

# **INVESTIGATION OF THE OCTOBER 30, 2003, FATAL PARKING GARAGE COLLAPSE AT TROPICANA CASINO RESORT, ATLANTIC CITY, NJ**

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

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# **REPORT**

## **INVESTIGATION OF THE OCTOBER 30, 2003, FATAL PARKING GARAGE COLLAPSE AT TROPICANA CASINO RESORT, ATLANTIC CITY, NJ**

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## **Executive Summary:**

On October 30, 2003, at about 10:40 AM, a parking garage under construction in Atlantic City, NJ, collapsed, killing four construction workers and injuring 21 others. The parking garage was a part of the Tropicana Casino and Resort expansion project. At the time of the incident, concrete was being cast on the P8 parking level. The collapse resulted in the failure of five levels of an exterior bay, P7 thru P3. The collapsed levels completely separated from the columns and the shear wall at the west end, while they remained attached at the east end (See Figures 1 to 8). In addition to the collapsed slabs, one of the columns also fell to the ground. A shear wall and four columns were left standing freely, without any lateral support. The shear wall and the columns were later saw cut and demolished in sections.

Within hours of the incident, the Marlton Area Office Director and his staff reached the incident scene and assisted the local fire and rescue squad in developing a plan to prevent secondary collapses. Discussions were held with the construction manager about proceeding in a controlled manner, stabilizing the unstable structure, and demolishing the free standing shear wall and columns. The Occupational Safety and Health Administration's (OSHA) Region II Administrator, requested the Directorate of Construction (DOC), National OSHA Office, to provide engineering assistance for causal determination. Two structural engineers from DOC arrived at the construction site the day after the incident and provided engineering assistance.

OSHA personnel interviewed construction workers and other professionals at the construction site, took photographs and videos of the collapsed area, obtained construction documents, and examined the retrieved, failed structural elements. DOC conducted an independent structural evaluation of the failed parking structure. Based on the above, the Occupational Safety and Health Administration conclude that:

1. Fabi Construction and Mitchell Bar Placement, Inc. did not provide the required embedment length for the welded wire mesh at the intersection of the exterior columns and the slab/beam on grid line 1, levels P4 thru P7, per contract requirements and standard industry practice. Correct placement of rebars and mesh were crucial to the integrity of the structure.
2. Fabi Construction failed to detail, fabricate and place bottom reinforcing steel, identified as B49, on grid line 1, levels P4 thru P7, as required by the structural contract drawings. Omission of the reinforcing steel compromised the integrity of the structure.
3. The Fabi superintendent failed to seek the immediate attention of the general contractor/construction manager or the structural engineer of record when Fabi employees brought to his attention the cracks around the exterior columns on grid line 1 at levels P4 thru P7. The employees noticed consistent cracks at the interior long face of the columns extending at an angle of forty five degrees towards the edge of the slab/beam. These cracks should not have been dismissed

as shrinkage cracks because of their uniformity, depth and pattern on all levels. The collapse could have been averted if immediate attention was sought.

4. Fabi failed to re-shore an adequate number of floors, as required by the project specification, at the time concrete was being cast on level P8. Only one level was re-shored instead of the required three. Levels P6 and P7 supported the dead load of wet concrete instead of four levels (i.e., P7, P6, P5 and P4). Given the fact that cracks were earlier noticed at and around the exterior columns, the fewer levels of re-shores were highly detrimental to the integrity of the structure. This contributed to the collapse.
5. Fabi ordered premature removal of re-shores between levels P5 and P6. Given the presence of cracks at and around the columns, premature removal of shores created hazardous situations. This contributed to the collapse.
6. Shores were “cracked” without verifying that the concrete had reached “sufficient strength”, as required by the OSHA standard.
7. Fabi Construction did not prepare shoring drawings for the area of the collapse, levels P4 thru P7, in violation of the OSHA standard.
8. Site Blauvelt Engineers, contracted by the general contractor to perform an independent inspection, performed poorly in inspecting the placement of reinforcing steel. He failed to notice that the structural mesh did not have proper embedment at the exterior columns on grid line 1, levels P4 thru P7, to achieve full strength. It was expected of him to immediately notify the general contractor of the discrepancy before the concrete was cast over them. The inspector failed to check whether the reinforcing steel and mesh were placed in accordance with approved shop drawings.
9. The structural engineer did not exercise reasonable diligence in approving the shop drawings, which did not contain the bottom rebars, marked B49 on the contract drawings. He further failed to comment on the desired placement of the longitudinal rebars of the exterior slab/beam within the confines of the columns. The structural engineer was in a unique position to address the integrity of the slab/beam connection to the columns as he had access to all information including the intent of his design.
10. The structural design of the slab/beam-column joints on grid line 1 was flawed. The structural engineer of record improperly proportioned the slab/beam-column joints, in the area of the collapse, to support the code prescribed loads.
11. The structural engineer relied heavily on the filigree fabricator for the structural design of the exterior bay between grid line 1 and 4, levels P4 thru P8. The structural engineer did not conduct independent structural calculations to proportion the negative reinforcements, shear reinforcement, and potential

torsion on the exterior beam. The responsibility of the design rested solely with the structural engineer of record.

12. The structural drawings lacked clarity. On certain levels B49 was indicated, while on other levels it was not. The section thru the exterior slab/beam-column was not shown on the contract structural drawings to clarify the position of reinforcing rebars in each direction, column reinforcements, or beam shear stirrups. Lack of clarity resulted in only one, and in some cases, no rebar in the confines of the column.
13. Concrete strength is not a suspect in the collapse.
14. Wind was not a causal factor.
15. The activities of the structural engineer of record in connection with this project were not covered by the construction standards, and therefore were considered to be outside OSHA's jurisdiction.

## **Incident:**

The incident occurred on October 30, 2003, at about 10:40 AM in Atlantic City, NJ, at the construction site of a ten-story parking garage. Four workers were killed and twenty-one injured. The 3,000 car parking garage was a part of an ongoing \$250 million project that included a 500-room hotel, and entertainment and dining facilities for the Tropicana Casino and Resort in Atlantic City, NJ. At the time of the incident, concrete was being placed on level P8 in an exterior bay. After the placement of concrete continued for nearly three hours, level P8 and four lower levels, P7 thru P4, suddenly collapsed. The concrete on level P8 was still in the wet stage and slid down over the collapsing slabs. The shores between levels P7 and P8 fell and were scattered among the debris. The re-shores between levels P6 and P7 were trapped between the fallen P6 and P7 slab. The slabs on level P7 thru P4 completely separated from the columns, and the shear wall on the exterior column grid line 1, but remained connected to the beams on grid line 4 (see figures 1 to 8).

Fire and rescue teams rushed to the scene to recover bodies and transport injured workers to medical facilities. OSHA's Marlton Area Office immediately responded to the incident site and worked in close cooperation with local enforcement officials to provide necessary assistance in the post-collapse site management. To prevent further collapse, the parking garage structure was shored, guyed and braced. As a result of the collapse, the free-standing concrete shear walls and four free-standing concrete columns were later demolished in a controlled manner, without any incident.

The Directorate of Construction, National OSHA Office was contacted by the Regional OSHA Administrator to provide engineering assistance and causal determination. Two structural engineers from the Office of Engineering, Directorate of Construction, arrived at the incident scene to provide assistance.

## **Description of the Project:**

The ten-story parking garage was designed as a reinforced concrete structure with shear walls and intermediate moment resisting frames to resist lateral loads. The framing generally consisted of a one way slab spanning in the east-west direction, supported by continuous cast in place beams in the north-south direction (see figures 9 to 16). The beams were supported by cast in place columns. The bays in the east-west direction generally were 36-feet wide, except for the exterior bay that was 48-feet wide. The incident occurred in the exterior bay. The one way slab was a composite construction consisting of a bottom piece of 2 ¼-inch pre-cast, pre-stressed concrete thin slabs known as Filigree slab (see figures 17 to 23), and 7 ¾-inch cast in place concrete. The bottom piece also acted as a form work for the cast in place concrete. The bottom piece contained mild reinforcing steel and high strength tendons to provide the positive flexural capacity for the composite slab. A bond between the pre-cast slab and the cast in place concrete was achieved by steel stirrups/trusses protruding from the Filigree slab into the

cast in place concrete. Similar to the one way slab, the interior and exterior beams also consisted of composite construction. The interior beams consisted of a U-shaped pre-cast, pre-stressed concrete element known as Filigree tubs. Concrete was then placed in the tub to form a composite beam.

For the exterior beams, L-shaped pre-cast, pre-stressed concrete element known as "half tub" was used. The structural engineer originally designed the exterior beam as a cast in place upturn beam on grid line 1, with the upturn beam acting as a crash wall between the columns. As a result of value engineering, however, the upturn beam was changed to the half tub Filigree beam, with a pre-cast crash wall. Fabi Construction Company Inc. and Midstate Filigree Systems, Inc. proposed to use the half tub beam on grid line 1 instead of the upturn beam, with the approval and concurrence of the structural engineer of record. The full and half tubs were reinforced with mild steel and pre-stressing tendons to provide positive flexural capacity. In addition, shear stirrups were also cast in the full and half tub. Concrete was then placed in the tubs to form continuous beams.

### **Design and Construction Teams:**

Among others, the following were members of the design team responsible for preparing contract documents:

1. WAT&G, Inc. Architects, NJ, retained by Tropicana Casino & Resort to provide architectural and engineering services for the expansion project.
2. SOSH Architects, Atlantic City, NJ, retained by WAT&G as an associate local architect.
3. DeSimone Consulting Engineers, New York, NY, structural engineer of record (SER), retained by WAT&G to design and prepare structural drawings and specifications.
4. Midstate Filigree System, Inc. (MSF), Cranbury, NJ, the designer and fabricator of Filigree panels and tubs.

The construction team consisted of, but not limited to, the following:

1. Keating Building Corporation (KBC), general contractor and construction manager;
2. Fabi Construction Inc. (Fabi), Egg Harbor Twp, NJ, concrete sub-contractor responsible for all cast in place concrete, placing form work, shoring and reshoring, detailing, fabricating and placing reinforcement steel, and placing concrete per contract drawings;
3. Site Blauvelt Engineers., an independent inspection company retained by Keating;
4. Southwest Rebar Company, Scottsdale, AZ, retained by Fabi to detail reinforcing steel on shop drawings; and
5. Forrest Consultants, Sugarloaf, PA, retained by Fabi to prepare shop drawings for structural steel mesh.

6. Mitchell Bar Placement, Inc., retained by Fabi to place the reinforcing steel and mesh.

## **Construction and Collapse:**

The discussion is generally limited to the area of the collapse.

The slabs, columns and shear wall were cast in an orderly manner. Generally, the slabs on levels P4 thru P8 were cast in intervals of one week or more. For example, slabs on levels P8 thru P4 were cast on October 30, October 18, October 10, October 2-3, and September 22-24, 2003, respectively. Figure 33 indicates the casting sequence of slabs, columns and the shear wall with the dates of casting. These dates have been derived from inspection company reports. The age of the concrete slabs on levels P7, P6, P5 and P4, on the day of the incident, was 12, 20, 28 and 38 days respectively. It is therefore expected that, other than the slab on level P7, the slabs on all other levels had achieved nearly full concrete strength.

Fabi ordered 6,000 psi concrete for the slabs and beams on levels P4 thru P8, though the structural drawings called for only 5,000 psi concrete. The concrete breaking strength, as reported by the inspection company, indicates that the concrete gained strength at the expected rate. Figure 34, 35, and 36 show the results of concrete testing, as reported by the inspection company. The seven and 28-day strengths appear adequate. The average seven day strengths for slabs on levels P4 thru P7 are 4,652, 4,981, 4,567, and 4918 psi respectively. The average 28-day strength for levels P4 thru P7 is 5,570, 5,908, 5,267, and 5,704 psi respectively. Similarly acceptable results are noticed for the concrete used in columns and shear wall. In addition, visual examination of the concrete in the failed members is unremarkable.

The area of collapse is shown in Figures 1 to 8. In the east-west direction, the slabs on levels P8 thru P4 between grid lines 1 and 4, collapsed, completely separating from the exterior columns and the shear wall on grid line 1, but remained hanging from the longitudinal beam on grid line 4. In the north-south direction, the failure extended full length, and although the last bay did not completely fail, it was severely cracked. Four columns (D-1, D.5-1, E.7-1 and F-1) were left standing freely. Column E-1 fell after separating from the slabs. The shear wall on grid line 1 was also left standing freely. During the fall, on level P7, the half tub beam remained connected to the 10" slab and fell in one piece, though cracks were visible at the junction of the half tub beam and the slab. On levels P6 thru P4, the half tub beams separated from the 10" slab and fell among the rubble. The free standing columns and shear wall were eventually demolished in pieces in a controlled manner and saved in a storage yard in Pleasantville, NJ, for examination by interested parties.



## **Discussion:**

### **1. Lack of proper embedment of reinforcing steel marked T418 (S9) and T66¾ (S21) in columns on Grid Line 1**

With the concurrence of the SER and for the sake of economy, Fabi replaced T418 and T66¾ reinforcing steel (see figures 12 and 13) with welded wire fabric mesh, marked as S9 and S21 (see figures 30 and 31) respectively on the mesh shop drawings. S9 and S21 provided an amount of steel equivalent to T418 and T66¾ respectively. But to develop the full strength of S9 and S21, the mesh must be embedded in the column for a specific length of embedment, as per American Concrete Institute (ACI) code. If the mesh placement is inadequate, the efficacy of the reinforcing steel is greatly compromised and the desired flexural strength is significantly reduced, thus compromising the integrity of the structure. These deficiencies were observed in the post collapse inspection of the columns on grid line 1, levels P4 thru P7.

The ACI 318-95 provisions contained in section 12.7 requires S9 mesh to be embedded at least a distance of 8" and S21 a distance equal to 9" to develop full strength. Consistently at all levels, P4 thru P7, the mesh was observed to have been embedded in the column at varying lengths. At some locations, the embedment length was only 1 ½", rendering the steel unable to support any load (see Appendix B for embedment lengths of S9 and S21 mesh). The S21 and S9 mesh did not meet the required embedment length at all locations.

Fabi was responsible for placing the mesh in the manner required by ACI code. Fabi contracted Mitchell Company to place the rebars, and at times provided its own employees to do the same. The placement was to follow the shop drawing indicating that mesh be placed up to the exterior edge of the exterior beam and columns on grid line 1. Mitchell and Fabi employees encountered difficulties placing the mesh extending up to the exterior edge. First, the parapet wall on the exterior portion of the exterior beam, prevented the mesh from extending up to the outer edge. So, employees placed the mesh to the inner edge of the parapet wall. Mitchell and Fabi employees should have brought the discrepancy to the attention of the general contractor/construction manager or the SER for resolution. Rebar placement personnel are not qualified or authorized to make such decisions.

Second, they were unable to meet the shop drawing requirement for placing mesh extending up to the outer edge of the column. The S9 and S21 mesh were prefabricated in widths of 8-feet and 7-feet respectively (see figures 30 to 32). As the span of the beams and location of columns were not multiples of the width of the mesh, the employees generally placed the mesh up to the inner column vertical reinforcing steel. The failure to comply with the shop drawing and standard industry

practice was never brought to the SER's attention. This was a grave error on the part of Mitchell and Fabi. Had the SER been apprised of the problem, he could have provided alternate solutions, possibly either lapping or refabricating the mesh for placement near the column.

