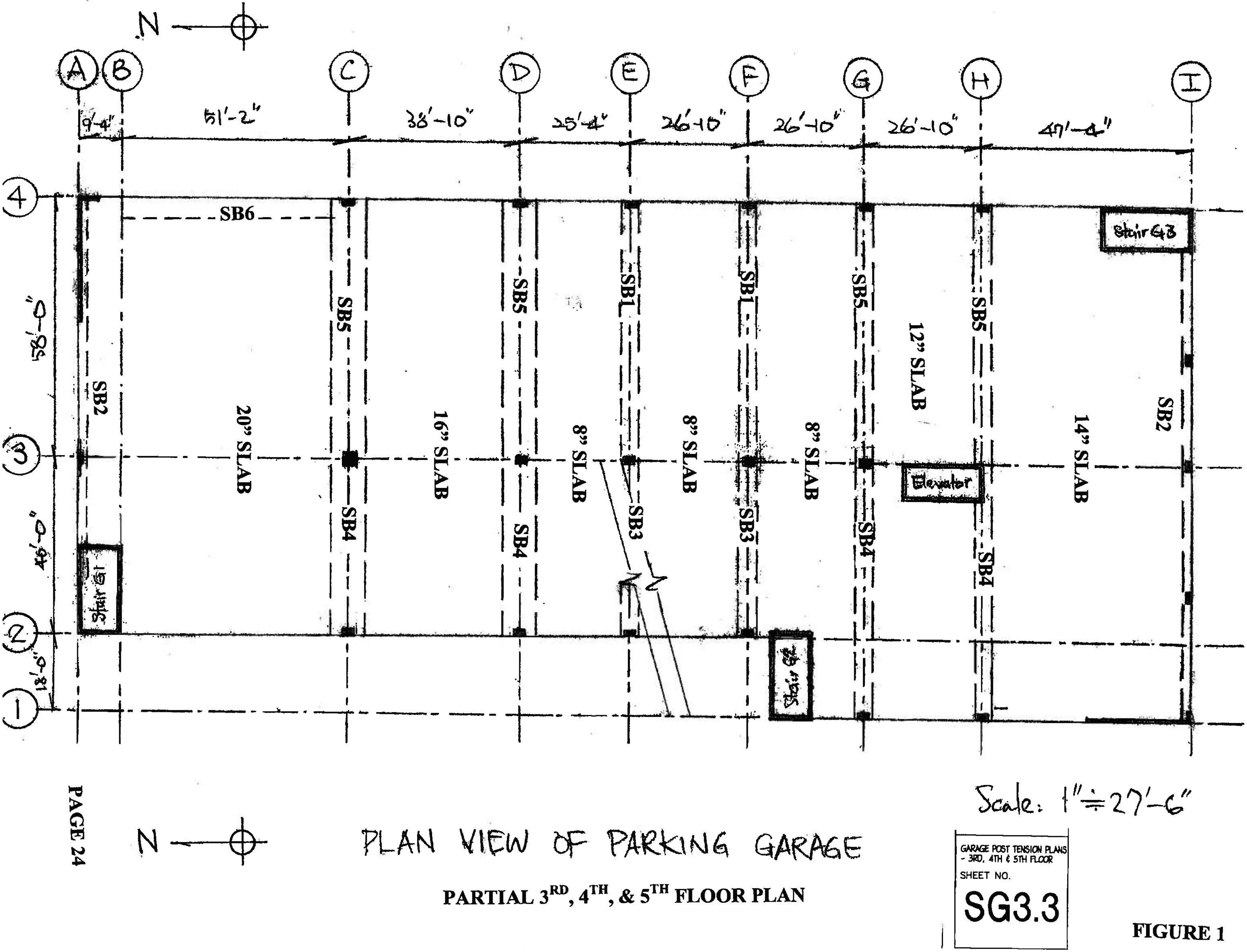
TABLE 3

SUMMARY OF CONCRETE CYLINDER TEST REPORT FOR COLUMNS

Report Number	Date Sampled	Pour Level	Floor To Floor Numbers	Column Numbers	Required 28-Day Strength (psi)	Test Results (psi)	
						7 Days	28 Days
105	3/5/07	G1	1st to 2nd	C3, I2.3	5,000	3,170	6,330
118	3/22/07	G1	1st to 2nd	I3, I3.5	5,000	4,590	7, 310
125	3/30/07	G1	1ST to 2nd	G4	5,000	4,620	7,320
137	4/11/07	G1	1ST to 2nd	H1	5,000	5,070	7,465
145	4/17/07	G1	1ST to 2nd	G1	5,000	4,810	7,355
216	5/30/07	G1	1ST to 2nd	E3, F3,G3	5,000	3,970	Not Identified in the report
164	4/26/07	G2	Foundation To 2nd floor	Gridline A	5,000	4,230	Not Identified in the report
302	7/23/07	G2	2nd to 3rd	C2, C3, C4	6,000	7,230	Not Identified in the report
319	8/1/07	G2	2nd to 3rd	I3.5, I3, I2.3,E3, G3	5,000	4,050	5,715
372	9/4/07	G3B	3rd to 4th	D4, E4, D3, D2, C2	5,000	4,610	6,880
435A	10/12/07	G4B	4th to 5th	C2, A3, C4, D4	5,000	3,720	5,770
436A	10/15/07	G4B	4th to 5th	B4, C4, E4, C2, D3, E3	5,000	4,760	6,480
476A	11/9/07	G5A	5th to 6th	G1, H1	5,000	5,080	6,655
504	11/29/07	G5A	5th to 6th	F3, G3, G4	5,000	3,090	Not Identified in the report

TABLE 4

SUMMARY OF CONCRETE CYLINDER TEST REPORT FOR SLAB AND BEAMS						
Report Number	Date Sampled	Pour Level	Pour Area	Required 28-Day Strength (psi)	Test Results (psi)	
					7 Days	28 Days
352	8/27/07	G3A	Beam SB-5 at Grid Line G0-G3	6,000	7,330	9,215
353	8/27/07	G3A	Ramp from 2nd to 3rd floor at Grid Line G0-G3	6,000	5,850	7,900
354	8/27/07	G3A	Ramp from 2nd to 3rd floor at GC-G3	6,000	7,320	8,460
355	8/27/07	G3A	At Grid Line C5-4	6,000	7,120	9,235
356	8/27/07	G3A	At Grid Line F-3	6,000	6,810	Not Identified in the report
