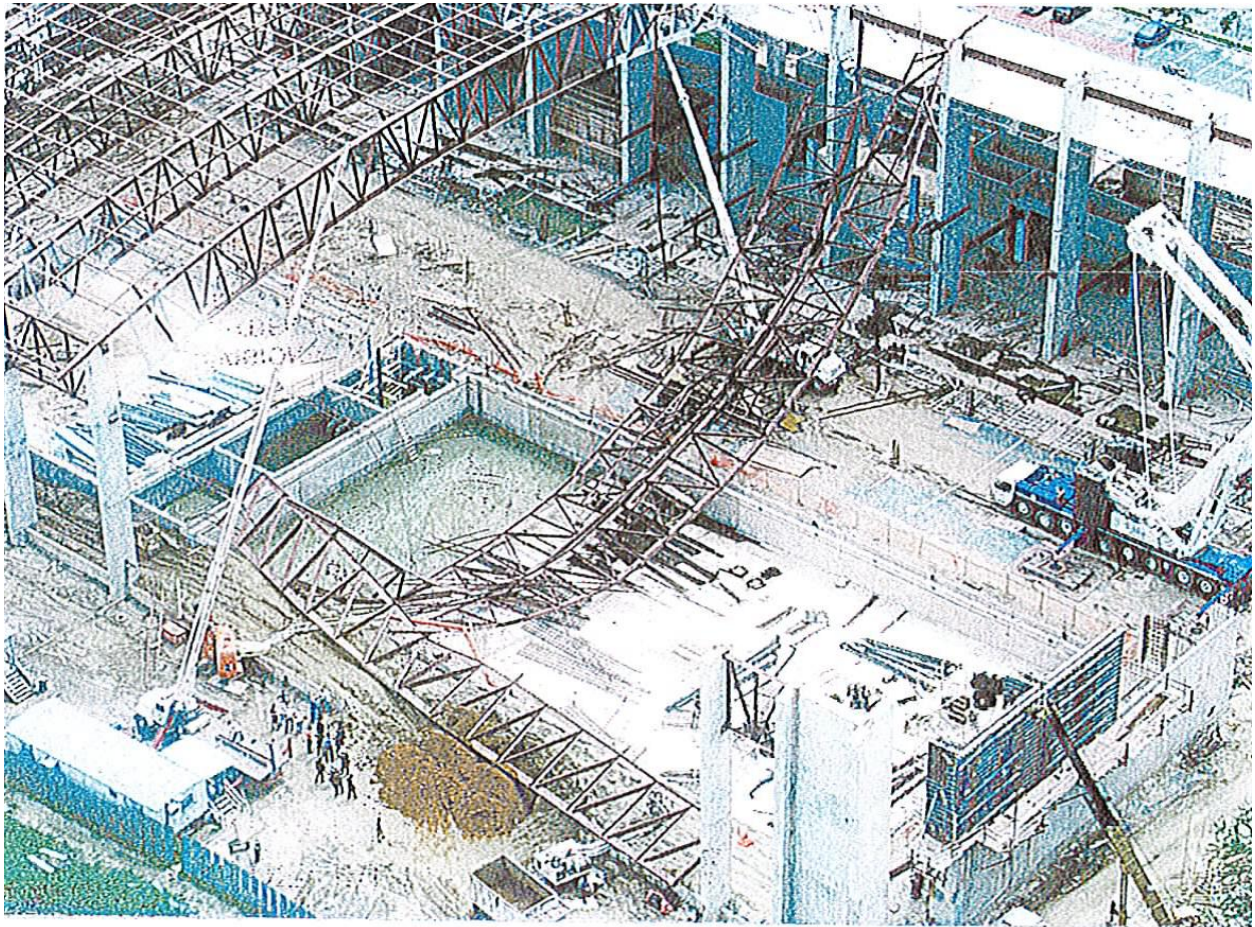


Investigation of the August 3, 2005, Collapse of roof trusses at Natatorio de, San Juan, PR

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

February 2006



REPORT

Investigation of the August 3, 2005, Collapse of roof trusses at Natatorio de, San Juan, PR

Report prepared by
Mohammad Ayub, PE

Natatorion de

Background:

An incident occurred on August 3, 2005 at about 11:30 AM at the construction site of “Natatorio de San Juan” (swimming and diving pool) at San Juan central park, San Juan, PR. The project was owned by the San Juan Municipality. The incident involved the collapse of three long span steel roof trusses and several steel bar joists that fell some fifty feet to the ground, killing two workers and injuring two others. San Juan PR OSHA Area Office’s compliance officer arrived at the scene of the incident within an hour and took photographs, and discussed the structural integrity of the remaining structure with the construction personnel. Local police and fire departments immediately responded and engaged in rescue and retrieval of bodies. Bodies were retrieved that same afternoon and the injured were sent to hospitals. The Compliance and Safety Health Officer (CSHO) obtained necessary construction documents to initiate the investigation of the incident to determine whether any OSHA or nationally recognized standards were violated.

The Puerto Rico OSHA, thru Region II, requested the Directorate of Construction to provide engineering assistance for causal determination and to identify potential OSHA standard violations. A structural engineer from the Office of Engineering, Directorate of Construction, visited the incident site on November 8, 2005. At the time of his visit, all failed structural members were already removed from the site as per the demolition plan. Construction in the failed area was not renewed.