During inspections of mesh placement and of reinforcing steel, the independent site inspector, Site-Blauvelt (SB), consistently failed to notice that the mesh was not placed, per the shop drawing, near and at the columns on level P4 thru P7. In fact, the inspector wrote in his inspection reports that all reinforcing steel was properly placed. Construction industry standard practice calls for inspectors to remain vigilant and thoroughly check the reinforcing steel before concrete is placed.

Lack of proper embedment of the reinforcing steel by Fabi significantly compromised the structural integrity of the slab and beams, contributing to the collapse.

## **2. Absence of B49 rebars**

Structural drawings indicate that B49 rebars, i.e., #4@9" reinforcing rebars at the bottom layer, be provided in 10-inch slabs in the E-W direction, terminating near the exterior edge of the edge beam and the columns on grid line 1. Structural drawings are, however, not consistent in indicating such rebars. For example, on S1.34 (see figure 11), B49 is shown in two locations, but on S1.35 (see figure 12), B49 is shown in only one location. Similar inconsistencies are found on S1.36 (see figure 13) on grid line 1. It is believed that the SER intended to indicate B49 at all locations on grid line 1 where the 10-inch slab framed in to the exterior edge beam. Correcting shortcomings such as these, during the shop drawings review process, is an accepted practice in the construction industry.

Fabi contracted with Southwest Rebar to produce shop drawings for reinforcing steel. Southwest detailed the reinforcing steel, but failed to detail B49 rebars on the shop drawing (see figures 24 to 29), even at those locations where they were indicated on the structural drawings (see figures 11 to 13). Southwest typically would discover the inconsistencies in the structural drawings regarding B49 rebars, as discussed above, and then bring inconsistencies to the attention of the structural engineer for clarification. This did not happen. Southwest did not even detail the B49 rebars at the indicated locations on the structural drawings.

Fabi received the shop drawings from Southwest and did not notice the omission of the B49 rebars. Fabi then forwarded the drawings to SER for approval. SER also failed to notice the absence of B49 rebars on the shop drawings and returned them without any comment on the B49 rebars. However, as per the construction industry's practice, it is Fabi's unequivocal responsibility to detail, supply, and place the reinforcing bars, per contract structural drawings, regardless of whether or not the rebars are detailed on the shop drawings.

Unfortunately, no one noticed the absence of the B49 rebars and so they were never placed on levels P4 thru P8. Absence of the B49 rebars compromised the integrity of the structure and contributed to the collapse.

**3. Cracks at exterior slab/beams on Levels P4 thru P7, along the interior long face of columns and the two short sides of the columns on Grid line 1**

Weeks before the collapse, Fabi employees noticed that on every level, P4 thru P7, as soon as the shores were "cracked," cracks began to appear on the concrete surface at the junction of the exterior beams and interior face of columns. The cracks propagated along the two short sides of the column at a 45 degrees angle toward the exterior. Employees consistently noticed cracks at almost every exterior column. Some employees reported that the cracks were up to 1/4" wide. The employees further reported that the cracks extended the full depth of the exterior beams, but did not go through the thickness of the filigree slab. Generally, the shores were cracked four days after concrete was cast.

Fabi employees pointed the cracks out to the Fabi superintendent at the job site. The superintendent inspected the cracks but failed to bring them to the attention of the SER, despite the fact that the cracks were in critical locations at the junction of exterior beams and columns, were uniform in appearance at all levels and at all columns, and were sufficiently wide and deep to cause concern. This was a serious matter that should have been brought to the immediate attention of the SER, per standard construction industry practice. If the SER had been informed, the lack of B49 rebars and inadequate embedment lengths of S9 and S21 could have been discovered and solutions devised. An opportunity was lost to remedy the situation and perhaps to avert the incident. The disregard for the importance of the cracks contributed to the collapse.

**4. Shores and re-shores**

Shores and re-shores on levels P5 thru P7, in the area bound by grid lines 1 & 4 and B.9 & F.6, collapsed with the slabs during the incident. They were either scattered in the rubble or trapped between the fallen slabs. The type and number of shores and re-shores could not be accurately ascertained. However, during the debris removal, it is expected that the total number of shores and their relative locations among the fallen pieces will be determined.

OSHA conducted a survey of existing shores and re-shores, adjacent to the failed area, bound by grid lines 4 & 10 and B.9 & G.1, levels P3 thru P7 (see Appendix A). The survey's purpose was to establish the general arrangement of shores and re-shores used by the contractor. Figures A-11 thru A-15 show the locations and type of shores on levels P3 thru P7. The contractor used three types of shoring frames (Peri, Burke and Waco), and three types of single post shores (Bill Jax N350, N260 and Atlas Mecs). Figures A-1 thru A-10 show different types of shores.

The Contractor generally used Peri frames for 6-foot post spacing to support beams on grid lines 4, 6, 8 and 10, levels P6 and P7. To support the slab, the contractor used Burke (Aluma) frames between grid lines 4 & 6, 6 & 8 and 8 & 10 levels P6 and P7. On level P5, single post shores supported both the beam and slab. On level P5, the posts consisted of three types, as discussed earlier. There were no shores present on level P4, between grid lines 4 & 10 and B.9 & E. The area between grid line 1 & 4 and grid line F.6 & G.1, on levels P5, P6 and P7, which did not collapse, indicated that shoring frames were used to support the exterior beam and slab.

Since load tables were not available, OSHA computed safe carrying capacities of different types of surveyed shores and determined that all shores had adequate capacities, when used properly.

Project specifications required that one level of shores and three of re-shores be provided during concrete casting (see specifications Section 03300). On October 30, 2003, during casting of level P8, Fabi provided one level of shores between levels P7 and P8, and one level of re-shores between levels P6 and P7. Some witnesses reported that the re-shores between levels P5 and P6 had been removed a day before the incident. At the time of the incident, level P5 had practically no re-shores. A site visit was made to inspect whether any re-shores were trapped between the fallen slab of levels P5 and P6. Standing at the edge of the beam on grid line 4, only a few fallen re-shores could be observed confirming the eyewitness reports that the majority of re-shores had been removed a day earlier. Therefore, at the time of the incident, the weight of the wet concrete and construction loads on level P8 were being supported by only two levels, P7 and P6. This was a direct violation of the contract requirements and industry practice.

By providing only two levels of supports, levels P7 and P6 were subjected to loads higher than intended in the project specifications. In previous instances when levels P7, P6, and P5 were cast, witnesses reported that one level of shores and three levels of re-shores were provided in accordance with the project specifications. Each level, therefore, supported approximately one-quarter of the wet concrete load and other construction loads. But on the day of the incident, for the first time on this project, one level of shore and one level of re-shores were provided.

The industry practice is to apply a load factor of 1.4 to concrete dead load, construction load of 50 psf and other formwork loads to determine the minimum number of levels of shores and re-shores needed. Based upon the above, one level of shore and three levels of re-shoes are required, provided all levels share the loads equally. As stated earlier, the project specifications also called for the same.

OSHA's standard requires that the formwork be designed to support the intended loads "without failure". In other words, load factors, discussed above, were not mandatory to meet OSHA's standard. Given the ultimate design strength of the slab to be 260 psf ( $1.4 \times 125 + 1.7 \times 50 = 260$  psf), it could be permissible, under OSHA standard, to use one level of shore and one level of re-shore to support the above

loads, provided the slab was not structurally distressed. 260 psf as the ultimate strength of the slab could only be safely assumed if the slab was free of cracks and distress. Fabi was well aware of structural distress at the critical location of the slab/column joints at a number of levels and still proceeded to use one level of shore and one level of re-shore.

## **5. Cracking of the shores**

OSHA standards require that the shores be removed when the concrete has gained “sufficient strength”, as determined by tests. Fabi cracked the shores at approximately three to four days after the concrete was cast, without obtaining test results. Fabi, however, reported that it relied on historic data of past breaking strengths of concrete to crack the shores without ascertaining the concrete strengths for each level. Further, Fabi ordered 6,000 psi concrete instead of the specified 5,000 psi concrete, which gave additional confidence to the contractor. While relying on historic data and higher concrete strengths might seem valid, cracking shores without test results presents an unacceptable level of risk.

## **6. Structural Design:**

One way Slab:

The 10” slab is designed as a one way slab spanning 48 feet between grid line 1 and 4, using ADOSS program. The program was run by Mid State Filigree (MSF). The positive moment was computed to be 47’K. The structural drawings indicated 46.7’K between the grid line 1 shear wall and grid line 4, and 30’K between the 16” deep beam on grid line 1 and 30” deep beam on grid line 4. Computations for 30’K moment were not available from SER. It is not readily understood why the change was suddenly made from 47’K to 30’K. If 30’K moment was derived on the basis of fixity of the one way slab on grid line 1, then the 16” beam must also be subject to the fixed end moment of the slab. There is no indication that the beam on grid line 1 was designed for any substantial torsion. MSF, however, provided 47’K flexural capacity of the one way slab at all locations even where SER indicated 30’K on the structural drawings. The longitudinal 46” x 16” beam on grid line 1 was designed as a continuous beam in north-south direction, again by MSF, to support the loads coming from the one way slab described above. The design was later reviewed and approved by SER. SER, contrary to the industry practice, relied heavily upon MSF for the structural design of the one way slab and longitudinal beam on grid line 1 for the exterior bay. The normal practice is that the SER designs the slabs and beams, and provides necessary information to the filigree producer to fabricate the precast panels. The filigree producer could ask SER for additional information, if needed. The responsibility of design of all structural members, other than precast filigree members, must rest with the structural engineer of record.

Slab/Beam-Column Joint:

OSHA evaluated the slab/beam-column joints on grid line 1. As a result of the evaluation, OSHA has concluded that the structural engineer of record improperly designed the slab/beam-column joints on grid line 1. The design was determined to be deficient to support the code prescribed loads. The joints in question are located on grid line 1, levels P3 and above. The structural design did not meet the design standards of the industry. The top reinforcing steel in the east-west direction was well under-proportioned, violating accepted industry standards and basic engineering principles. The slab/beam-column joints on grid line 1, columns E.7 and F, where T66 $\frac{3}{4}$ , in lieu of T418, was indicated have been determined to be satisfactory in flexure.

OSHA requested SER to furnish original computations to demonstrate that the slab/beam-column joints had been properly proportioned to transfer the loads to the column. SER failed to submit original computation but performed new computation justifying the use of T418. The computations furnished to OSHA were deemed unsatisfactory. SER erroneously reduced the flexural demand of the negative bending by the shear capacity of the contact area between the column and the beam. In reality, no such reduction could be taken for flexure. For shear transfer, however, consideration of the contact area is valid. SER used lower load factors of 1.2 and 1.6 for dead and live loads respectively but failed to reduce the strength reduction ( $\phi$ ) factor to 0.8 from 0.9. SER used a reaction of 174 kips, though the ADOSS run indicated a reaction of 183 kips. SER has used a width of eight feet to transfer the moment to the column which is generally permitted in a two way slab design. ACI 352.1R recommends a width equivalent to six feet unless the edge beam has been designed for torsion. Filigree and SER's drawings indicate that the edge beam has not been designed for torsion. In fact, the 16" deep beam does not possess adequate torsional rigidity to provide meaningful rotational restraint to the one way slab.

OSHA analyzed the joints on grid line 1, column D and D.5 using three different methods to cover all possibilities. In all methods, the joint failed to satisfactorily support the code prescribed loads.

#### Method 1:

The 10" slab was assumed to span 48 feet between grid line 1 and 4 as a one way slab. The positive moment was computed and the magnitude of 47'K was determined to be satisfactory. The reaction of the one way slab was then applied over the north-south beam and resulting flexural demands were verified. The negative and positive moments shown on the structural drawings for the north-south beams were generally satisfactory. Then, the question of transferring the reaction of the north-south beam to the column was addressed. As the c.g., of the north-south beam did not coincide with the centerline of the column on grid line 1, an eccentricity of 12  $\frac{1}{2}$ " to the face of the column was considered. Flexural and shear forces were computed. It was determined that T418 did not provide adequate flexural capacity to support the code prescribed dead and live loads. In addition, it was determined that T418 can not

adequately support the factored dead load of the slab plus the imposed factored loads from one level of shores, three levels of re-shores, weight of the wet concrete and other construction loads per industry practice. In fact, T418 can barely support the factored dead load of the slab itself. However, the slab could support its own unfactored dead load plus the unfactored dead load from one level of shore, three levels of re-shores, weight of the wet concrete and construction load of 50 psf ( all unfactored), provided that the reinforcements indicated on the structural drawings are properly placed.

#### Method 2:

The joint was considered as a slab-column joint and analyzed, in the east-west direction, on the premise that the slab acted as a two way slab near the column. The beam could be considered a thickened slab at the end because a large number of longitudinal re-bars of the beam were not and could not be placed through the confines of the column. It was determined that the slab/beam-column joint could not support the intended loads, failing in flexure and shear under code prescribed dead and live loads. It also failed in flexure and shear under its own dead load and the loads from the shores, re-shores and the distributed load of the wet concrete. In fact, the joint could not adequately support the dead load of the slab. The joints with T66 $\frac{3}{4}$  rebars were, however, determined to be adequate in flexure but the shear stresses were higher than the allowable.

#### Method 3

A limited non-linear finite element analysis was performed using the ANSYS program, analyzing the slab with T418 and other reinforcing steel indicated on the structural drawings. The purpose of the analysis was to determine whether the T418 bars could safely support the code prescribed loads without reaching their yield values. The analysis indicated that T418 yielded even at the unfactored dead and live loads.

As stated earlier, the beam/slab-column joint fails in all three methods for the code prescribed loads. OSHA believes that the Method 1 closely depicts the actual behavior of the beam/slab-column joint.

It is interesting to note that the SER was afforded an opportunity to correct the design flaw during a review of the shop drawings at which time he could have re-examined the slab/beam-column joint. A look at the shop drawing should have raised an alarm in the mind of the reviewer, as to why a large difference existed between the amount of steel in S9 (equivalent to T418) and S21 (T66 $\frac{3}{4}$ ) under similar conditions. S21 provided six times more reinforcement than S9.

## **7. Lack of clarity in structural drawings**

The SER produced project structural drawings that contained certain inconsistencies. The B49 bottom rebars were not consistently shown on plans for levels P4 thru P7. On drawing S1.34 (see figure 11), B49 is shown at two locations, but on S1.35 and S1.36 (see figures 12 and 13), B49 is shown at only one location. Extent of the placement of B49 rebars is not shown. It is not clearly understood whether or not B49 was intended to be placed in the slab framing into the shear wall.

For the 10" slab, levels P4 thru P7, negative reinforcing steel, T418 was abruptly changed to T66¾. It is not clear why the presence of the ramp could significantly increase the negative reinforcing to T66¾ from T418, an increase to 600% (+). The extent of the use of T418 and T66¾ in the north-south direction is not indicated on the structural drawings.

A 6-inch precast parapet wall was formed into the edge beam on grid line 1. Mid State Filigree (MSF) drawings of the exterior beam indicated that the first leg of shear stirrup, in the transverse direction, was located 7-inches from the edge of the filigree exterior beam. The filigree panel was set 1½-inches inside the exterior face of the column, and the first shear stirrup was placed 8½-inches, in the transverse direction, from the exterior face of the column (see figure 16). The exterior face of the parapet wall was in line with the exterior face of the column on line 1. Given the vertical reinforcing steel of the column going thru the edge beam, and given the standard practice of placing the top beam rebars between the shear stirrups, only one reinforcing rebar of the north-south beam, could be placed near the outer fringes of the inside face of the column. This weakened the integrity of the connection between the edge beam and the column in the north-south direction. SER approved MSF's drawings and the shop drawings of the reinforcing steel, failing to notice that the top rebars could not be placed within the confines of the column. SER did not instruct either Fabi or MSF to revise the MSF drawings or modify the reinforcing shop drawings to place at least a few rebars within the confines of the column. Post-collapse inspection revealed that at a few locations, one top reinforcing rebar was placed near the inside edge of the column, while at other locations, none was placed.