357	8/27/07	G3A	At Grid Line B-4	6,000	6,870	7,940
358	8/27/07	G3A	At Grid Line E-3	6,000	7,000	9,445
463	11/7/07	G5A	Not Identified in the report	6,000	5,460	Not Identified in the report
464	11/7/07	G5A	Not Identified in the report	6,000	5,590	Not Identified in the report
465	11/7/07	G5A	Not Identified in the report	6,000	6,110	Not Identified in the report
466	11/7/07	G5A	Not Identified in the report	6,000	6,580	Not Identified in the report
467	11/7/07	G5A	Not Identified in the report	6,000	5,100	Not Identified in the report
468	11/7/07	G5A	Not Identified in the report	6,000	5,110	Not Identified in the report



PLAN VIEW OF PARKING GARAGE

PARTIAL 3RD, 4TH, & 5TH FLOOR PLAN

FIGURE 1

POST-TENSION REINFORCING SCHEDULE

BOTTOM BARS

MARK	BAR REQUIREMENTS
B1	#6 X 31' @ 14" O.C.
B2	#6 X 16' @ 24" O.C.
B3	#6 X 18' @ 12" O.C.
B4	#6 X 32' @ 10" O.C.

TOP BARS

MARK	BAR REQUIREMENTS
T1	#5 X 8' @ 12" O.C.
T2	#5 X 16' @ 18" O.C.
T3	#5 X 12' @ 10" O.C.
T4	#5 X 46' @ 10" O.C.
T5	#5 X 46' @ 8" O.C.
T6	#5 X 24' @ 7" O.C.
T7	#5 X 14' @ 7" O.C.

BEAM SCHEDULE

MARK	REINFORCING
SB-1	27"D x 38"W w/ 44 STRANDS & 6-#8 TOP @ BOTTOM FULL LENGTH WITH #4 TIES @12" O/C.
SB-2	27"D x 48"W w/ 39 STRANDS & 6-#8 TOP @ BOTTOM FULL LENGTH WITH #4 TIES @12"O/C.
SB-3	27"D x 18"W w/ 24 STRANDS & 6-#8 TOP @ BOTTOM FULL LENGTH WITH #4 TIES @12" O/C.
SB-4	33"D x 60"W w/ 62 STRANDS & 6-#8 TOP @ BOTTOM FULL LENGTH WITH #4 TIES @12" O/C.
SB-5	33"D x 60"W w/ 78 STRANDS & 6-#8 TOP @ BOTTOM FULL LENGTH WITH #4 TIES @12" O/C.
SB-6	72"D x 30" W WITH 36 STRANDS & 6-#8 TOP @ BOTTOM FULL LENGTH WITH #4 TIES @12" O/C.