The following is the basic information of the project:

- | | |
|--|--|
| 1. Owner: | San Juan Municipality, PR |
| 2. Name of Project: | Natatorio de San Juan |
| 3. Site Address: | San Juan Central Park, PR |
| 4. Consulting Engineer: | Orlando E. Sanchez Rivoleda & Associates, Hato Rey, PR |
| 5. Architect: | Jorge Oliver Architects, AIA, Isabella, PR |
| 6. Structural Consultants: | Jose A. Quinones, Jr. Associates, San Juan, PR (SER) |
| 7. General Contractor and Concrete contractor: | F & R Contractors Corporation, PR (FR) |
| 8. Steel Fabricator and Erector: | Santiago Metal Manufacturing Corp., Carolina, PR (SM) |
| 9. Electrical and Mechanical contractor: | Belmac |
| 10. Re-bar installation contractor: | Max Steel Corporation |
| 11. Masonry and plastering contractor: | Luis Vazque |
| 12. Excavation and earth work contractor: | JJJ Heavy Equipment |
| 13. Independent inspection agency retained by the owner: | May Engineers, PSC |
| 14. Welding inspectors retained by Santiago Metals: | GMTS |
| 15. Structural consultant retained by Santiago Metals: | Arturo Beale, PE |
| 16. Consultant retained by Santiago Metals: | Dr. Jose Guevara, PhD |

Project:

The project consisted of construction of a large, one-story concrete and steel structure for an indoor swimming and diving pool with bleachers. The roof consisted of steel trusses spanning in an east-west direction, truss girders spanning in a north-south direction, rolled shapes, bar joists

spanning in a north-south direction, and steel deck (figures 1 to 8). The area of interest for the investigation was limited to the area where the failure occurred, i.e., from column lines B to K and 4 to 10. Roof trusses marked “A” (approximate weight 80,000 pounds) spanned approximately 190 feet between column lines B & K at 31’-0” on center, supported on concrete columns on B line and on truss girder on K line. The roof trusses and the truss girders comprised of WT shapes for top and bottom chord members, and angles for vertical and diagonal members. The truss girder (approximate weight 70,000 pounds) was 15 feet deep and spanned 153 feet. The truss girder bottom chord was approximately 40 feet above the ground. The roof trusses were curved and spanned approximately 190 feet. Over the top chords of the roof trusses, 28” deep joists were placed at intervals of 8 feet with a 22 gauge metal deck over them. Plans called for three rows of bridging for the joists.

Construction:

Construction of the project began approximately two years prior to the incident. Steel erection started early in 2005. SM erected the low roof on the west side first because FR had already completed the concrete framing on the west side. SM installed the roof trusses and bar joists on the low roof without any reported incident. SM then proceeded to erect the high roof. SM erected the truss girders on column line K beginning from the south to north, i.e., from column line 20 to column line 1. In this row, there were two long girder trusses, one was marked T-93 spanning 93 feet between column line 17 & 14, and the other was marked T-153 spanning 153 feet between column lines 10 & 4. After all truss girders were erected on column line K, SM proceeded to erect roof trusses “A” between column lines B & K. A crane with a capacity of 650 tons was used.

The first roof truss was installed on column line 20. The crane did not release the load of the first truss until another crane erected the adjoining truss on column line 17, and all bracings between the two trusses were completed. SM then proceeded to erect other trusses moving towards the north on column lines 16, 15, 14, 10, 9 and 7. The truss on column line 7 was the last truss to be erected about a week before the incident, see figure 9. The following are the dates of erection of truss girders and roof trusses:

Girder trusses:	T-91	June 17, 2005
	T-153	June 17, 2005
Roof trusses:	Column line 20	June 27, 2005
	17	June 28
	16	June 29
	15	June 30
	14	Unknown
	10	July 11
	9	One week before the incident
	7	One week before the incident

When SM erected the truss girder T-153 on column line K, they installed two guy wire ropes intended to brace the top chord of the truss girder approximately at one-third points. The wire

rope was wrapped around the top chord of the truss and sloped approximately 45 degrees to the ground and was fastened to concrete blocks on both sides. There were four guy wires, two on the east side and two on the west side. SM reported that the location and size of bracings were determined by Mr. Arturo Beale, PE. SM further said that Mr. Beale advised against bracing the other long truss girder T-91. In interview with OSHA, Mr. Beale, however, denied having advised SM. SM claimed that the four guy wires were tensioned by a ratchet near the top chord of the girder truss and the truss was aligned vertically. SM later admitted, however, that they did not monitor the vertical alignment of the truss except on the very first day. SM's foreman in interview with OSHA, however, said that there were no turnbuckles to tighten the wire ropes.

After the roof trusses on column lines 10, 9 and 7 were erected, SM removed the two guy wires on the south side approximately one week before the incident, thus leaving the girder truss with only one set of guy wires on the north side. A few minutes before the incident, an SM foreman noticed that the truss girder T-153 was bowing out (deflecting laterally) towards the east and wanted to tension the wire rope but the failure occurred before he could correct the truss alignment. In fact, a construction aerial photo taken by FR (figure 9) showed that the bow of the girder truss existed since at least June 30, 2005 when the photograph was taken.

Progress of construction up to the time of the incident:

At the time of the incident, roof trusses for the high roof were erected up to the column line 7, beginning from south towards north. The low roof was practically completed. All girder trusses were erected on column line K. Bar joists were placed between all roof trusses except for a length of about 30 feet on the east side between column lines 10 & 9; and between column line 9 & 7. Roof deck was placed on approximately 50% of the bar joists.