## **8. Flawed inspection of Site Blauvelt Engineers**

Site Blauvelt Engineers (SB) was retained by the general contractor/construction Manager (GC/CM) to conduct independent inspections of the site, take concrete samples and provide test results of concrete strengths, among other things. One of the prime responsibilities of SB was to inspect the placement of reinforcing rebars and structural welded wire fabric mesh before concrete was cast over them. Reinforcing steel must be placed in accordance with approved shop drawings and contract structural drawings. If the placement does not follow contract documents or standard industry practice, SB must report immediately to GC/CM for remedial measures. Once the concrete was placed, it was too late to take corrective measures.



SB's inspectors were present on the days that levels P4 thru P8 were cast. They took samples of concrete and inspected the placement of reinforcing rebars and structural mesh. For each visit, they provided a report to GC/CM for levels P4, P5, P6, P7 and P8. The inspector recorded in his report that "steel was checked and verified as accurate upon completion for size, spacing, number, cover, lap, clearance, cleanliness, and overall placement in reference to provided shop drawings from KBC." Post-collapse inspection of the recovered column and shear wall pieces revealed serious discrepancies in the placement of reinforcing steel.

- (a) The structural mesh, S9 and S21, did not have the required embedment in the columns and shear wall in the east-west direction on grid line 1, which severely reduced their strength. The shop drawing indicated that the mesh was to be placed extending up to the exterior face of the column. Industry standards (e.g., American Concrete Institute (ACI)) require minimum embedment lengths for the reinforcing rebars and mesh. The contract specifications required the contractor to follow industry standards. If the vertical reinforcing rebars of the columns or the width of fabricated mesh interfered and presented any difficulty, the SER or GC/CM should have been advised so they could take remedial measures. This was not done.
- (b) The north-south reinforcing rebars of the beam were placed outside the confines of the columns on grid line 1, levels P4 thru P7. There was a conflict of standard practices in this situation. On the one hand, the practice is to place as many reinforcing rebars within the width of column to secure the connection between the beam and the column. On the other hand, the practice is also to place reinforcing rebars within the shear stirrups of the beam which would have placed only one reinforcing rebar within the width of the column. The shop drawing and the structural drawings schematically showed the reinforcing rebars outside the width of the column for the sake of clarity. The inspector should have brought the conflict to the attention of the SER and GC/CM for immediate resolution. This was not done.
- (c) The mesh S9, in the 10-inch slab framing into the shear wall, was not placed in accordance with the contract drawings or standard industry practice. Post-collapse inspection of the recovered shear wall indicated that the mesh had been placed in the slab at varying depths. For about half the length of the shear wall, the mesh was placed at mid-depth of slab thickness, significantly reducing the flexural capacity of the mesh.

All of the above contributed to the collapse.

#### **9. Lack of shoring drawings at the construction site for the area of the collapse**

Fabi did not have the shoring plan for the area bound between grid lines 1 to 6, and B.9 to G.1, levels P4 thru P8. OSHA standard requires that the shoring plan be

available at the construction site to ensure that slabs and beams have been properly shored and re-shored.

### **Conclusions:**

1. Fabi Construction and Mitchell Bar Placement, Inc. did not provide the required embedment length for the welded wire mesh at the intersection of the exterior columns and the slab/beam on grid line 1, levels P4 thru P7, per contract requirements and standard industry practice. Correct placement of rebars and mesh were crucial to the integrity of the structure.
2. Fabi Construction failed to detail, fabricate and place bottom reinforcing steel, identified as B49, on grid line 1, levels P4 thru P7, as required by the structural contract drawings. Omission of the reinforcing steel compromised the integrity of the structure.
3. The Fabi superintendent failed to seek the immediate attention of the general contractor/construction manager or the structural engineer of record when Fabi employees brought to his attention the cracks around the exterior columns on grid line 1 at levels P4 thru P7. The employees noticed consistent cracks at the interior long face of the columns extending at an angle of forty five degrees towards the edge of the slab/beam. These cracks should not have been dismissed as shrinkage cracks because of their uniformity, depth and pattern on all levels. The collapse could have been averted if immediate attention was sought.
4. Fabi failed to re-shore an adequate number of floors, as required by the project specification, at the time concrete was being cast on level P8. Only one level was re-shored instead of the required three. Levels P6 and P7 supported the dead load of wet concrete instead of four levels (i.e., P7, P6, P5 and P4). Given the fact that cracks were earlier noticed at and around the exterior columns, the fewer levels of re-shores were highly detrimental to the integrity of the structure. This contributed to the collapse.
5. Fabi ordered premature removal of re-shores between levels P5 and P6. Given the presence of cracks at and around the columns, premature removal of shores created hazardous situations. This contributed to the collapse.
6. Shores were "cracked" without verifying that the concrete had reached "sufficient strength", as required by the OSHA standard.
7. Fabi Construction did not prepare shoring drawings for the area of the collapse, levels P4 thru P7, in violation of the OSHA standard.
8. Site Blauvelt Engineers, contracted by the general contractor to perform an independent inspection, performed poorly in inspecting the placement of reinforcing steel. He failed to notice that the structural mesh did not have proper embedment at the exterior columns on grid line 1, levels P4 thru P7, to achieve full strength. It was expected of him to immediately notify the general contractor of the discrepancy before the concrete was cast over them. The inspector failed to

check whether the reinforcing steel and mesh were placed in accordance with approved shop drawings.

9. The structural engineer did not exercise reasonable diligence in approving the shop drawings, which did not contain the bottom rebars, marked B49 on the contract drawings. He further failed to comment on the desired placement of the longitudinal rebars of the exterior slab/beam within the confines of the columns. The structural engineer was in a unique position to address the integrity of the slab/beam connection to the columns as he had access to all information including the intent of his design.
10. The structural design of the slab/beam-column joints on grid line 1 was flawed. The structural engineer of record improperly proportioned the slab/beam-column joints, in the area of the collapse, to support the code prescribed loads.
11. The structural engineer relied heavily on the filigree fabricator for the structural design of the exterior bay between grid line 1 and 4, levels P4 thru P8. The structural engineer did not conduct independent structural calculations to proportion the negative reinforcements, shear reinforcement, and potential torsion on the exterior beam. The responsibility of the design rested solely with the structural engineer of record.
12. The structural drawings lacked clarity. On certain levels B49 was indicated, while on other levels it was not. The section thru the exterior slab/beam-column was not shown on the contract structural drawings to clarify the position of reinforcing rebars in each direction, column reinforcements, or beam shear stirrups. Lack of clarity resulted in only one, and in some cases, no rebar in the confines of the column.
13. Concrete strength is not a suspect in the collapse.
14. Wind was not a causal factor.
15. The activities of the structural engineer of record in connection with this project were not covered by the construction standards, and therefore were considered to be outside OSHA's jurisdiction.





Figure 1. General View of the Collapsed Portion of the Parking Garage (Looking toward the Project South).





Figure 2. General View of the Collapsed Portion of the Parking Garage (Looking toward the Project Southeast).



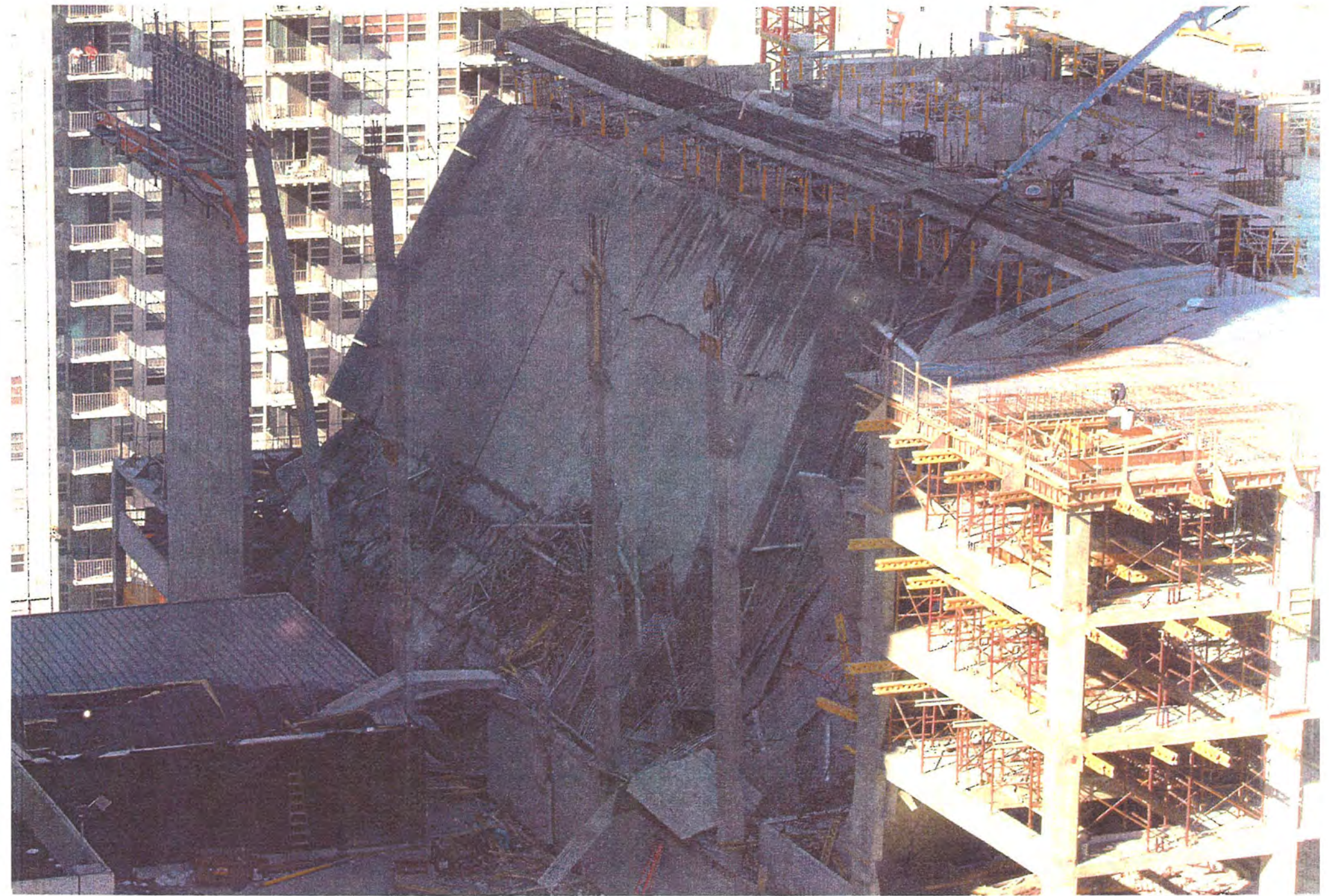


Figure 3. General View of the Collapsed Portion of the Parking Garage (Looking toward the Project Northeast).





Figure 4. Surface of the Collapse between Shear Wall and Column 1-D (Looking toward Project East).



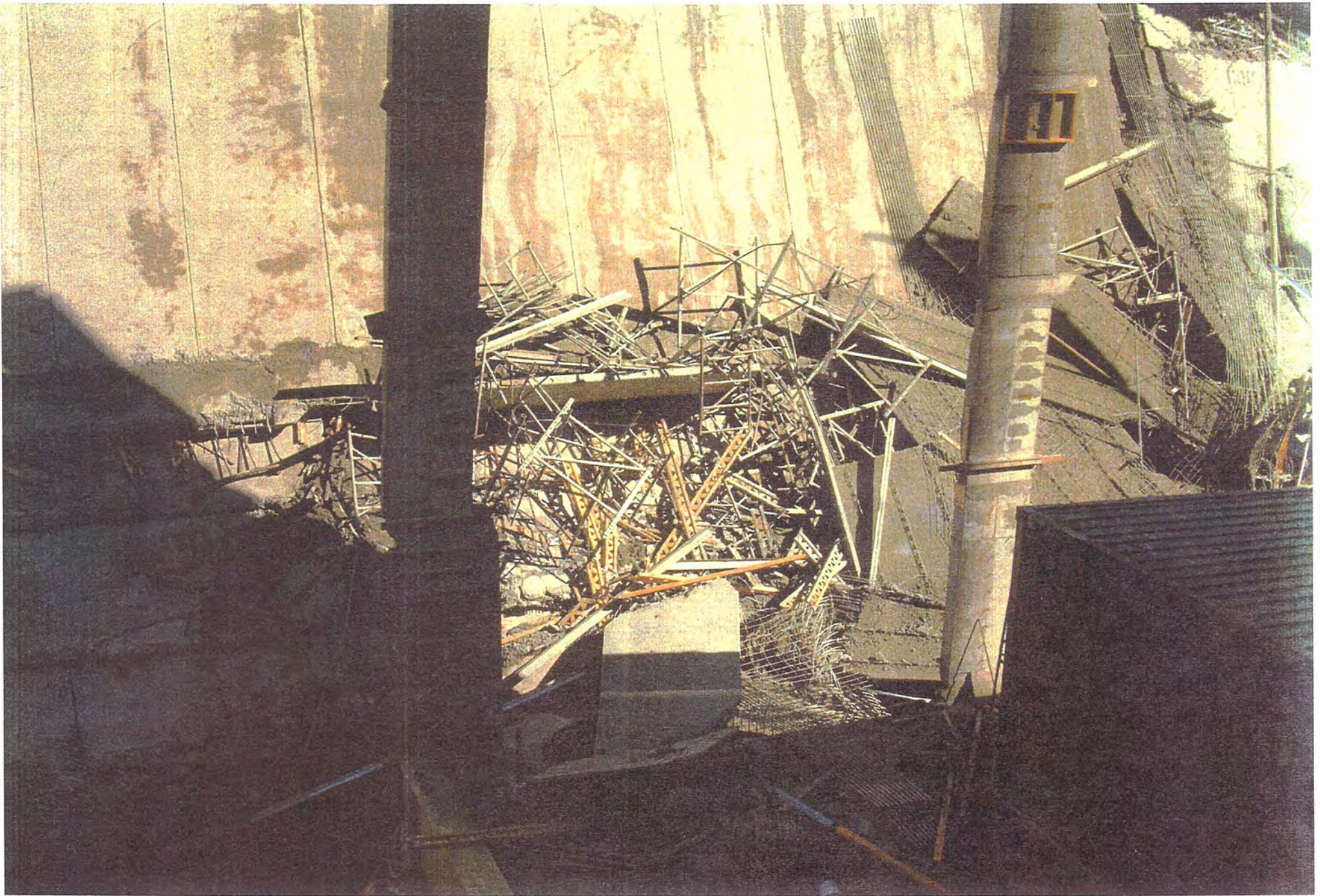


Figure 5. Surface of the Collapse between Columns 1-D and 1-D.5 (Looking toward Project East).





Figure 6. Surface of the Collapse between Columns 1-D.5, 1-E (Fallen), 1-E.7, and 1-F (Looking toward Project East).