GARAGE STRUCTURAL NOTES

SHEET NO.

SG0.1

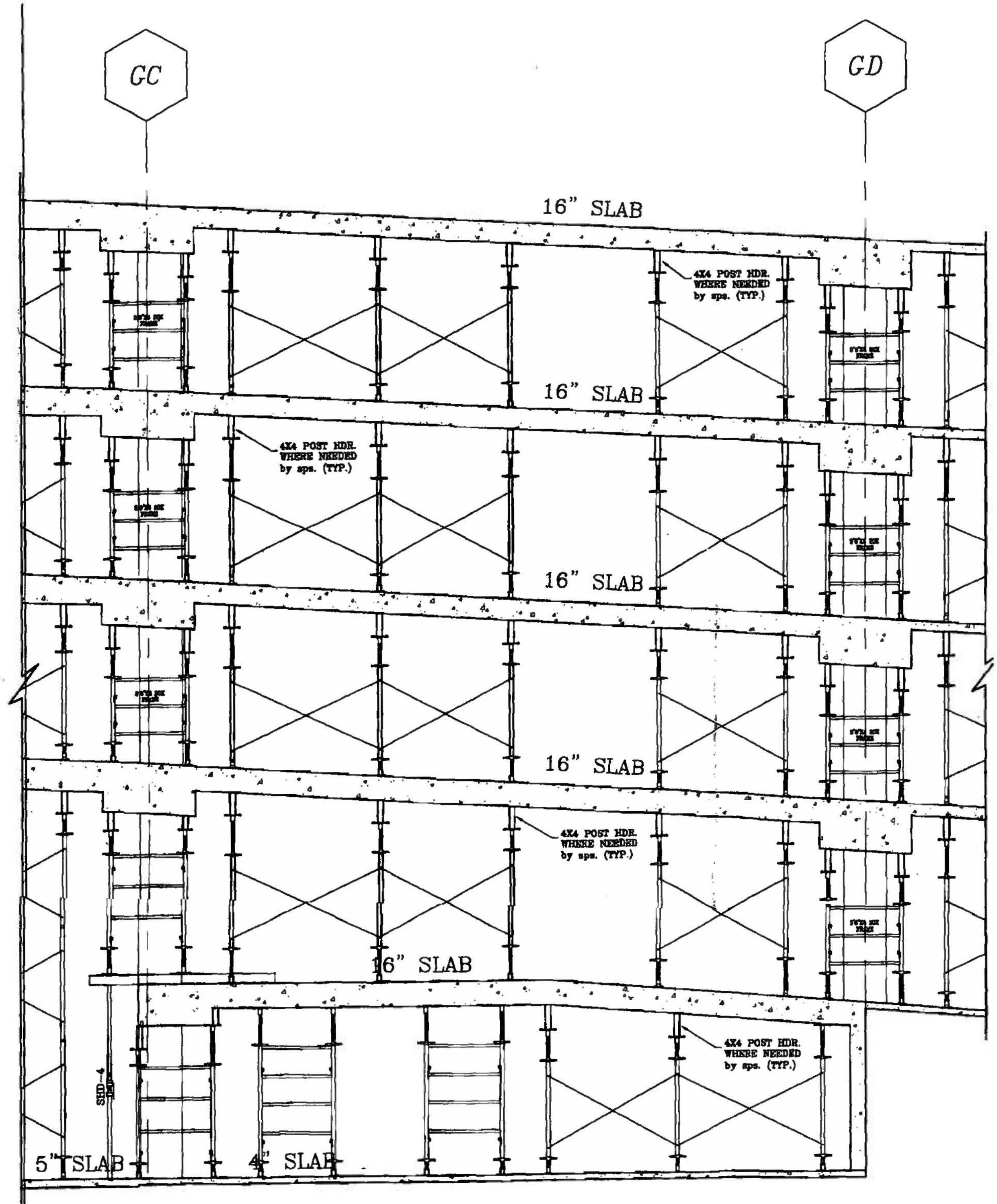
SHEAR WALL SCHEDULE

MARK	DIMENSIONS	REINFORCING
SW-1	8" THICK CONCRETE WALL FULL HEIGHT, SEE SHEET SG3.1	#6 @ 16" o.c. VERT. E.F.,#4@16"o.c. HORIZ E.F. & 4-#4 VERT. E.E.
SW-2	14" THICK CONCRETE WALL FULL HEIGHT, SEE SHEET SG3.1	#6 @ 16" o.c. VERT. E.F.,#4@16"o.c. HORIZ E.F. & 4-#4 VERT. E.E.