The incident:

On August 3, 2005, workers went out for lunch at about 10:30 AM. On return from lunch at 11:30 AM, two SM employees prepared to go up in a boom basket to resume placing bar joists over the roof trusses between column lines 10 & 9, and between column lines 9 & 7. As the employees were halfway up to the roof, the truss girder on column line K and the roof trusses on column line 9 and 7 collapsed to the ground, injuring the two employees seriously. A third SM employee was on the ground and was fatally injured. An employee of the earthwork sub-contractor was also nearby and was fatally struck. The roof trusses and the girder truss each fell in one piece, though they were bent and crooked in shape. The truss girder slid away from both supports at the concrete columns on column lines 4 and 10, leaving the bottom and top supports at the concrete columns intact. The truss girder was wavy both in the longitudinal and transverse directions. The roof trusses also fell in a curved position but remained connected to the concrete columns on the west side and to the girder trusses on the east side. The roof joists were scattered over a large area. The guy wires did not break during the collapse, but the concrete block where the bottom ends of the guy wire was fastened moved longitudinally parallel to column line K. For the collapsed roof trusses, see figures 10 to 20.

Post-incident:

SM's structural consultant, Mr. Arturo, prepared a demolition plan to cut the failed steel pieces into sections to haul them away from the site. FR approved the plan and SM accomplished the task of removing all collapsed steel. Neither any survey was done to identify where the steel fell on the ground nor was any steel saved for later examination by interested parties for forensic evaluation.

SM retained Dr. Jose Guevara of the University of Puerto Rico to advise if any changes in the erection procedure should be made in the future. Dr. Guevara prepared a report. The following is the summary of his recommendations. Dr. Guevara did not opine on the cause of the collapse.

- The truss T-153 will deflect horizontally 1.0" and vertically 1.6" under its own weight during construction. This deflection can be reduced by providing four cables, two on each side at one third span length
- To reduce further lateral deflection, 8 cables shall be provided, four on each side at one fifth span length.
- Once the entire structure is complete, the horizontal deflection will be less than 0.125".
- Dr. Guevara used SAP 2000 for gravity load evaluation. He did not review for lateral seismic or wind loads.

Engineering Evaluation:

Bowing of the girder truss:

Bowing of the girder truss T 153 was evident since at least June 30, 2005. It should not have escaped the attention of the project personnel in the field as it was in the clear sight of everyone. Neither any corrective measures were taken by SM nor the structural engineer of record or SM's expert consulted to determine the cause of the bowing of the girder truss. If such actions were taken on a timely basis, the incident could have been avoided. Such bowing occurs when the compression flanges are not properly braced against lateral torsional buckling at regular intervals. The braces, to be effective, must be able to prevent lateral translation and rotation of the top chord.

Wire ropes used to brace the girder truss top flange:

There were two sets of ½" wire ropes used by SM intended to brace the top chord at approximately 1/3rd points, i.e., 51 feet on centers. SM initially claimed that the size and spacing of the wire ropes were recommended by their consultant, Mr. Arturo Beale, PE. Mr. Beale, however, denied having advised SM about either the size or spacing of the wire ropes. Mr. Beale said that his advice was limited to the proper lifting of the trusses, and did not include bracing the trusses.

SM stated that the wire ropes were wrapped around the top chord of the girder truss and then, sloping at approximately 45 degrees, they were each tied to a concrete dead man on either side of

the truss. Initially, SM claimed that there were turnbuckles on the wire ropes near the top. The foreman, Mr. Rivera, however, said that there were no turnbuckles to tension the wire ropes. Mr. Rivera further said that neither he nor anyone else ever retightened the wire ropes. Mr. Rivera attempted to tension the wire ropes on the day of the incident but he could not do so because the trusses collapsed before he could reach them.

It is evident that the wire ropes were not tightened to the required tension because this was done only by human strength. Wire ropes do not lend themselves to be tightened around an 18-inch deep structural section without proper equipment. Wire ropes in such conditions would neither prevent the top chord's lateral translation nor its rotation. The wire ropes were therefore ineffective to act as braces. The following reasons support rejection of the use of wire ropes as viable braces to prevent lateral torsional buckling:

1. The wire ropes were loosely wrapped around the top chord. SM had no mechanism available to tighten the wire rope around the top chord or around the concrete dead man.
2. There were no turnbuckles to tighten the ropes
3. There was no monitoring of the condition and sag of the wire ropes after they were erected.
4. The wire ropes are flexible members, and under the conditions described above, could not have offered any restraint against translation and rotation of the top chord of the girder truss.

OSHA analyzed T-153 for the loads imposed at the time of the erection, and prior to its collapse. The maximum axial compressive force in the top chord was estimated to be approximately 87 kips at the time of erection and 227 kips at the time of the collapse. The compressive force significantly increased due to the placement of two trusses marked A. The buckling strength of the top chord was computed to be 15k, 43k, 139k and 215k for unbraced lengths of 153 ft., 92 ft., 51 ft. and 41 ft. respectively, as per LRFD provisions with a phi factor of 1.0.

For the sake of discussion, if it is assumed that the wire ropes were in fact tightened to the desired level and indeed acted as effective braces, it appears that the 1/3rd location of the braces would have been adequate. On this assumption, the critical buckling load of the top chord would be 139 kips against an axial load of only 87 kips arising out of the girder truss dead load. However, if the wire ropes loosened and remained untightened, as was the case in this instance, the unbraced length of the top chord would suddenly increase to 153 feet with a buckling strength of only 15 kips. After the two trusses marked A were placed, the axial load of the top chord increased to 215 kips due to the dead loads of these two trusses. The unbraced length of the top chord would then be reduced to 92 feet because the trusses marked A would provide lateral support. The maximum buckling strength of the top chord would then be increased to 43 kips from 15 kips, but would remain well below the desired strength of 215 kips.

Structural design by the engineer of record:

OSHA reviewed the computations provided by the structural engineer of record and has discovered a number of deficiencies that must be addressed. These deficiencies did not contribute to the collapse.