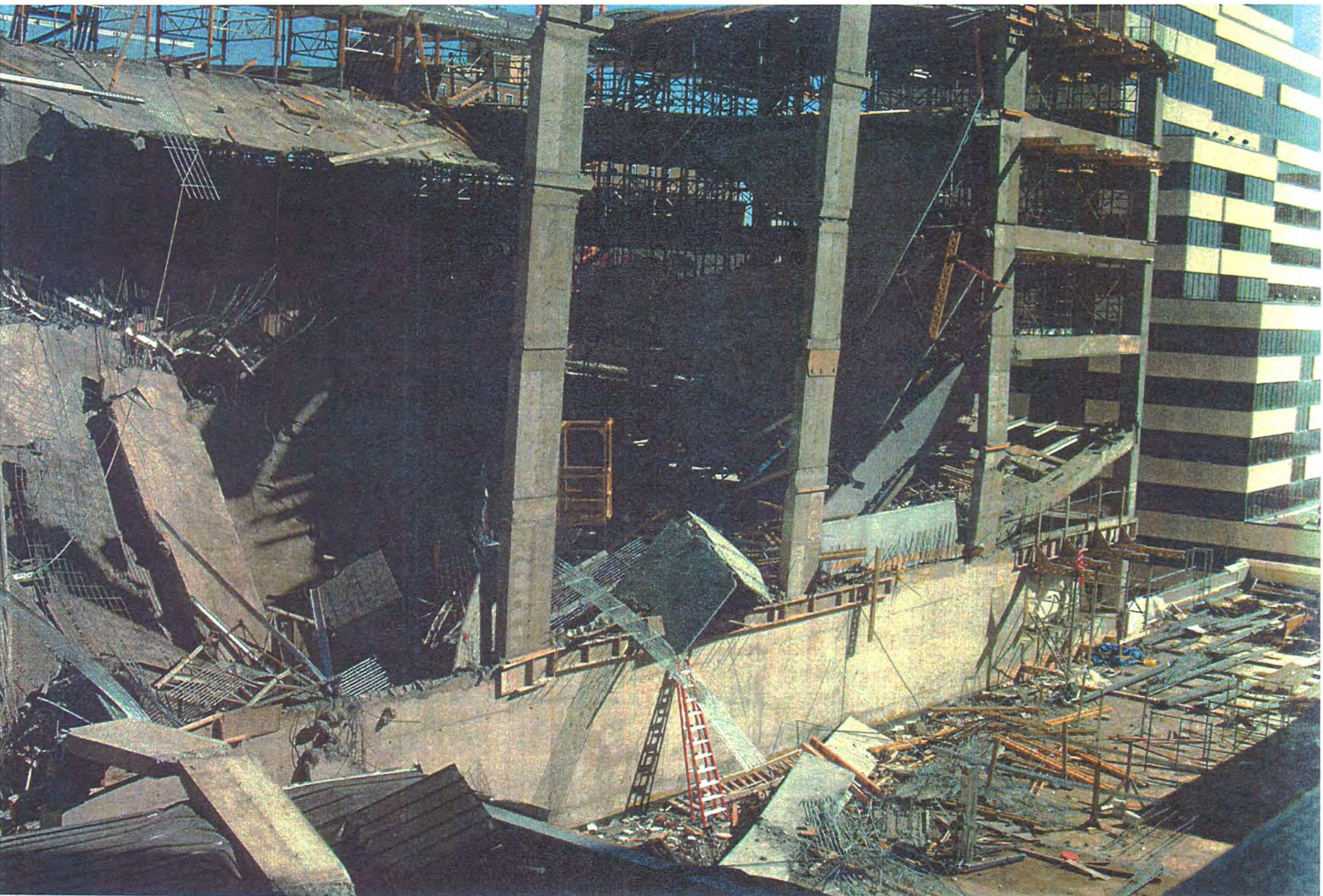


Figure 7. Surface of the Collapse between Columns 1-E (Fallen), 1-E.7, 1-F, 1-F.6 and 1-G1 (Looking toward Project Southeast).



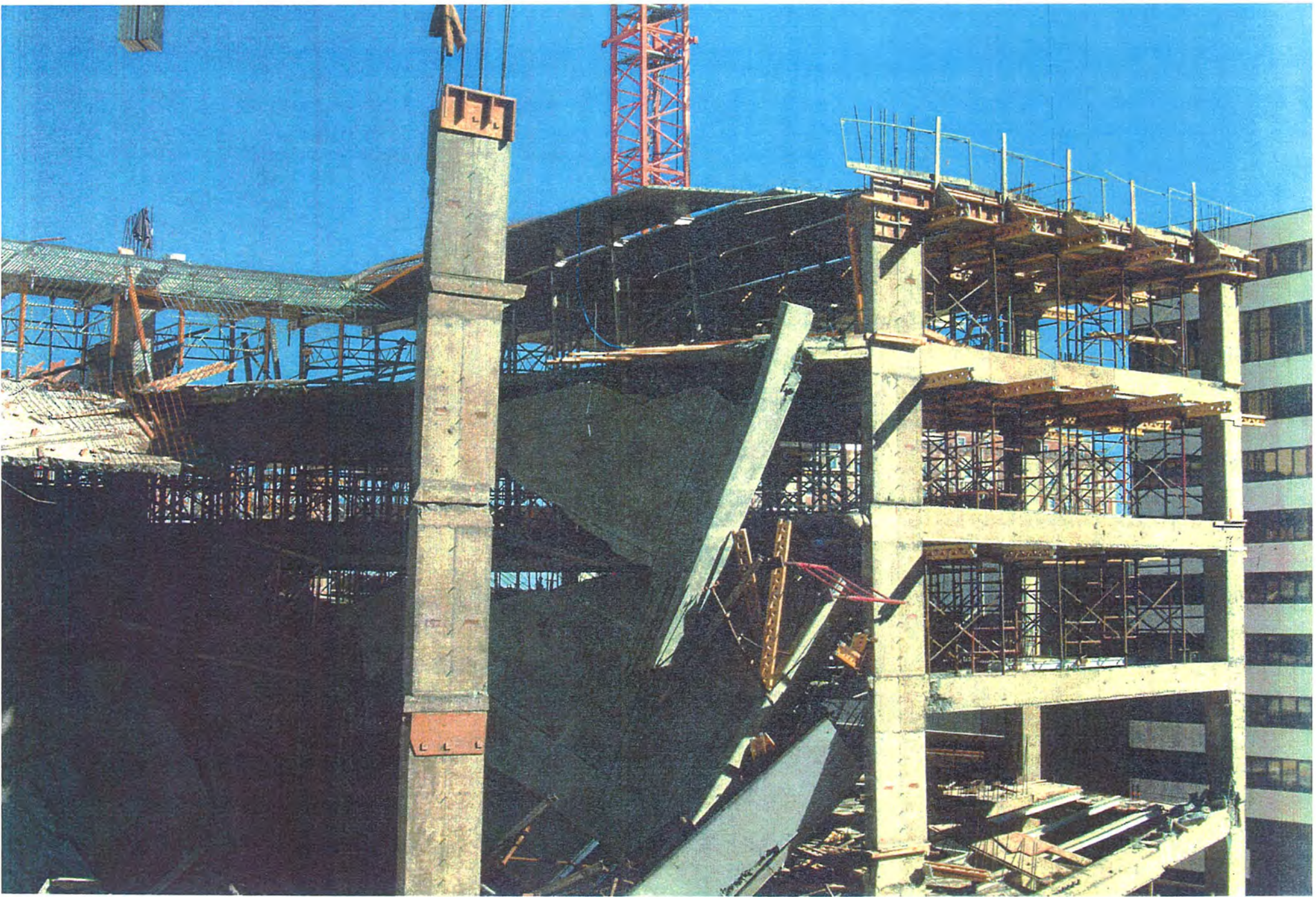


Figure 8. Surface of the Collapse between Columns 1-F, 1-F.6 and 1-G1 (Looking toward Project Southeast).



# GENERAL NOTES:

## I - CODES

1. THE BOCA NATIONAL BUILDING CODE/1996 AS MODIFIED BY THE STATE OF NEW JERSEY (BOCA/'96)
2. AMERICAN INSTITUTE OF STEEL CONSTRUCTION "SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS - ALLOWABLE STRESS DESIGN AND PLASTIC DESIGN", JUNE 1, 1989 ("AISC").
3. AMERICAN CONCRETE INSTITUTE "BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE" ACI 318-95 ("ACI 318")
4. STEEL JOIST INSTITUTE "STANDARD SPECIFICATIONS, LOAD TABLES AND WEIGHT TABLES FOR STEEL JOISTS AND JOIST GIRDERS", 1992 ("SJI").

## II - MATERIALS

UNLESS OTHERWISE SHOWN OR NOTED ON DRAWINGS:

### 1. STRUCTURAL STEEL:

ALL ROLLED SHAPES 20 PLF AND HEAVIER:	ASTM A572, GRADE 50
ALL ROLLED SHAPES LIGHTER THAN 20 PLF:	ASTM A36
ALL PLATES AND CONNECTION MATERIAL:	ASTM A36
ALL TUBULAR SECTIONS:	ASTM A500, GRADE B
ALL PIPE SECTIONS:	ASTM A53, GRADE B
ANCHOR BOLTS, U.O.N.:	ASTM A307 2.

### 2. METAL DECK:

FABRICATE FROM ASTM A446 GRADE A STEEL WITH ASTM A525 G60 GALVANIZING. SIZE AND GAGE AS NOTED ON DRAWINGS. U.O.N.,  
FLOOR DECKING SHALL BE COMPOSITE DECK WITH CONFIGURATION THAT PERMITS FULL AISC SHEAR CONNECTOR VALUE.

### 3. SHEAR CONNECTORS:

3/4" DIAMETER X 3<sup>3</sup>/<sub>16</sub>" HEADED STUDS, U.O.N.

### 4. CAST-IN-PLACE CONCRETE:

FOUNDATIONS:	4 KSI NORMAL WT. (U.O.N.)
SLABS ON GROUND:	3 KSI NORMAL WT.
FORMED SLABS:	AS NOTED ON DRAWINGS.
SLABS ON METAL DECK:	3 KSI LT. WT. 115 PCF.
COLUMNS AND WALLS:	AS NOTED ON DRAWINGS.

### 5. REINFORCEMENT:

DEFORMED BARS:	ASTM A615, GRADE 80.
WELDED WIRE FABRIC:	ASTM A185, GRADE 60.
GARAGE PARKING AND RAMP	
AREA TOP BARS - ALL BARS	ASTM A615, GRADE 80 WITH
W/IN 2" OF TOP OF SLAB:	ASTM A767 CLASS I GALVANIZING.

### 6. WELDING ELECTRODES:

E70XX LOW HYDROGEN.

### 7. BOLTING MATERIALS:

ASTM A325 OR A490, U.O.N.

### 8. LIGHT GAGE FRAMING:

ASTM A446, A570 OR A611;  
GRADE 50 FOR 16 GAGE AND  
HEAVIER, GRADE 33 FOR 18  
GAGE AND LIGHTER; WITH  
ASTM A525 G60 GALVANIZING.

CONSTRUCTION  
DOCUMENTS

SHEET TITLE

GENERAL NOTES

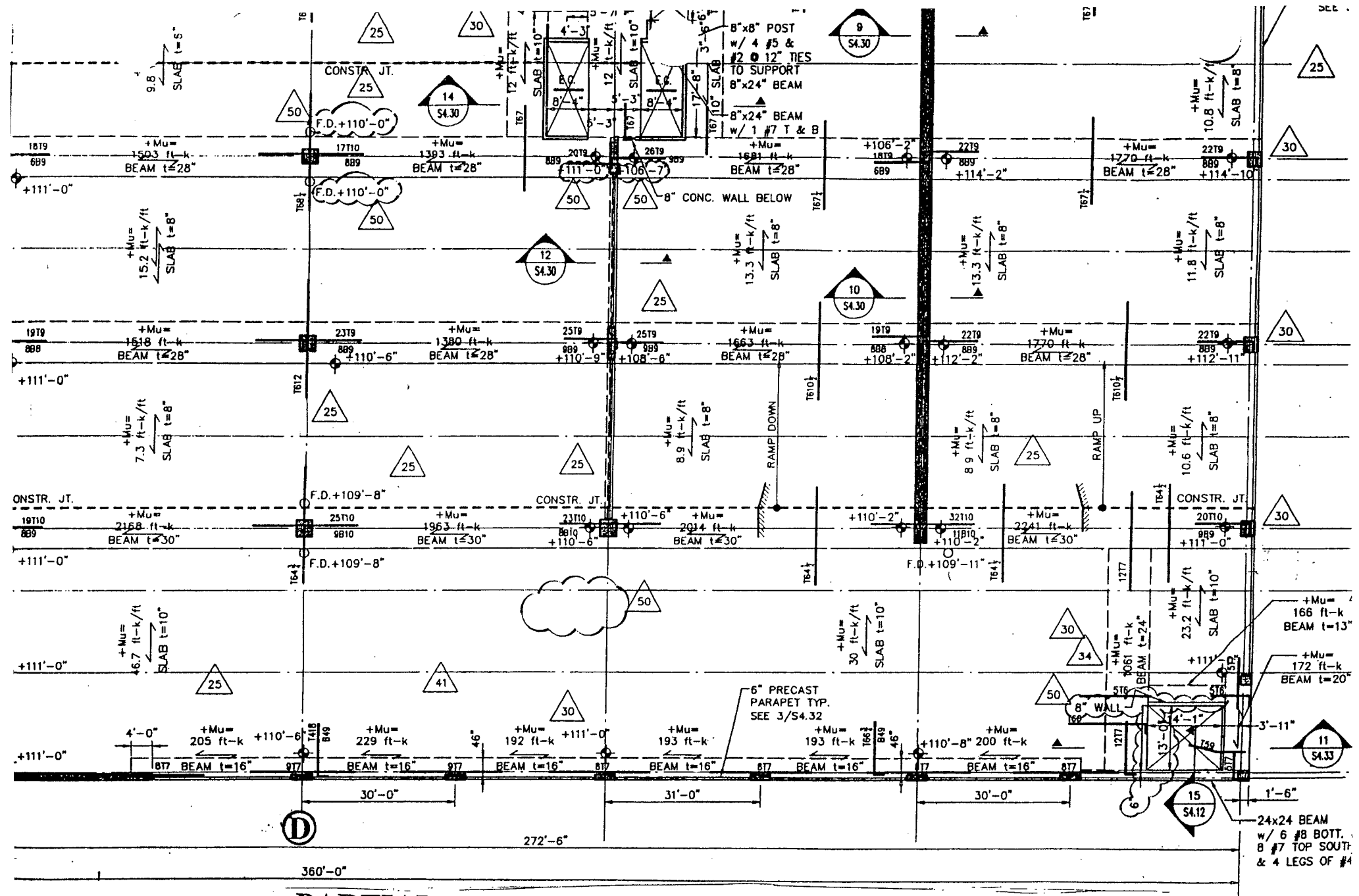
SCALE	DATE	SHEET NO.
N.T.S.	27 JUNE 2001	<b>S0.01</b>
DRAWN BY	PROJECT NO.	
SAG	3626	

**DCE** DESIMONE  
CONSULTING ENGINEERS  
20 WATERSIDE PLAZA - LEVEL C  
NEW YORK, NEW YORK 10001  
TEL. 212-512-1211 FAX. 212-512-1211

FIGURE 9

(NTS)