COLUMN SCHEDULE

MARK	f'c = 5000 psi	
	DIMENSIONS	REINFORCING **
C1	14" x 28"	6-#7 VERT. w/ #3 TIES @ 12" O.C.
C2	14" x 28"	8-#8 VERT. w/ #3 TIES @ 12" O.C.
C3	24" x 28"	WITH 10-#9 VERT. #3 TIES @ 12" O.C.
C4	25" x 48"	WITH 12 #9 VERT. #3 TIES @12" O/C.
C5	28" x 28"	WITH 16 #9 & #3 TIES @12" O/C.

FIGURE 2



PARTIAL RE-SHORING SECTION (TYP.)

Scale: 3/16"=1'-0"

FIGURE 3

DRAWN: CH	CHECKED:	APPROVED:	DATE: 5/20/07	SCALE: AS NOTED
TITLE: SECTION D/D & TYP. RESHORE				
PROJECT: BERKMAN PLAZA GARAGE				
LOCATION: JACKSONVILLE, FL				
CUSTOMER: SOUTHERN PAN SERVICES				
Patent Construction Systems Harsco Corporation			DRAWING NUMBER 8607K038	SHEET 7 OF

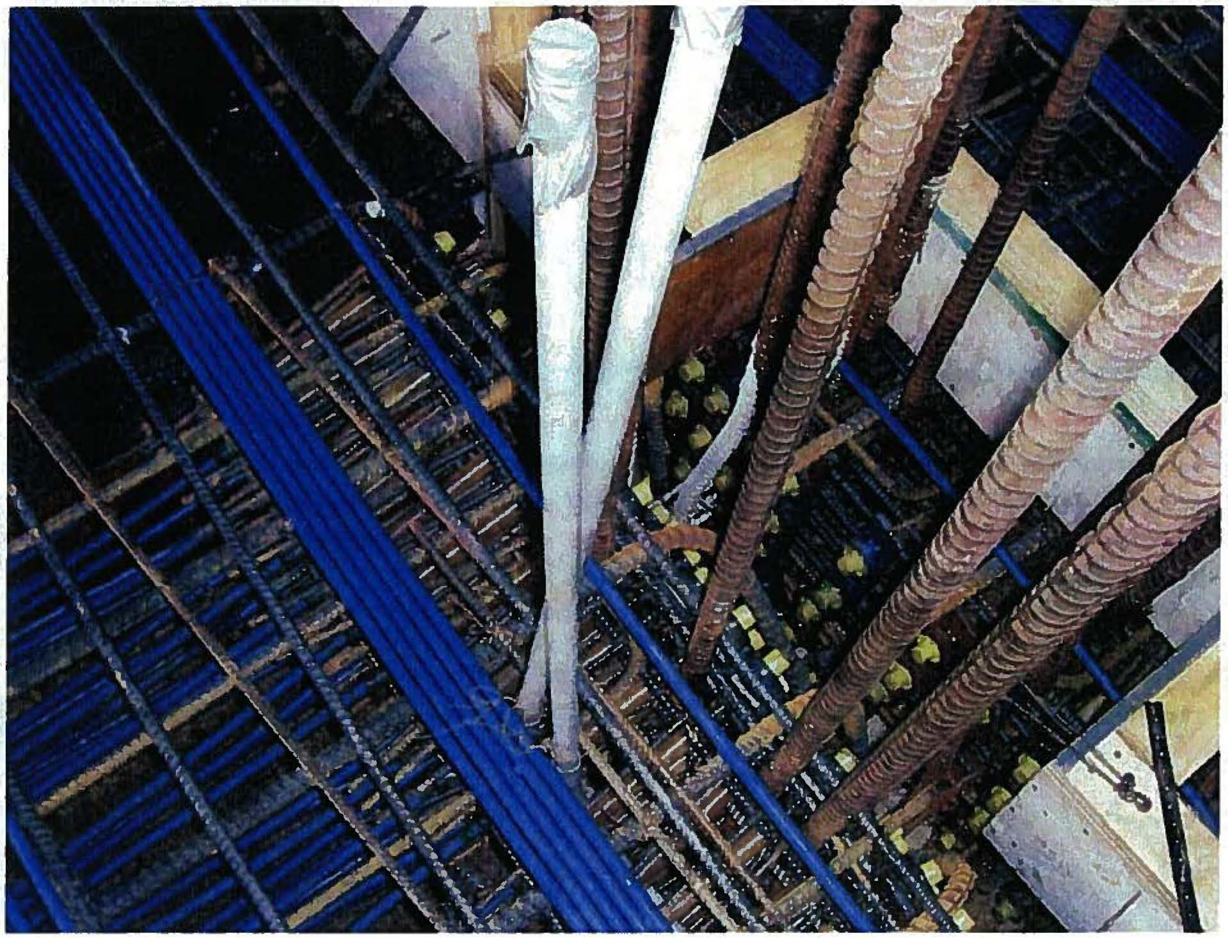


FIGURE 4

FIGURE 5

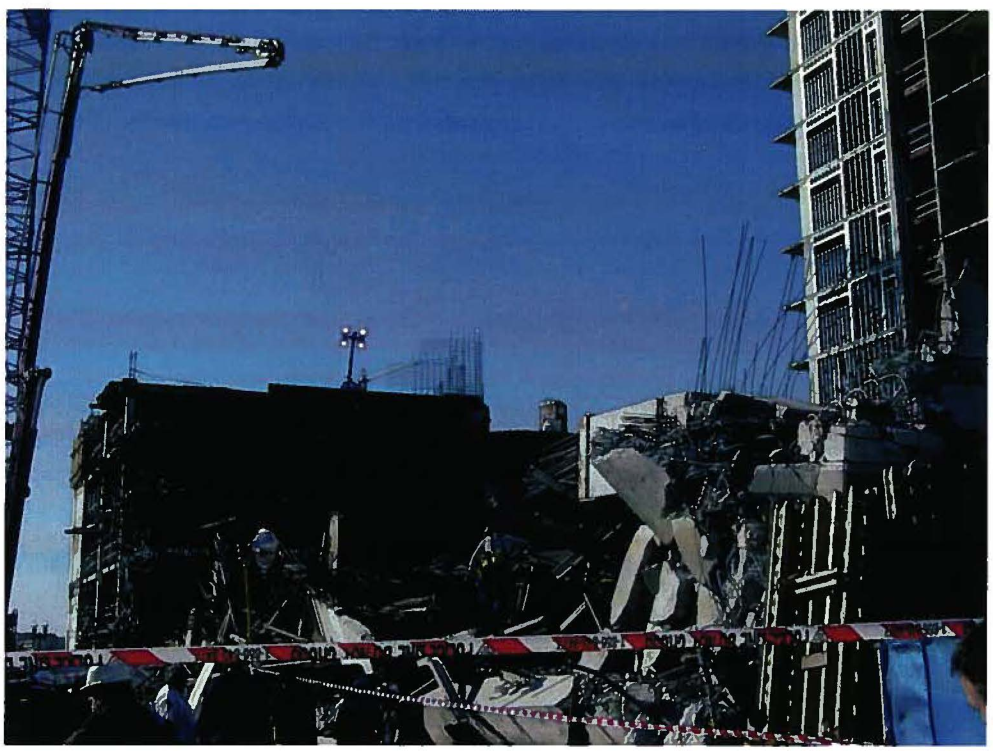


FIGURE 6

FIGURE 7

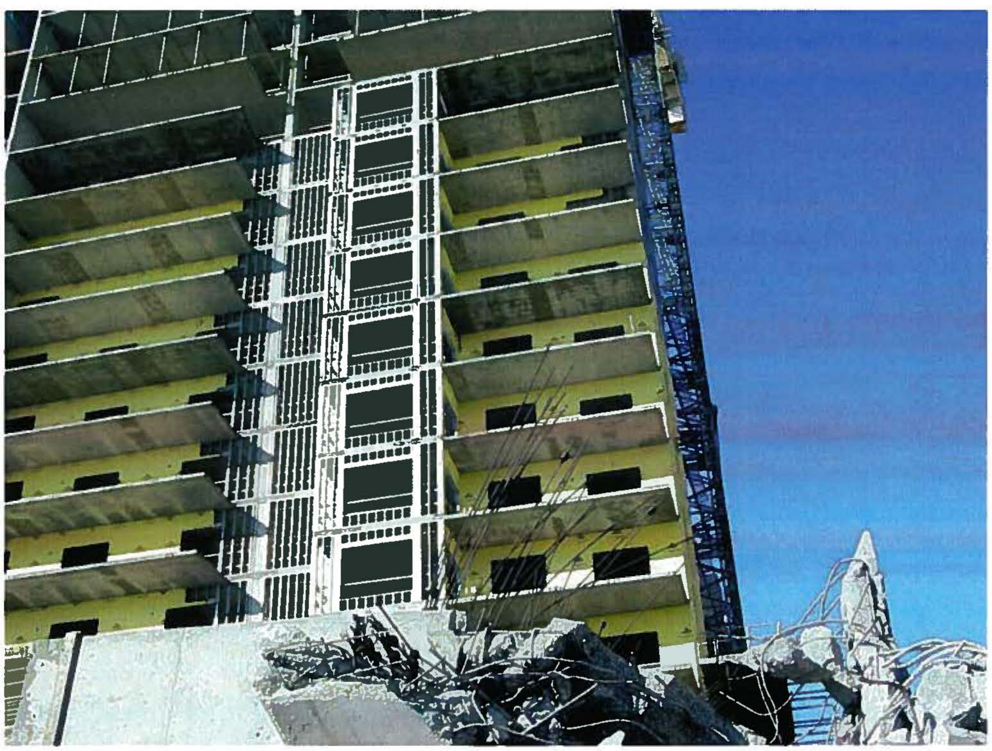


FIGURE 8

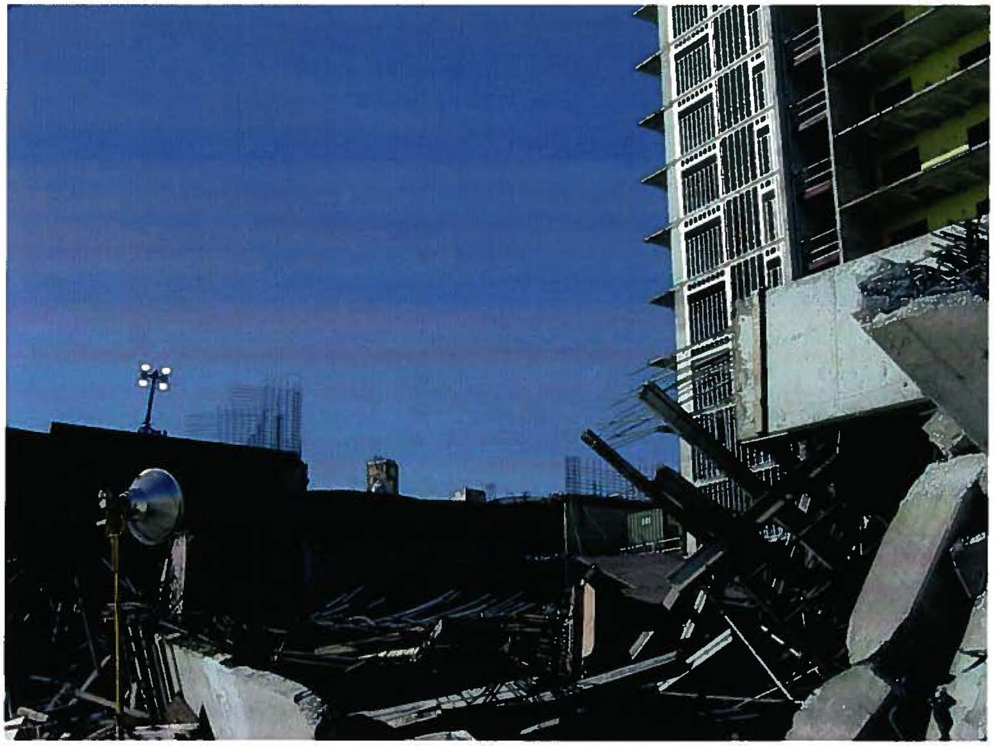


FIGURE 9

FIGURE 10

FIGURE 11

FIGURE 12

FIGURE 13

FIGURE 14

FIGURE 15

FIGURE 16

FIGURE 17

FIGURE 18