Based upon the above, OSHA concludes that:**Construction deficiencies:**

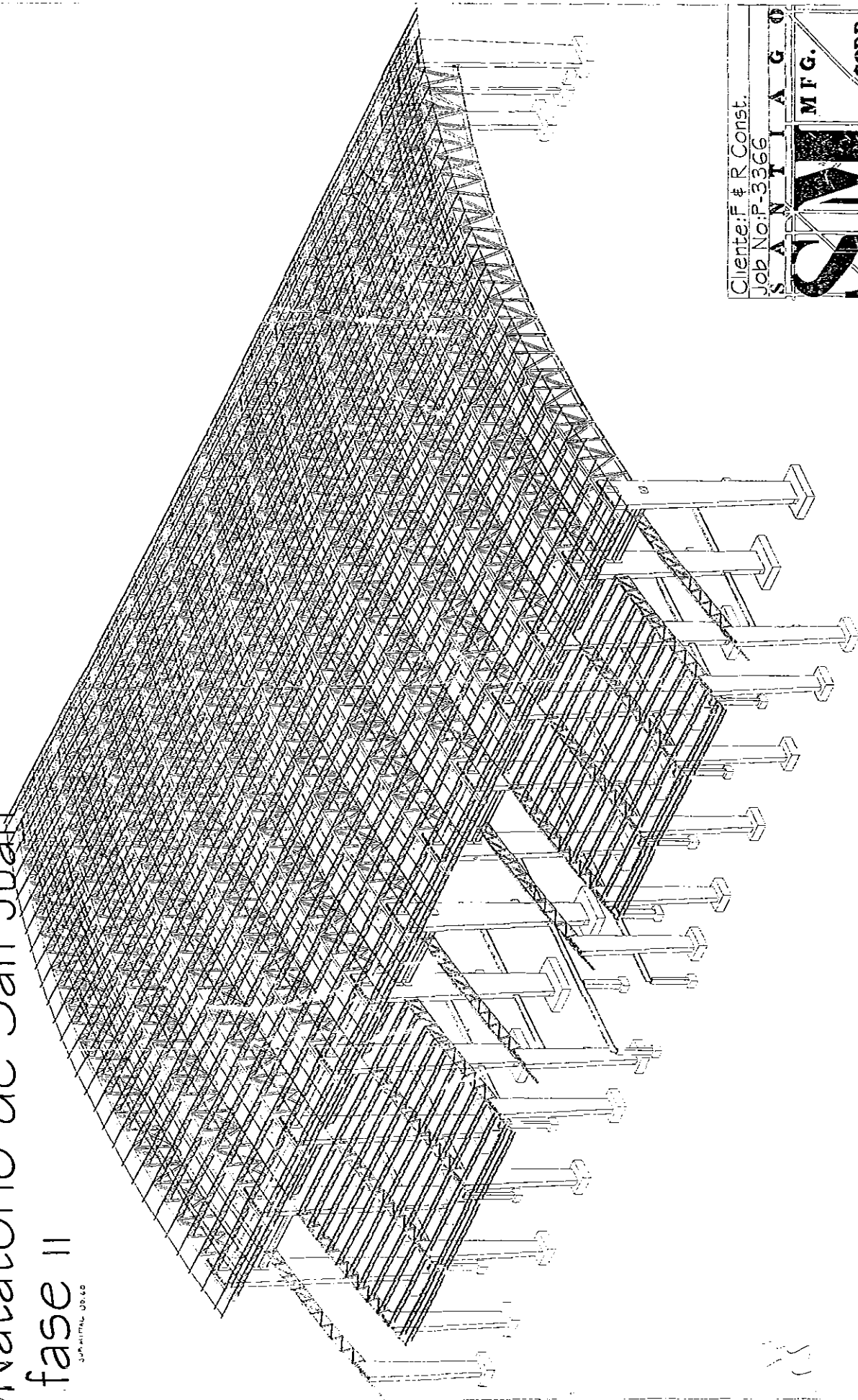
1. The steel erector did not follow the generally accepted standard practice to provide stability to the girder truss T-153 during erection against lateral-torsional buckling. There was no approved erection procedure for the roof trusses.
2. The steel erector failed to obtain engineering assistance when the truss laterally bowed out of alignment days before the incident. If engineering assistance was sought, this incident could have been avoided.
3. Wind was not a causal factor.

Design deficiencies:

1. San Juan, PR falls in seismic zone 3 as per Uniform Building Code 1997, the governing building code for the design of the building. SER did not perform seismic computations to determine the magnitude of seismic forces and the manner they would be resisted by the lateral load resisting system.
2. SER has called for a 22-gage roof deck to act as a flexible roof diaphragm to transmit lateral wind and seismic forces to the shear wall/moment frames. The capacity of the roof deck to perform as a viable diaphragm has not been verified.
3. The design of the lateral load resisting system, particularly in the north-south direction, to resist seismic forces is critical because of the lack of positive connections of the longer girder trusses to the supporting columns which could be a part of lateral load resisting systems.
4. The long girder trusses were not evaluated for horizontal deflections under wind and seismic loads.
5. None of the design deficiencies contributed to the collapse.