### FIGURE 10

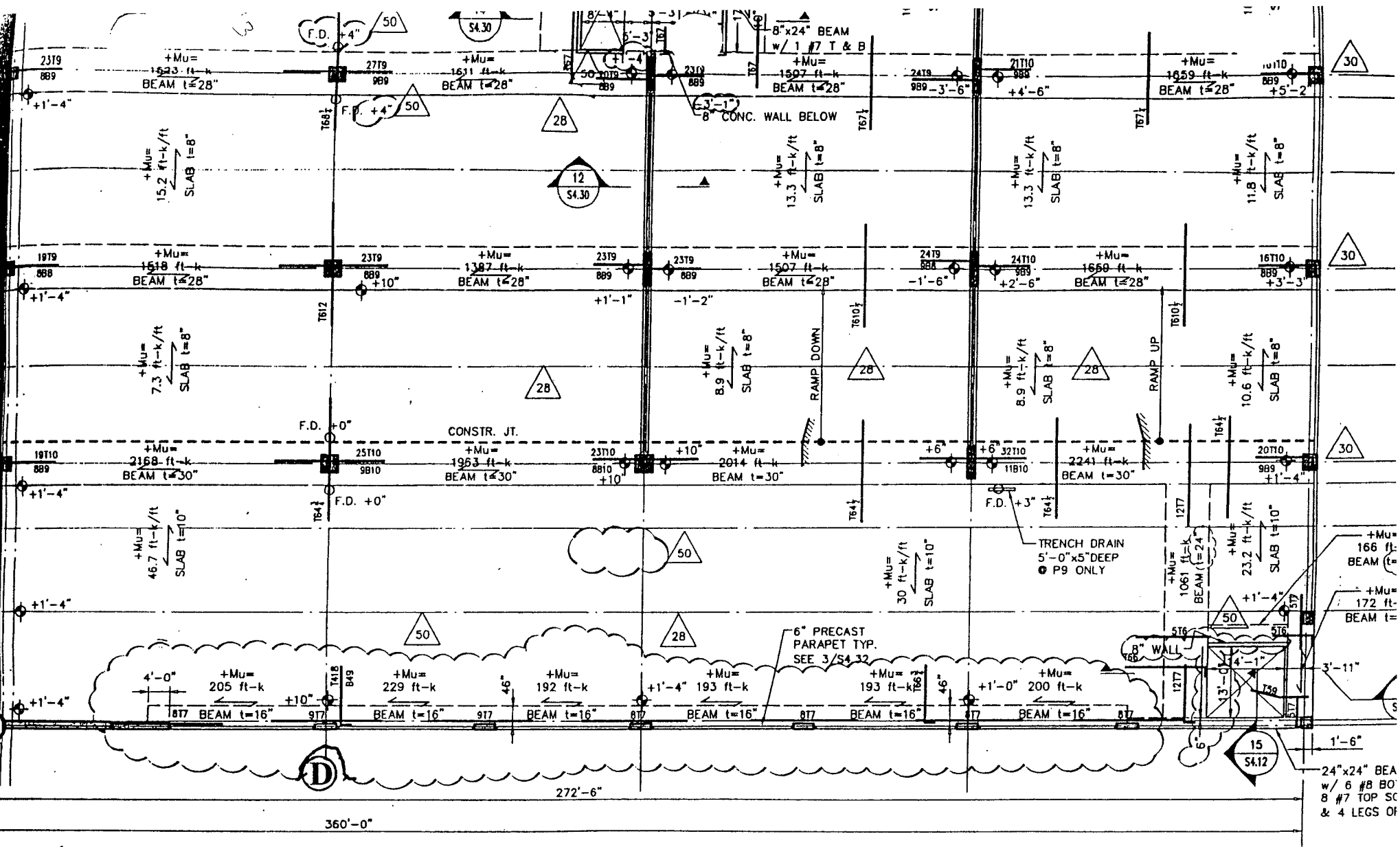


**PARTIAL P4 FRAMING PLAN** (CONTRACT DRAWING S-1.34, REV. 50)  
(NTS)

**FIGURE 11**

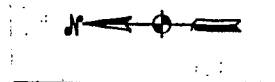


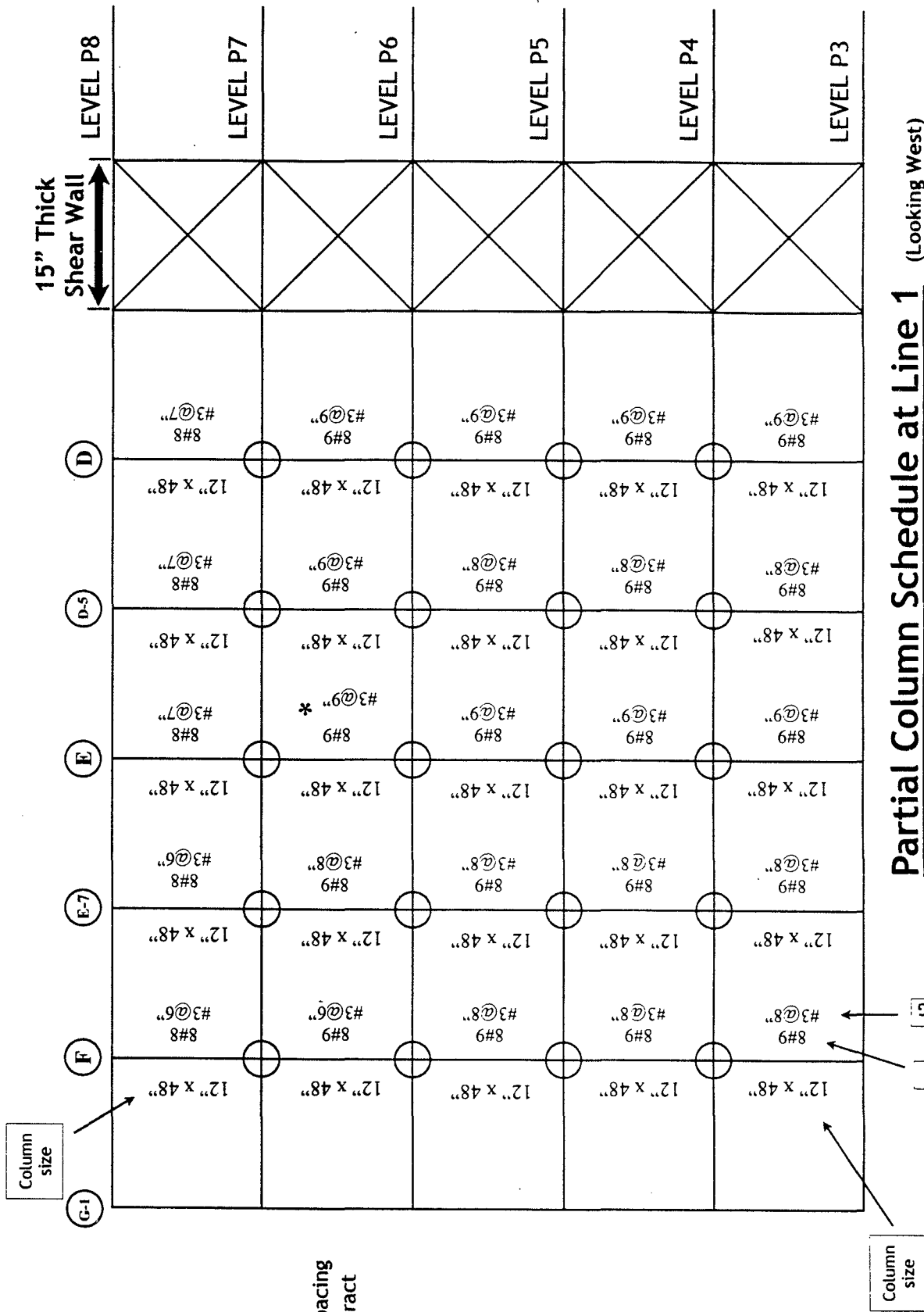




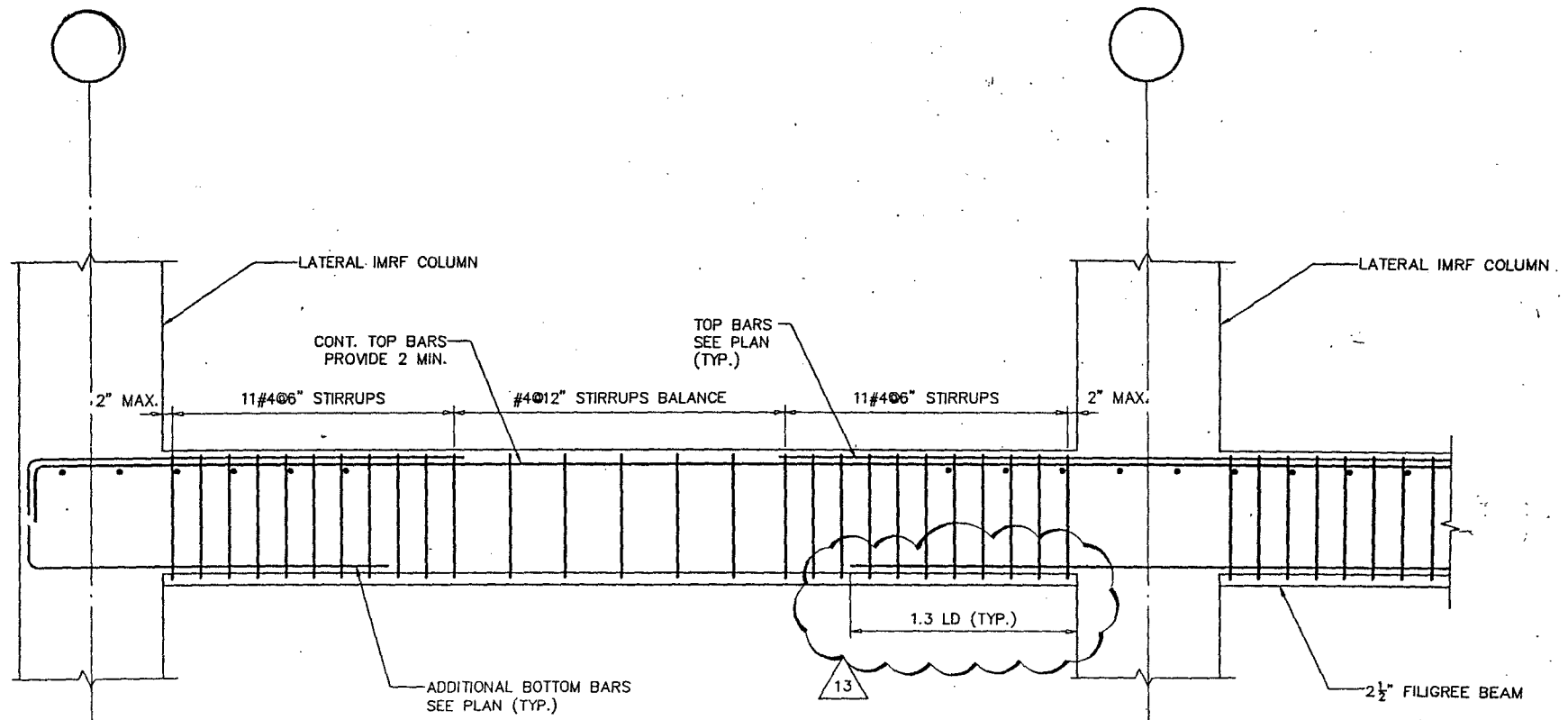
**PARTIAL P6 TO P9 FRAMING PLAN** (CONTRACT DRAWING S-1.36, REV. 50)  
(NTS)

**FIGURE 13**





**FIGURE 14**



- \*NOTES: 1. SEE TYP. CONCRETE BEAM DETAIL FOR TOP BAR LENGTHS  
 2. REFER TO COLUMN SCHEDULE DWG S3.02 FOR LOCATION OF IMRF COLS.

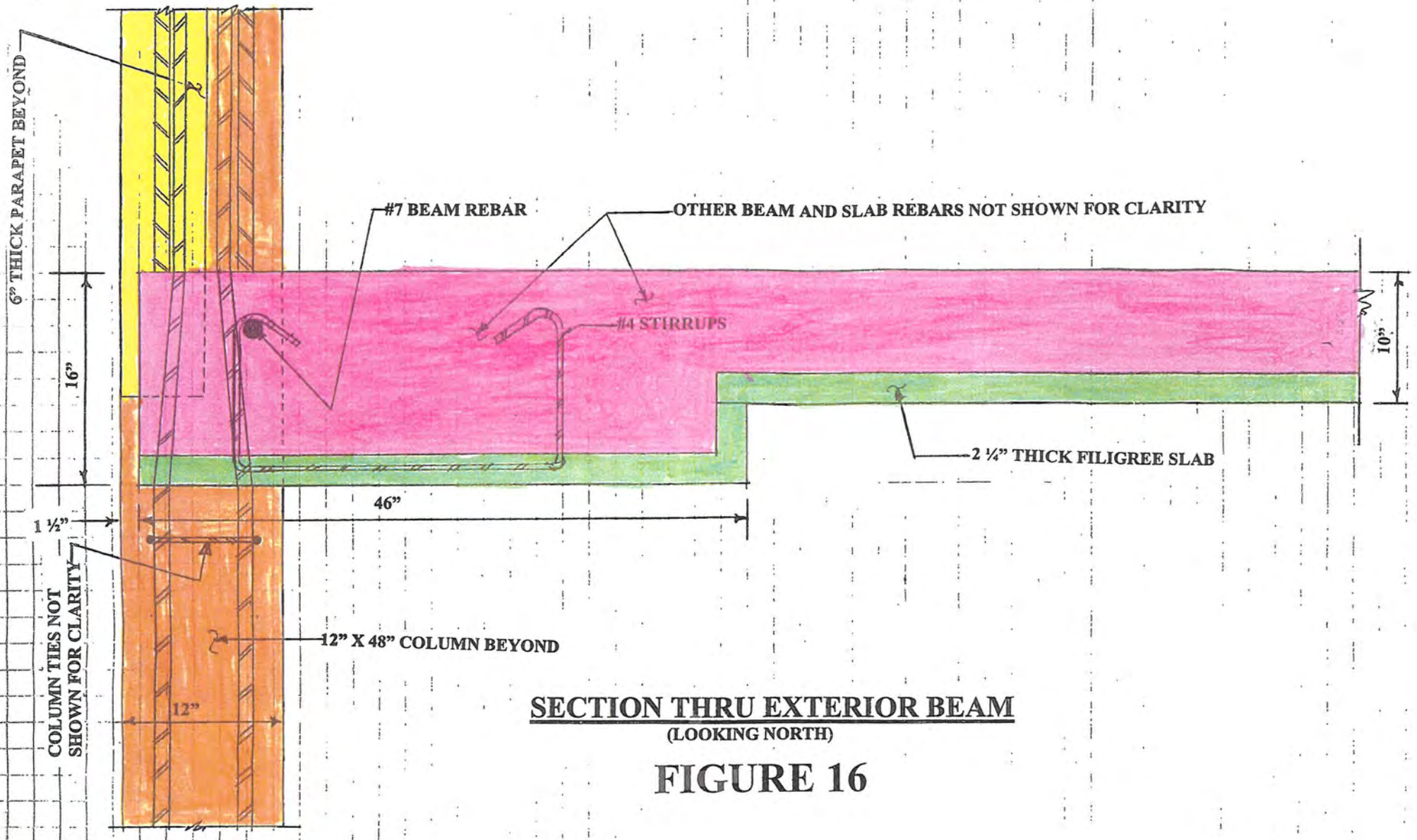
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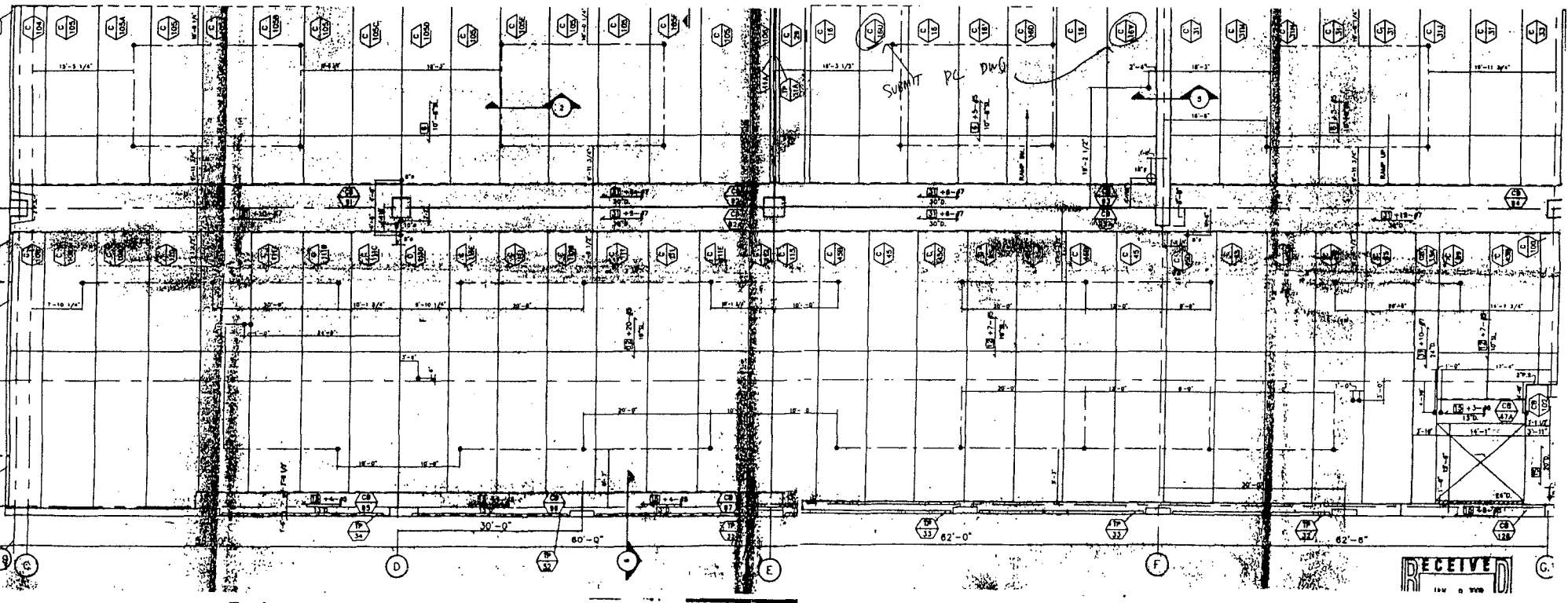
FIGURE 15

10

TYPICAL IMRF BEAM DETAIL

SCALE:  $\frac{1}{2}" = 1'-0"$





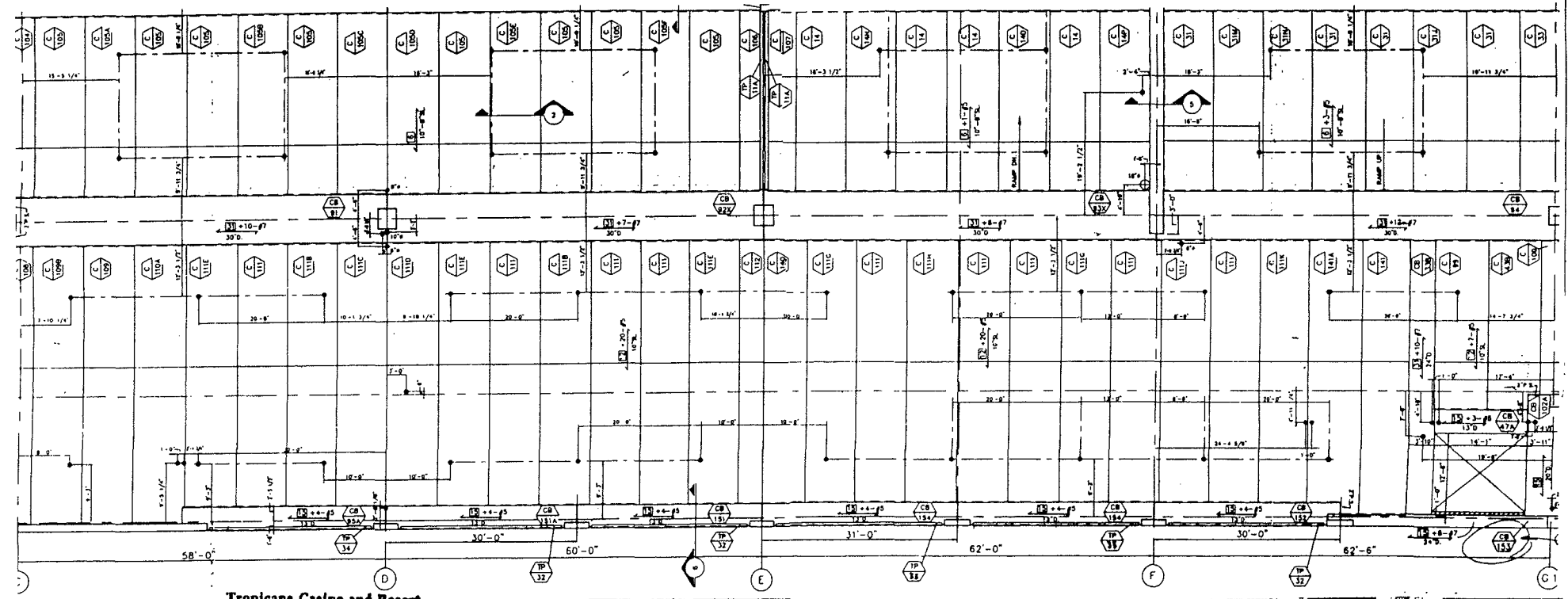
**Tropicana Casino and Resort  
Expansion Project**  
Reviewed Subject to Contract Requirements  
Submittal No.: 034102-SD-100  
Revision No.:  
Date: 03-14-03  
Initials: HL  
Kesting Building Corporation

**NOTES:**

1. INDICATES No. OF PRESTRESSED CABLES PER 8'-0" WIDTH OF PANEL.
2. ALL SLABS ARE 8" THK. U.N.O.
3. ALL BEAMS ARE 28" THK. U.N.O.
4. = INDICATES FIELD POURED CONCRETE. SEE STRUCTURAL DWG'S. FOR BEAM SIZES AND DEPTHS.
5. ALL FILIGREE SLABS TO CLOSE 1" ON FIELD POURED WALLS U.N.O.
6. ALL FILIGREE BEAMS TO CLOSE 3/4" ON COLUMNS.
7. E.O.F. = EDGE OF FILIGREE
8. 2" P.S. = 2" FIELD POURED CONCRETE AT END OF BEAM OR SLAB TO PROTECT CABLES
9. = INDICATES ELECTRICAL BOX LOCATION.

**PART PLAN  
FILIGREE WIDE SLAB AT LEVEL P3**

**FIGURE 17**



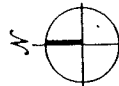
**Tropicana Casino and Resort  
Expansion Project**

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Submittal No.: 03-110-SD-100

Revision No.: 03-114-Q3

Date: 11/11/03

Initials: JK  
Keating Building Corporation



**NOTES:**

1. INDICATES No. OF PRESTRESSED CABLES PER 8'-0" WIDTH OF PANEL.
2. ALL SLABS ARE 8" THK. U.N.O.
3. ALL BEAMS ARE 28" THK. U.N.O.
4. = INDICATES FIELD POURED CONCRETE. SEE STRUCTURAL DWG'S. FOR BEAM SIZES AND DEPTHS.
5. ALL FILIGREE SLABS TO CLOSE 1" ON FIELD POURED WALLS U.N.O.
6. ALL FILIGREE BEAMS TO CLOSE 3/4" ON COLUMNS.
7. E.O.F. = EDGE OF FILIGREE
8. 2" P.S. = 2" FIELD POURED CONCRETE AT END OF BEAM OR SLAB TO PROTECT CABLES

**PART PLAN  
FILIGREE WIDE SLAB AT LEVEL P4**

**FIGURE 18**

**Tropicana Casino and Resort  
Expansion Project**

Reviewed Subject to Contract Requirements

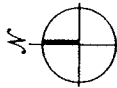
Submittal No.: 03-110-50-100

Revision No.: 03-14-03

Date: 03-14-03

Initials: [Signature]

Keating Building Corporation

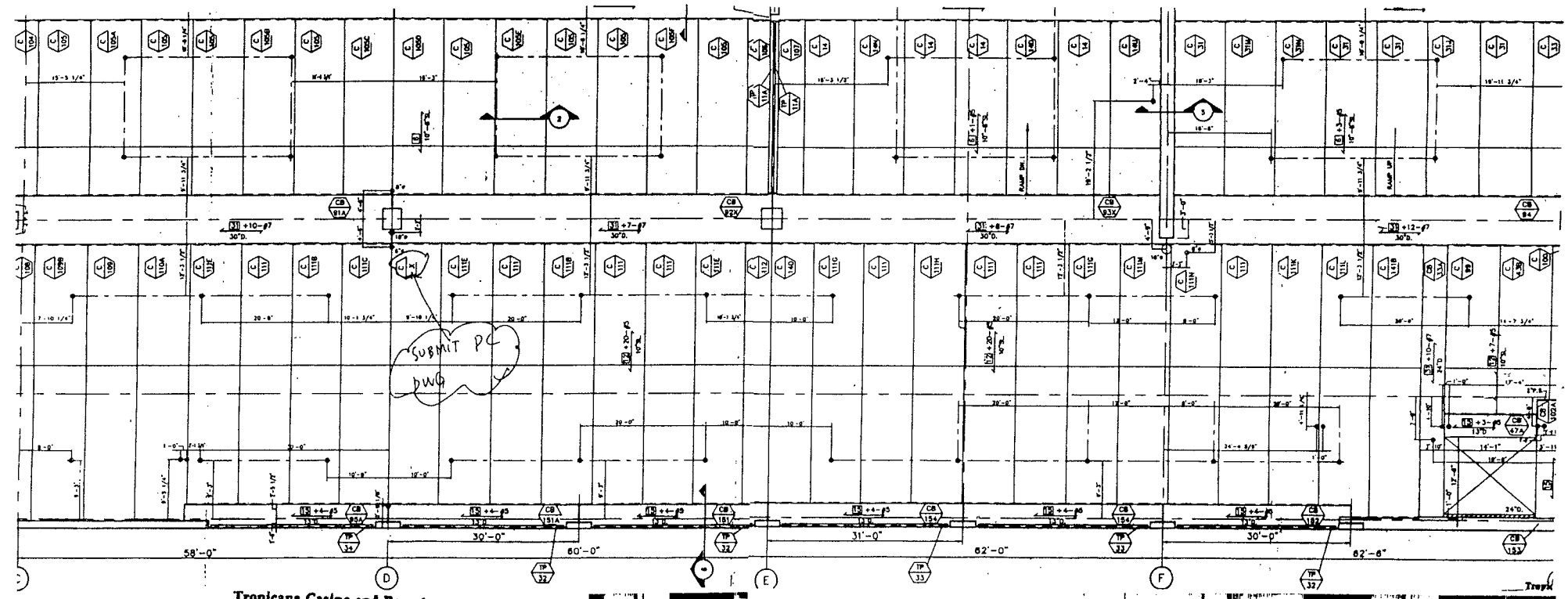


**NOTES:**

1. INDICATES No. OF PRESTRESSED CABLES PER 8'-0" WIDTH OF PANEL.
2. ALL SLABS ARE 8" THK. U.N.O.
3. ALL BEAMS ARE 28" THK. U.N.O.
4. = INDICATES FIELD POURED CONCRETE. SEE STRUCTURAL DWG'S. FOR BEAM SIZES AND DEPTHS.
5. ALL FILIGREE SLABS TO CLOSE 1" ON FIELD POURED WALLS U.N.O.
6. ALL FILIGREE BEAMS TO CLOSE 3/4" ON COLUMNS.
7. E.O.F. = EDGE OF FILIGREE
8. 2" P.S. = 2" FIELD POURED CONCRETE AT END OF BEAM OR SLAB TO PROTECT CABLES
9. = INDICATES ELECTRICAL BOX LOCATION

**PART PLAN  
FILIGREE WIDE SLAB AT LEVEL P5**

**FIGURE 19**



**Tropicana Casino and Resort  
Expansion Project**




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Revision No.: 03-14-03

Date: 03-14-03

Initials: [Signature]  
Keating Building Corporation

**NOTES:**

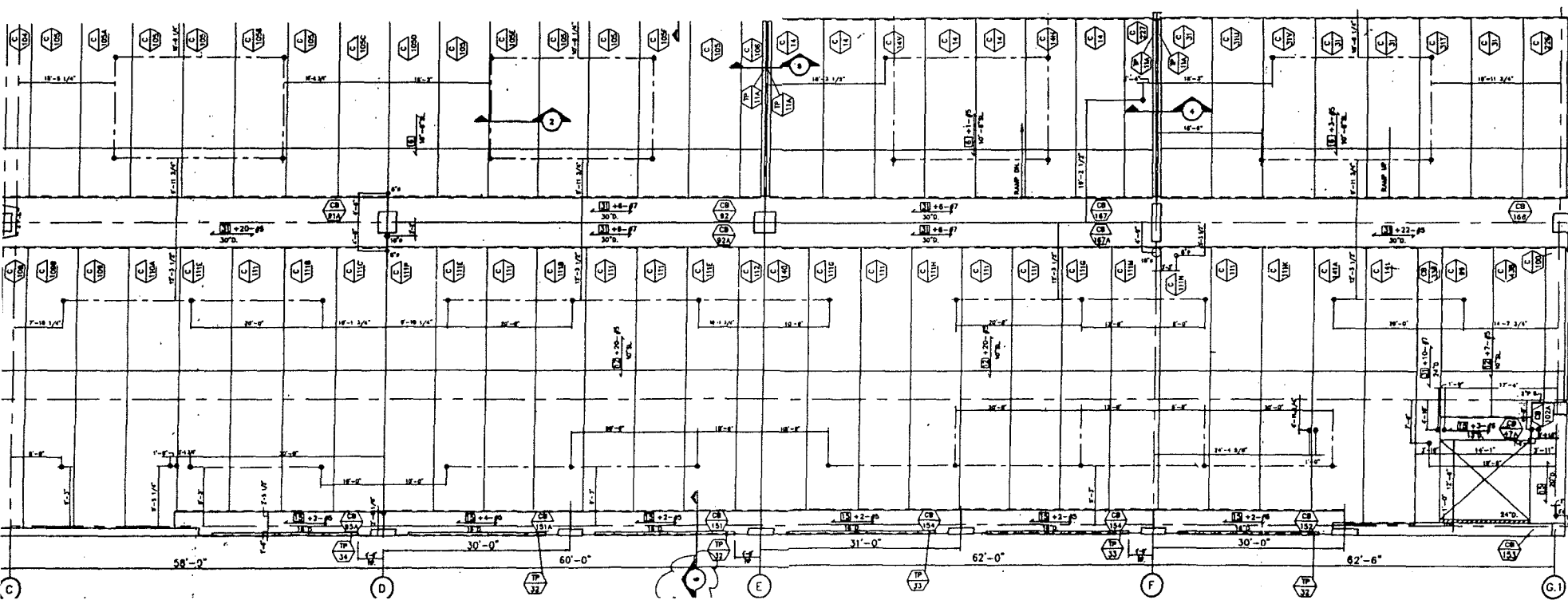
1.  INDICATES NO. OF PRESTRESSED CABLES PER 8'-0" WIDTH OF PANEL.
2. ALL SLABS ARE 8" THK. U.N.O.
3. ALL BEAMS ARE 28" THK. U.N.O.
4.  = INDICATES FIELD POURED CONCRETE. SEE STRUCTURAL DWG'S. FOR BEAM SIZES AND DEPTHS.
5. ALL FILIGREE SLABS TO CLOSE 1" ON FIELD POURED WALLS U.N.O.
6. ALL FILIGREE BEAMS TO CLOSE 3/4" ON COLUMNS.
7. E.O.F. = EDGE OF FILIGREE
8. 2" P.S. = 2" FIELD POURED CONCRETE AT END OF BEAM OR SLAB TO PROTECT CABLES
9.  = INDICATES ELECTRICAL BOX LOCATION.