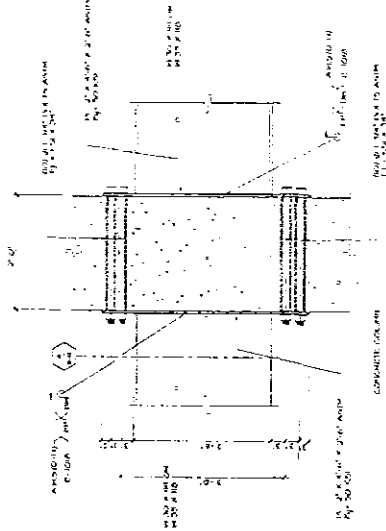
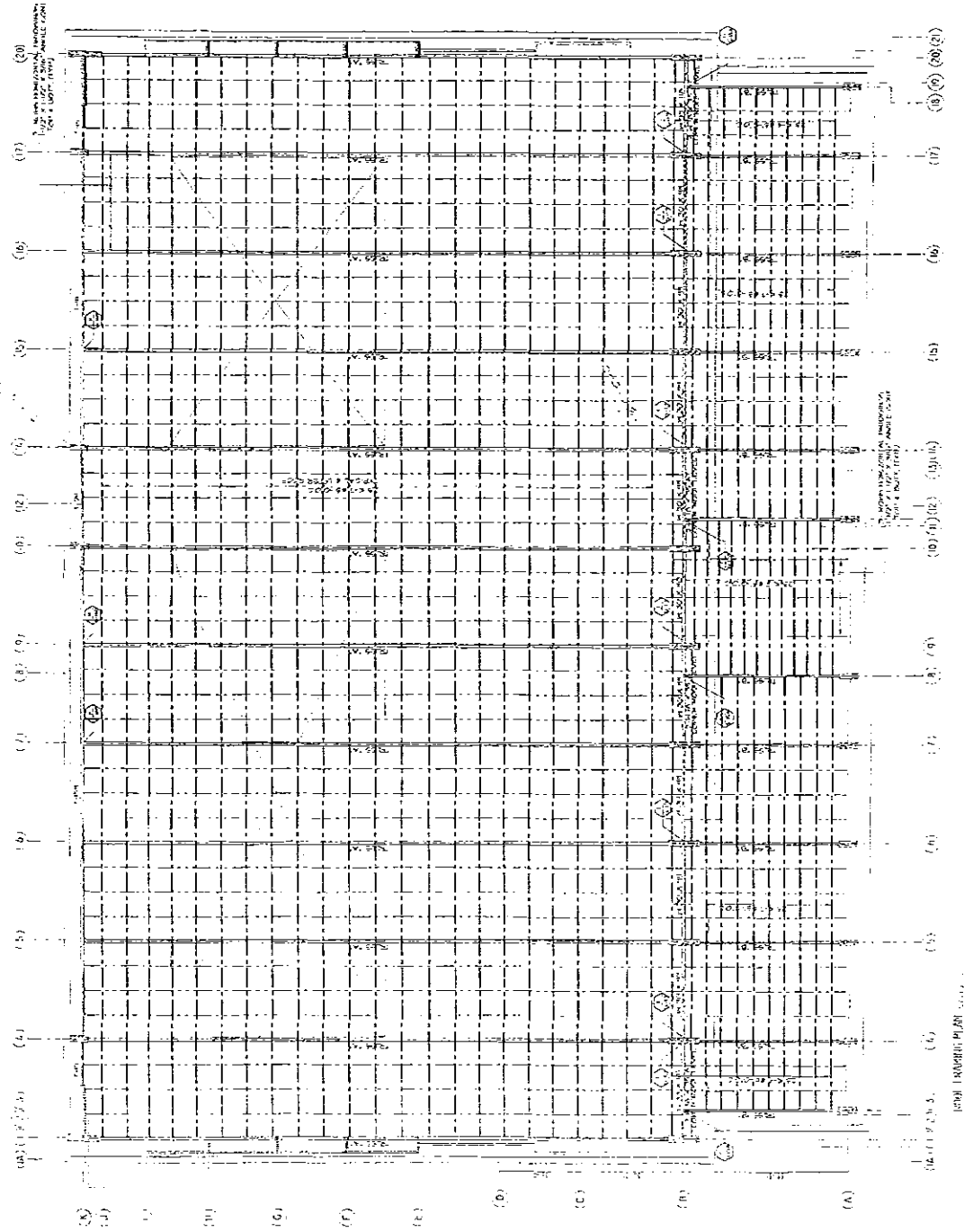
Natatorio de San Juan fase II

3/24/1974 U.S. 60

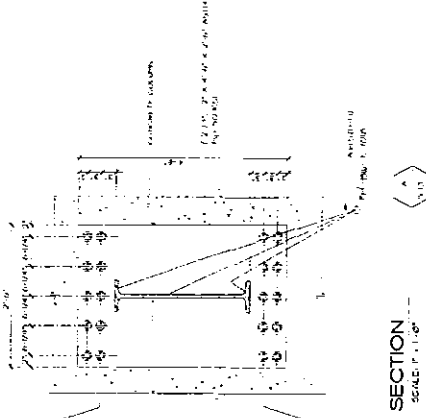


Cliente: F & R Const.	
Job No: P-3366	
S	A
N	T
I	A
G	O
S M MFG. CORP.	
M	E
T	A
L	L

FIGURE 1



CONN. DETAIL
SCALE 1/4" = 1'-0"
FROM AXES (1) TO (20)
ALONG AXIS (B)



SECTION
SCALE 1/4" = 1'-0"



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CONSULTING ENGINEERS
311 WEST 10TH STREET, SUITE 200
SAN JOSE, CALIF. 95110

Oliver
ARCHITECTS
1000 CALIFORNIA STREET, SUITE 100
SAN JOSE, CALIF. 95133

HON. JORGE SANTINI PADILLA
MAYOR OF SAN JOSE CITY

DATE

REVISION

SCALE

AUDITORIUM FLOOR PLAN

S-13

FIGURE 2

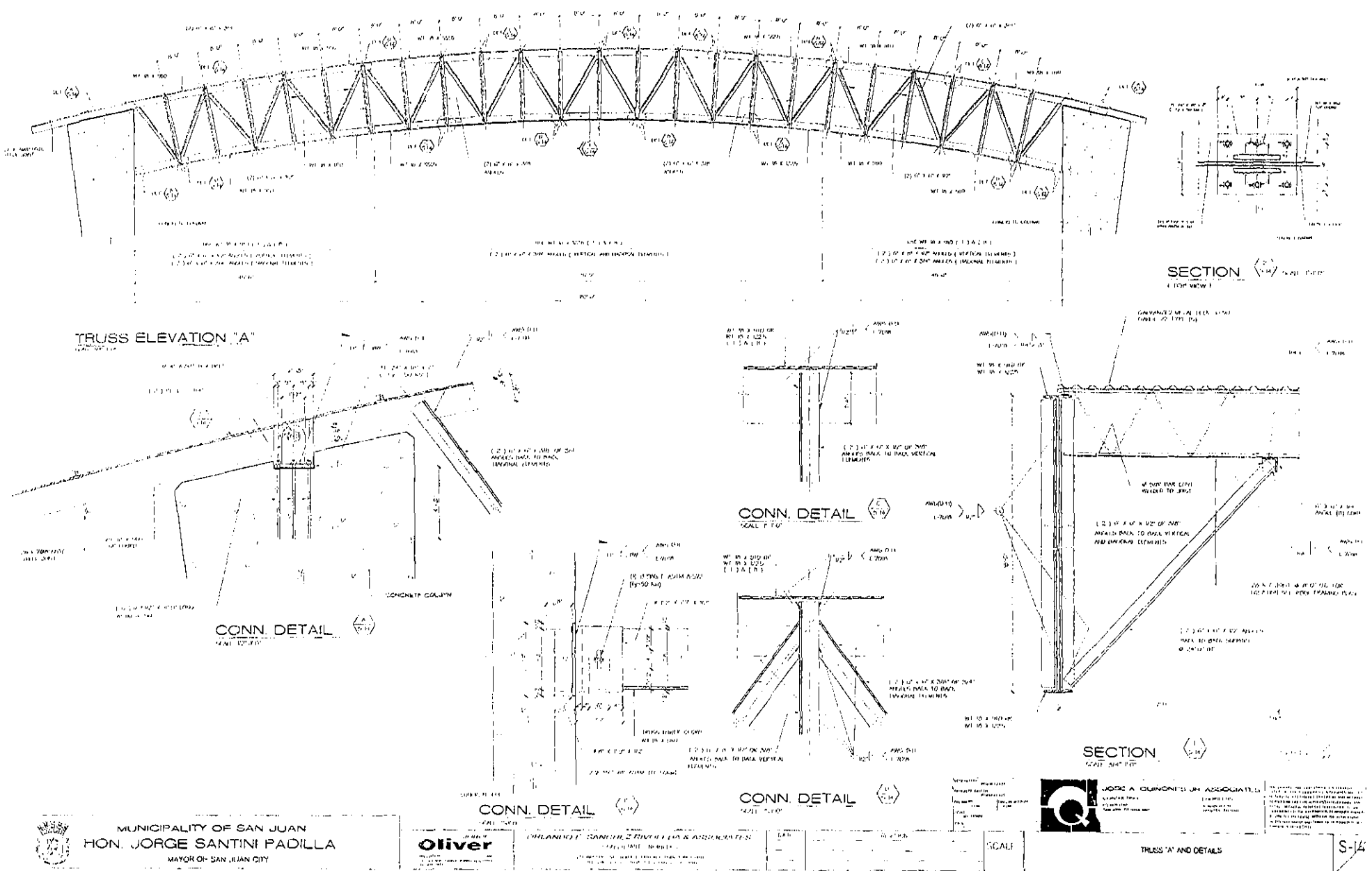
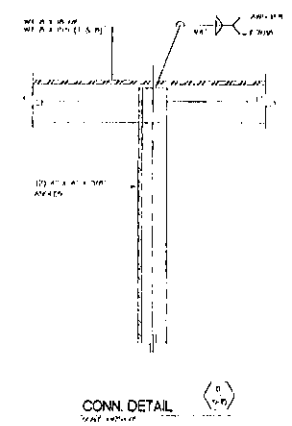
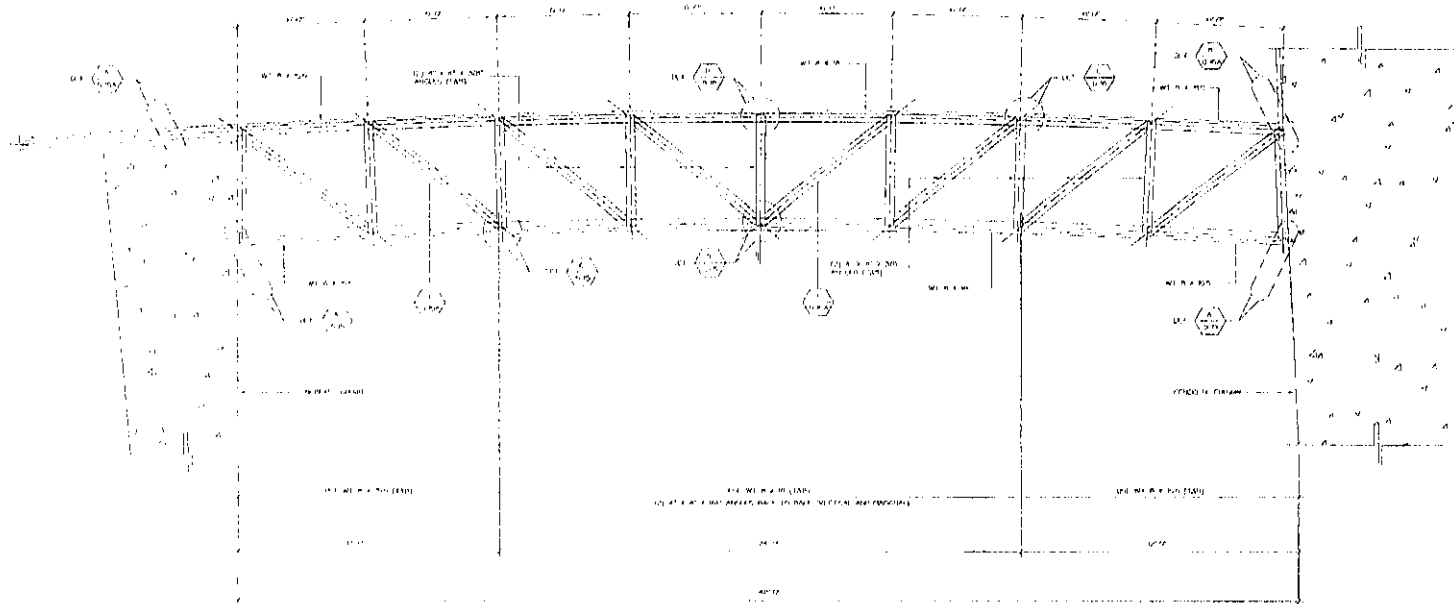
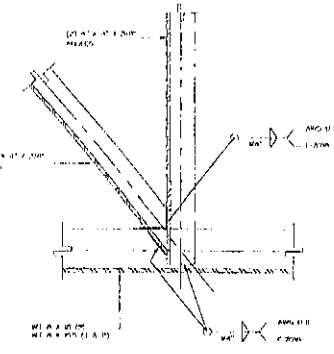
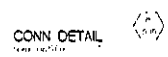
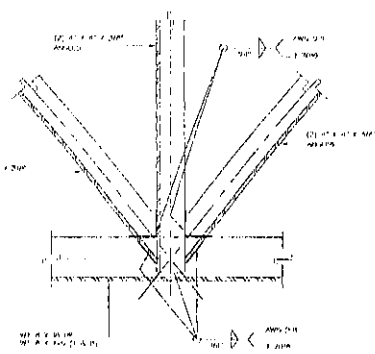
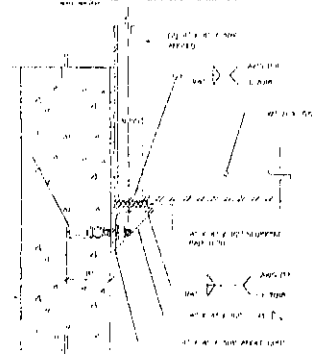