**PART PLAN  
FILIGREE WIDE SLAB AT LEVEL P6**

**FIGURE 20**







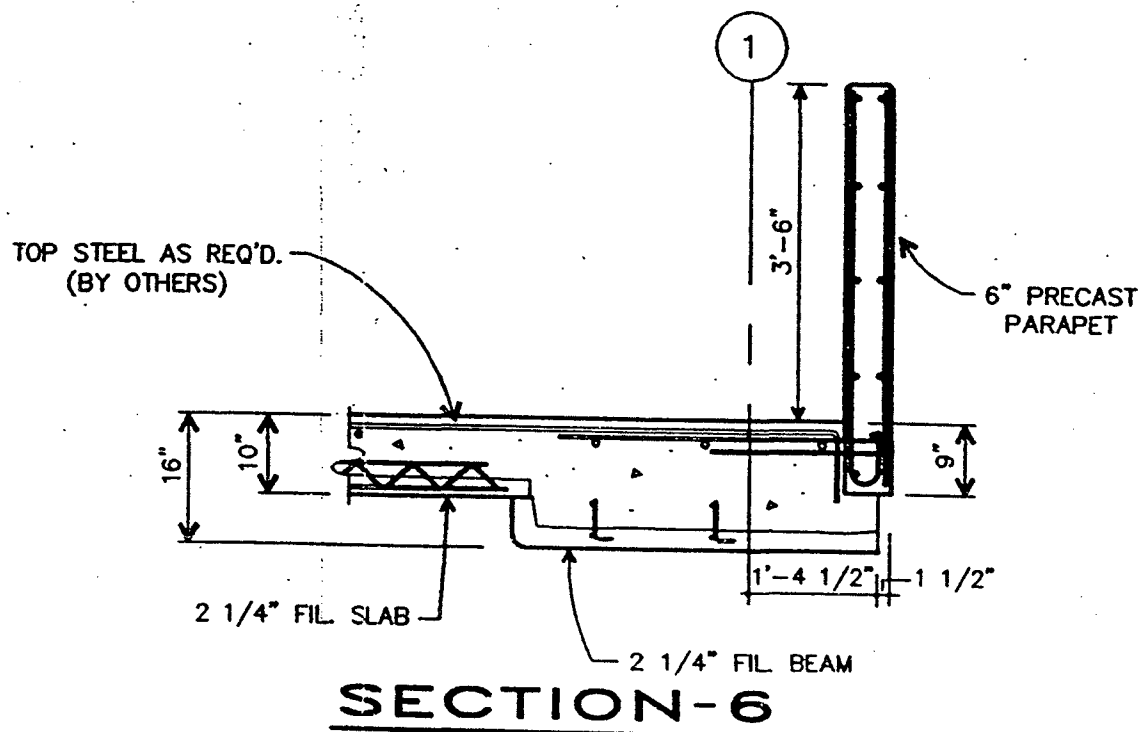
**Tropicana Casino and Resort  
Expansion Project**  
Reviewed Subject to Contract Requirements  
Submittal No.: 03410-50-100  
Revision No.:  
Date: 03-14-03  
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Keating Building Corporation

**NOTES:**

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2. ALL SLABS ARE 8" THK. U.N.O.
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4. = INDICATES FIELD POURED CONCRETE. SEE STRUCTURAL DWG'S. FOR BEAM SIZES AND DEPTHS.
5. ALL FILIGREE SLABS TO CLOSE 1" ON FIELD POURED WALLS U.N.O.
6. ALL FILIGREE BEAMS TO CLOSE 3/4" ON COLUMNS.
7. E.O.F. = EDGE OF FILIGREE
8. 2" P.S. = 2" FIELD POURED CONCRETE AT END OF BEAM OR SLAB TO PROTECT CABLES
9. = INDICATES...

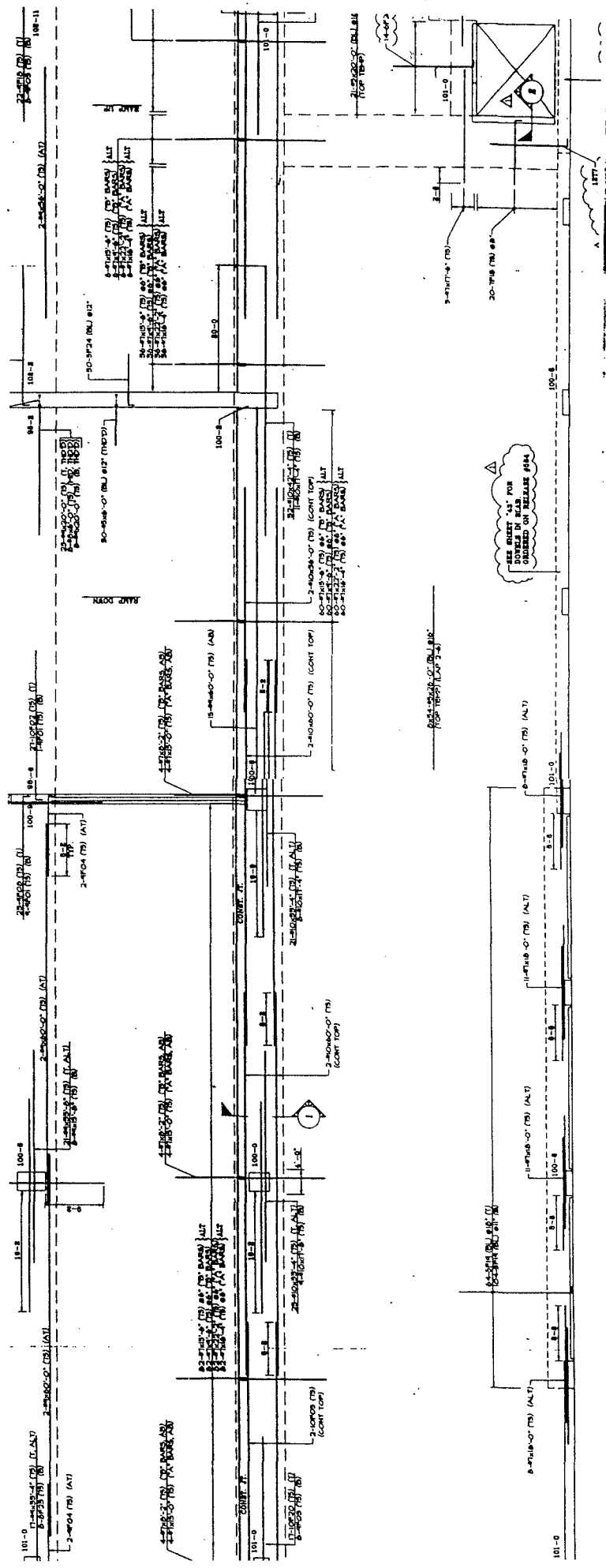
**PART PLAN  
FILIGREE WIDE SLAB AT LEVEL P8**

**FIGURE 22**



EXTERIOR BEAM SECTION AT LINE 1, LEVEL P4-P8

**FIGURE 23**



# TOP N-S REINFORCING STEEL AT LEVEL P3

(REF.: SHOP DRAWING)

## FIGURE 24

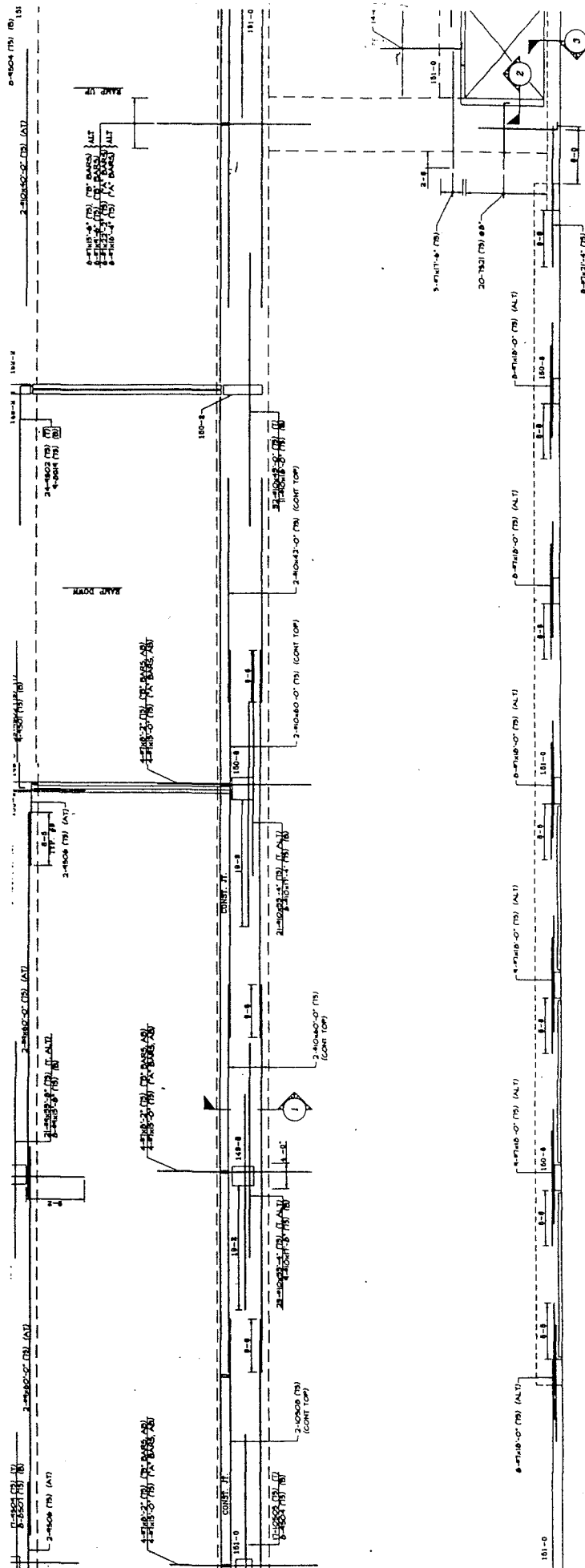
## FIGURE 25



FIGURE 27







# TOP N-S REINFORCING STEEL AT LEVEL P8

(REF.: SHOP DRAWING)

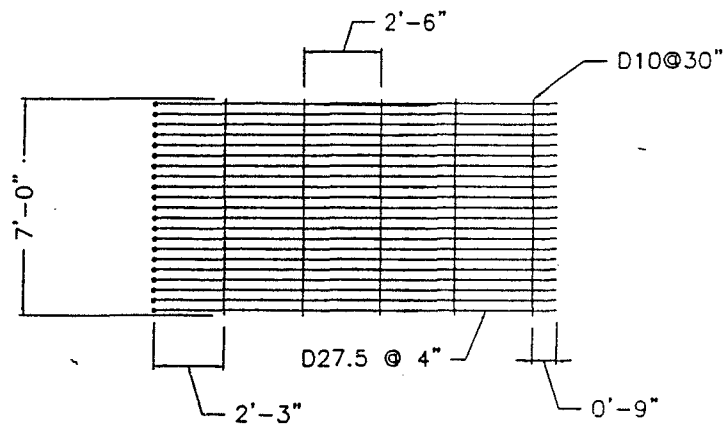
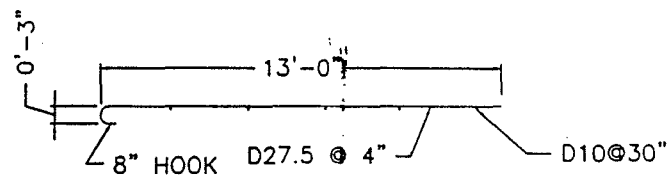
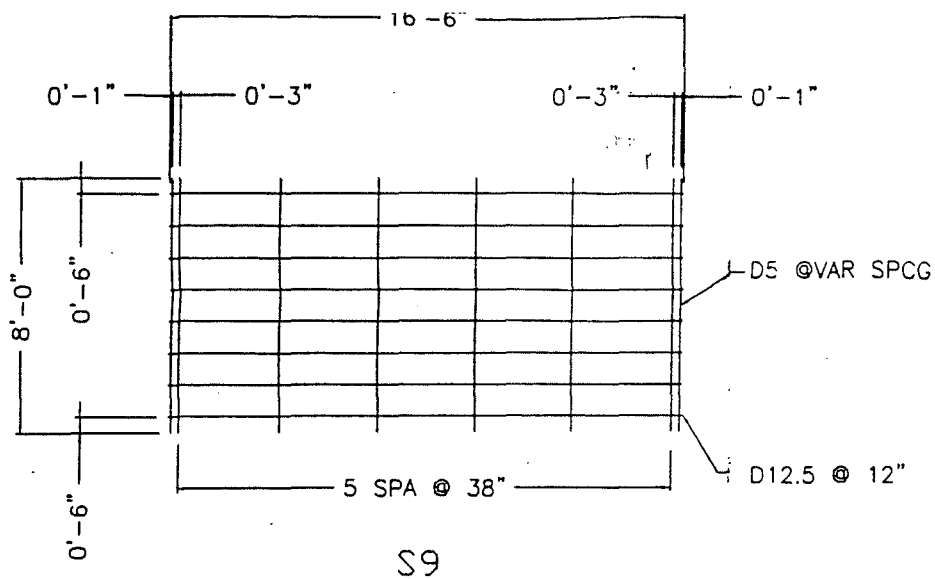
FIGURE 29

[illegible]

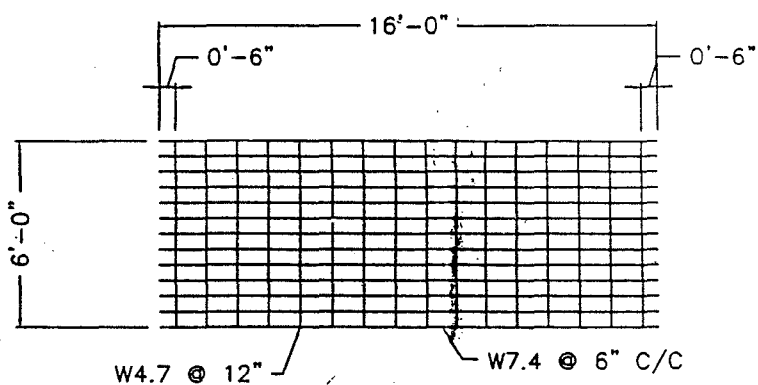
## TOP E-W REINFORCING STEEL AT LEVELS P5-P9