FIGURE 3



TRUSS ELEVATION 'B'



MUNICIPALITY OF SAN JUAN
HON. JORGE SANTINI PADILLA
MAYOR OF SAN JUAN CITY

JORGE
COMUNICACIONES

ORLANDO F. SANCHEZ RIVERO & ASSOCIATES
CONSULTANTS ENGINEERS

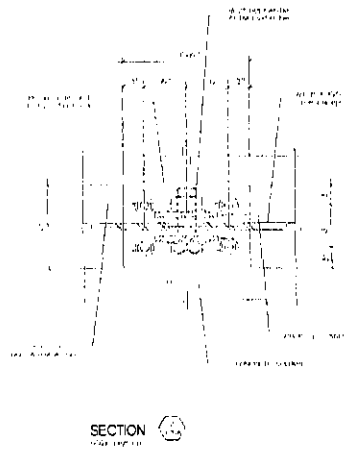
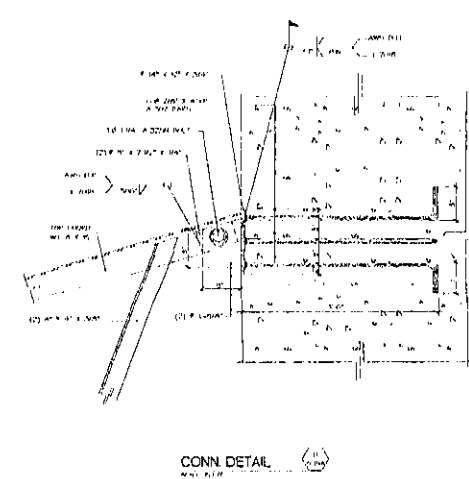
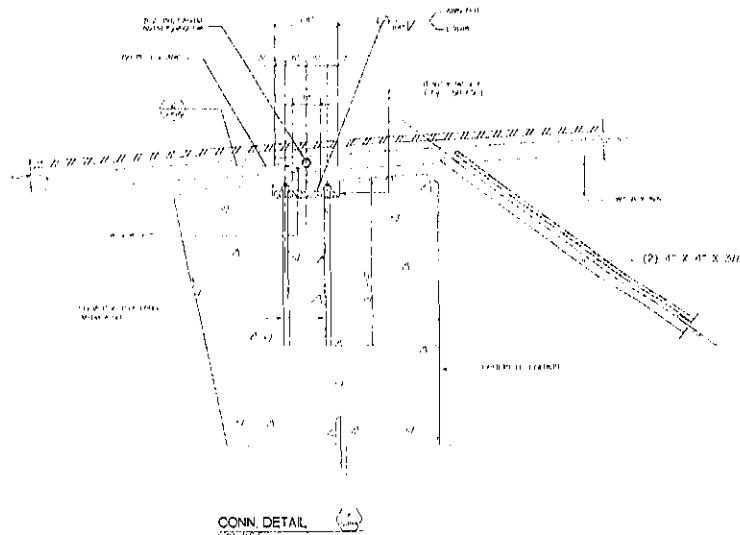
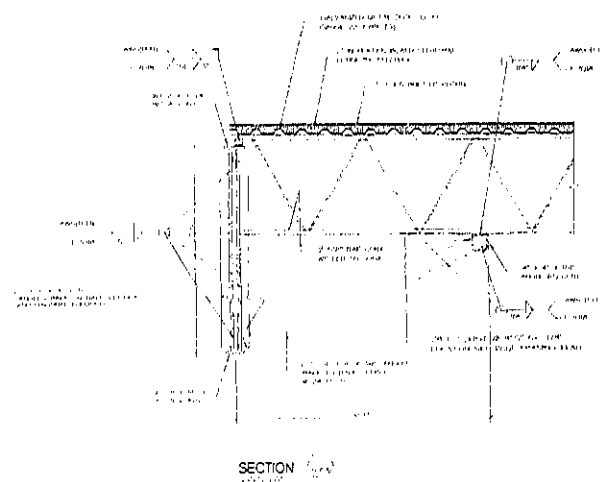
DATE: 10/10/2013
BY: [Signature]
SCALE: 1/4" = 1'-0"