(REF.: SHOP DRAWING SHEET P5-P9)

**FIGURE 30**



S21

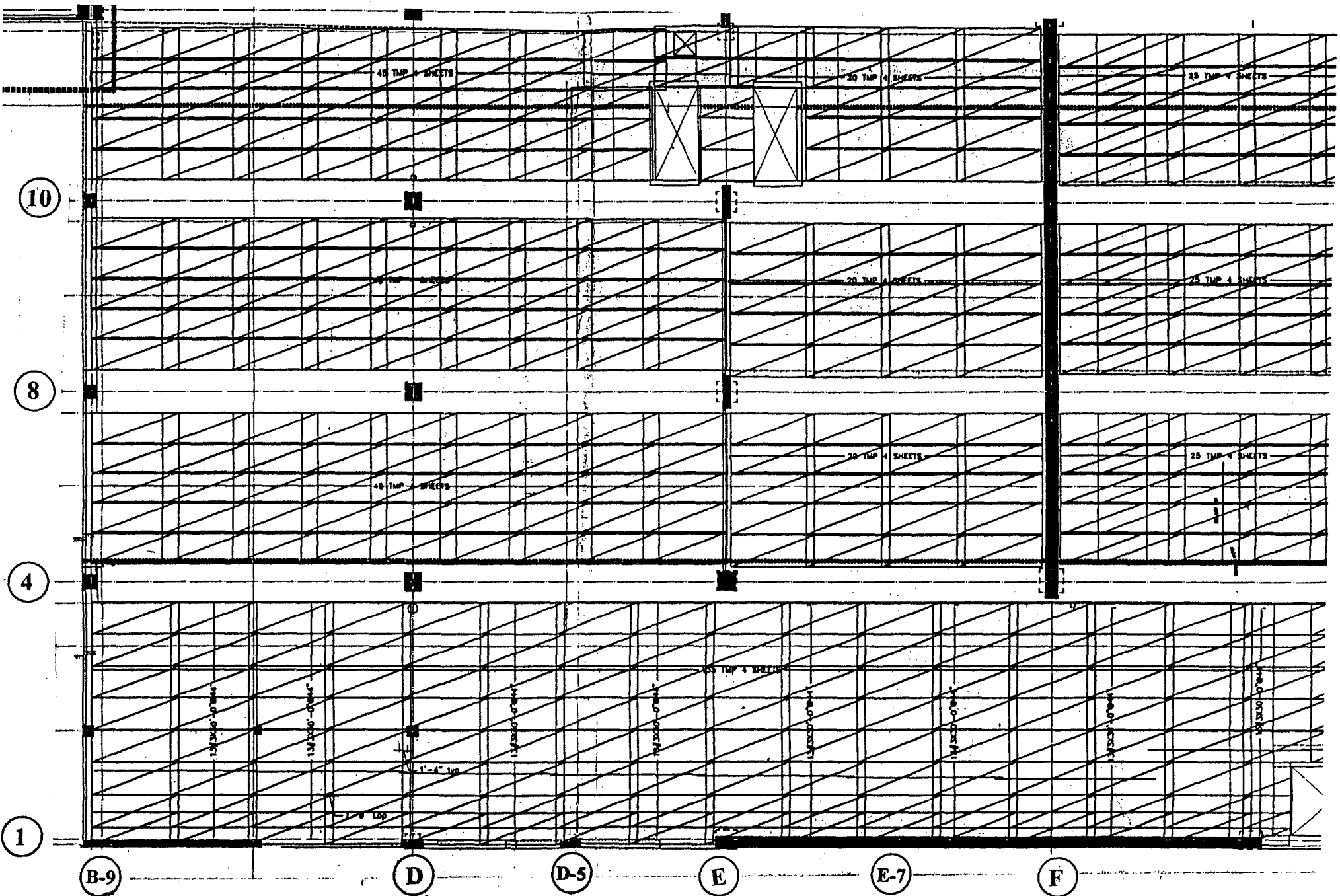


TMP4

## DETAIL OF S9, S21 AND TMP4 REINFORCEMENT

(REF.: SHOP DRAWING)

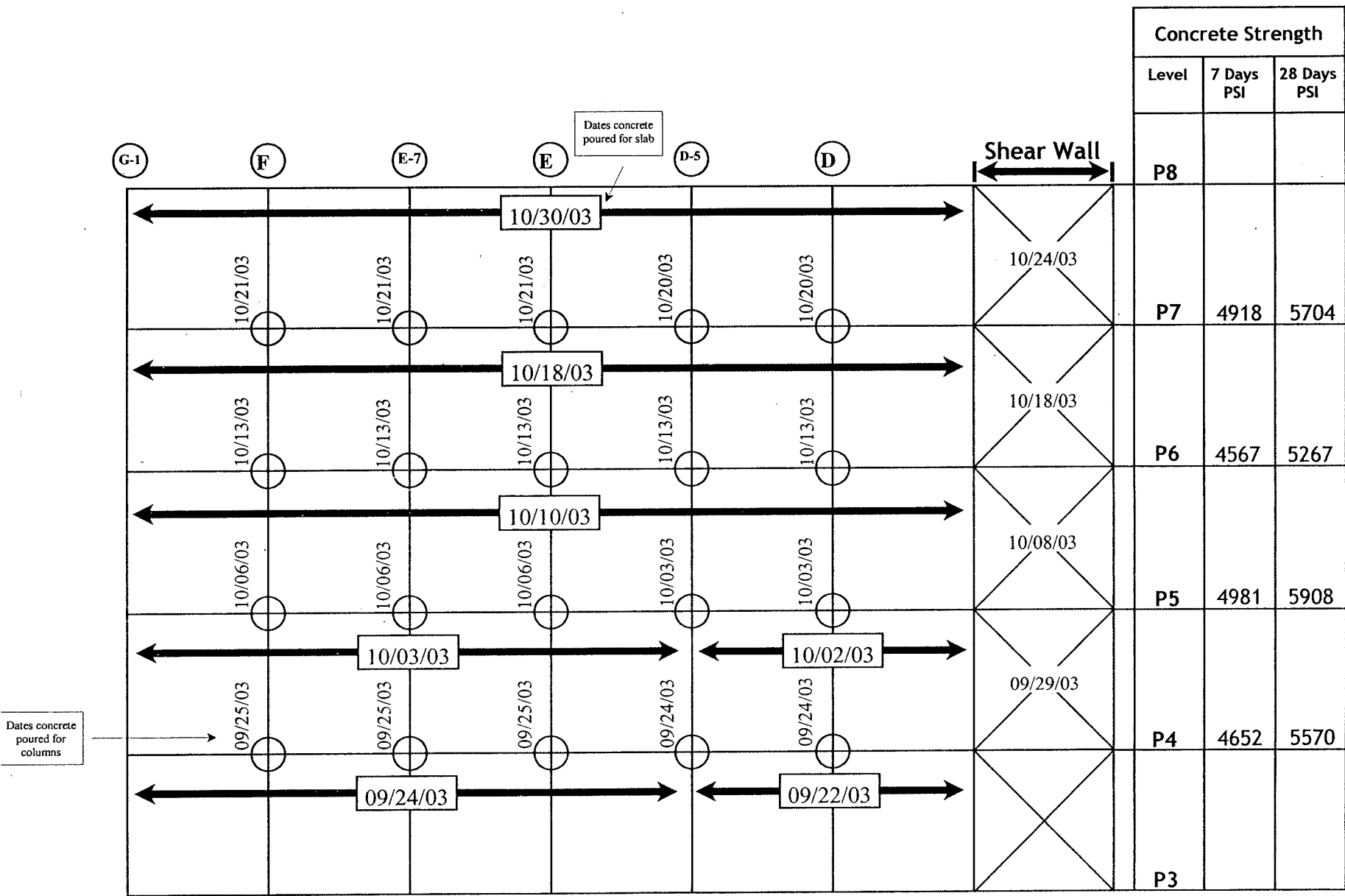
FIGURE 31



**TOP TEMPERATURE STEEL AT LEVELS P5 THROUGH P9**

(REF.: SHOP DRAWING SHEET P5-P9T)

**FIGURE 32**



Dates of Concrete Placement at Parking Levels and Shear Walls

**PARTIAL ELEVATION AT COLUMN LINE 1**

(Looking West)

(Not to scale)

**FIGURE 33**

# FIGURE 34

## Tropicana Expansion, Atlantic City, NJ

### Summary of Concrete Inspection Reports

Level	Day Bk Time (Days)	Average Component Strength					
		Slab		Columns		Shearwall	
		PSI	% Design	PSI	% Design	PSI	% Design
P3	3						
	7			6125 (based on 3 breaks)	77		
	28						
P4	3	4507 (based on 3 breaks)	75				
	7	4652 (based on 6 breaks)	78	5266 (based on 5 breaks)	66	6445 (based on 1 break)	81
	28	5570 (based on 11 breaks)	92	6468 (based on 10 breaks)	81	7618 (based on 2 breaks)	95
P5	3						
	7	4981 (based on 5 breaks)	83	5743 (based on 5 breaks)	72	4655 (based on 1 break)	67
	28	5908 (based on 10 breaks)	98	7013 (based on 10 breaks)	88		
P6	3						
	7	4567 (based on 5 breaks)	76	4930 (based on 5 breaks)	70	5170 (based on 1 break)	74
	28	5267 (based on 5 breaks)	88	6045 (based on 2 breaks)	87	6445 (based on 2 breaks)	92
P7	3						
	7	4918 (based on 5 breaks)	82	5156 (based on 5 breaks)	68		
	28	5704.5 (based on 8 breaks)	95	6582 (based on 4 breaks)	87		

\*Please note that level P2 was poured during the month of June 2003.

# FIGURE 35

## Tropicana Expansion, Atlantic City, NJ

### Column & Shearwall Concrete Inspection Reports

Level	Column Line	Placement Date	Ambient Temp (°F)	Concrete Temp (°F)	Design Strength (psi)	7Day Bk (psi)	% of Design	28 Day Bk (psi)	% of Design	SLUMP (inches)	YDs3
P2-P3	Shear Wall	POURED DURING 6/03									
	D-1										
	D.5-1										
	E-1										
	E.5-1										
	F-1										
P3-P4	Shear Wall										
	D-1										
	D.5-1										
	E-1	09/09/03	65-74	75	8000	6125	77			6.75	40
	E.5-1	09/09/03	65-74	75	8000	6125	77			6.8	40
	F-1	09/09/03	65-74	75	8000	6125	77			6.8	40
P4-P5	Shear Wall	09/29/03	59-68	68	8000	6445	81	7617.5	95	7.5	303
	D-1	09/24/03	65-78	73	8000	5410	68	6682.5	84	7	282
	D.5-1	09/24/03	65-78	73	8000	5410	68	6682.5	84	7	282
	E-1	09/25/03	70-75	73	8000	5170	65	6325	79.5	7	279
	E.5-1	09/25/03	70-75	73	8000	5170	65	6325	79.5	7	279
	F-1	09/25/03	70-75	73	8000	5170	65	6325	79.5	7	279
P5-P6	Shear Wall	10/08/03	68-71	66	8000	4655	67			7.8	363
	D-1	10/03/03	43-65	68	8000	5530	69	6762.5	85	6.75	260.5
	D.5-1	10/03/03	43-65	68	8000	5530	69	6762.5	85	6.75	260.5
	E-1	10/06/03	51-68	65	8000	5885	74	7180	89.5	7	236
	E.5-1	10/06/03	51-68	65	8000	5885	74	7180	89.5	7	236
	F-1	10/06/03	51-68	65	8000	5885	74	7180	89.5	7	236
P6-P7	Shear Wall	10/18/03	54-61	60	7000	5170	74	6445	92	8.0	448
	D-1	10/13/03	66-71	73	7000	4930	70	6045	86.5	7.25	14
	D.5-1	10/13/03	66-71	73	7000	4930	70	6045	86.5	7.25	14
	E-1	10/13/03	66-71	73	7000	4930	70	6045	86.5	7.25	14
	E.5-1	10/13/03	66-71	73	7000	4930	70	6045	86.5	7.25	14
	F-1	10/13/03	66-71	73	7000	4930	70	6045	86.5	7.25	14
P7-P8	Shear Wall										
	D-1	10/20/03	60	65	7000	4775	68	6285	90	8	159
	D.5-1	10/20/03	60	65	7000	4775	68	6285	90	8	159
	E-1	10/21/03	62-70	68	8000	5410	68	6780	85	7.5	249
	E.5-1	10/21/03	62-70	68	8000	5410	68	6780	85	7.5	249
	F-1	10/21/03	62-70	68	8000	5410	68	6780	85	7.5	249

# FIGURE 36

## Tropicana Expansion, Atlantic City, NJ

### Concrete Inspection Reports for Slab

Level	Column Line	Placement Date	Ambient Temp (°F)	Concrete Temp (°F)	Design Strength (psi)	7 Day		28 Day		SLUMP (inches)	YDs3
						PSI	% of Design	PSI	% of Design		
P4	C-E	09/22/03	67-72	77	6000	4630	77	5685	94.5	8.0	176
P4	C-E	09/22/03	67-72	74	6000	4685	78	5745	96	7.5	176
P4	D.7-G.1	09/24/03	65-78	75	6000	4915	82	5737.5	95.5	7.5	282
P4	D.7-G.1	09/24/03	65-78	71	6000	4595	77	5462.5	91	8.5	282
P4	D.7-G.1	09/24/03	65-78	71	6000	4595	77	5462.5	91	8.5	282
P4	D.7-G.1	09/24/03	65-78	67	6000	4490	75	5090	85	8.5	282
P5	C-D.7	10/02/03	55-60	62	6000	5355	89	6382.5	106.5	7.0	282
P5	C-D.7	10/02/03	55-60	60	6000	5055	84	6097.5	101.5	7.75	282
P5	D.7-G.1	10/03/03	43-65	61	6000	4880	81	5800	96.5	7.5	260.5
P5	D.7-G.1	10/03/03	43-65	59	6000	4720	79	5630	93.5	7.0	260.5
P5	D.7-G.1	10/03/03	43-65	60	6000	4895	82	5630	93.5	8.0	260.5
P6	C-G.1	10/10/03	74-68	68	6000	4950	82	5515	92	7.5	489
P6	C-G.1	10/10/03	74-68	68	6000	4505	75	5230	87	7.5	489
P6	C-G.1	10/10/03	74-68	78	6000	4420	74	4930	82	7.25	489
P6	C-G.1	10/10/03	74-68	76	6000	4365	73	5250	88	7.5	489
P6	C-G.1	10/10/03	74-68	68	6000	4595	77	5410	90	8.25	489
P7	C-G.1	10/18/03	54-61	65	6000	5305	88	6312.5	105	7.5	448
P7	C-G.1	10/18/03	54-61	65	6000	4545	76	5375	90	7.785	448
P7	C-G.1	10/18/03	54-61	51	6000	5375	90	6080	101.5	7.0	448
P7	C-G.1	10/18/03	54-61	62	6000	4735	79	5630	93.5	7.5	448
P7	C-G.1	10/18/03	54-61	62	6000	4630	77	5125	85	7.5	448

\*Please note that level P2 was poured during the month of June 2003.