JOSE A. GUINONES JR. ASSOCIATES
CONSULTANTS
1000 N. 10TH AVE.
SUITE 200
SAN JUAN, P.R. 00906

TRUSS 'B' AND COLUMN DETAILS

FIGURE 4



MUNICIPALITY OF SAN JUAN
HON. JORGE SANTINI PADILLA
MAYOR OF SAN JUAN CITY

JORGE
SANTINI PADILLA

ORLANDO E. SANCHEZ RIVERO & ASSOCIATES
STRUCTURAL ENGINEERS

DATE

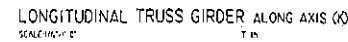
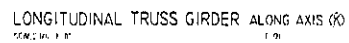
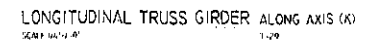
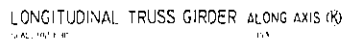
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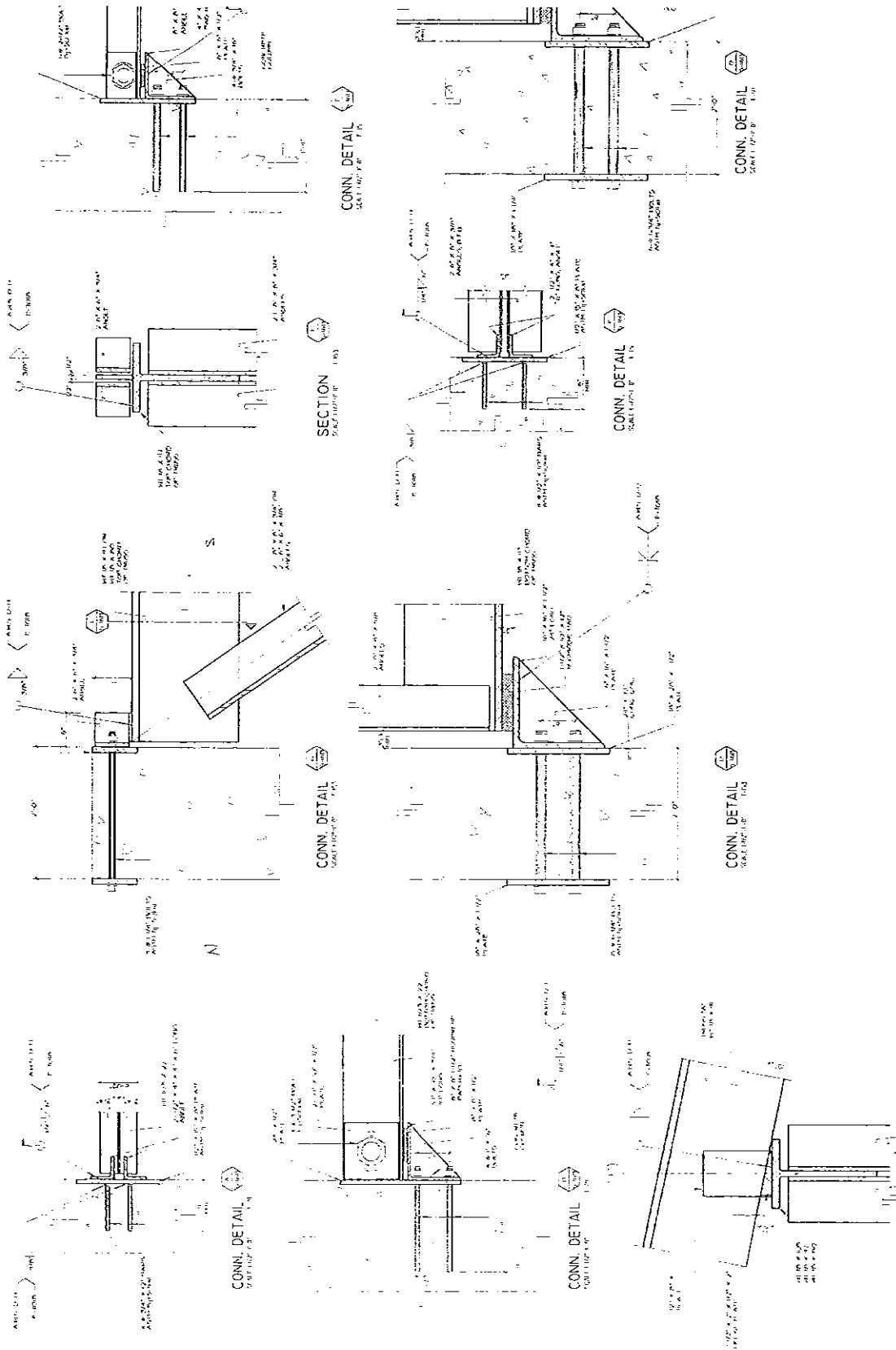
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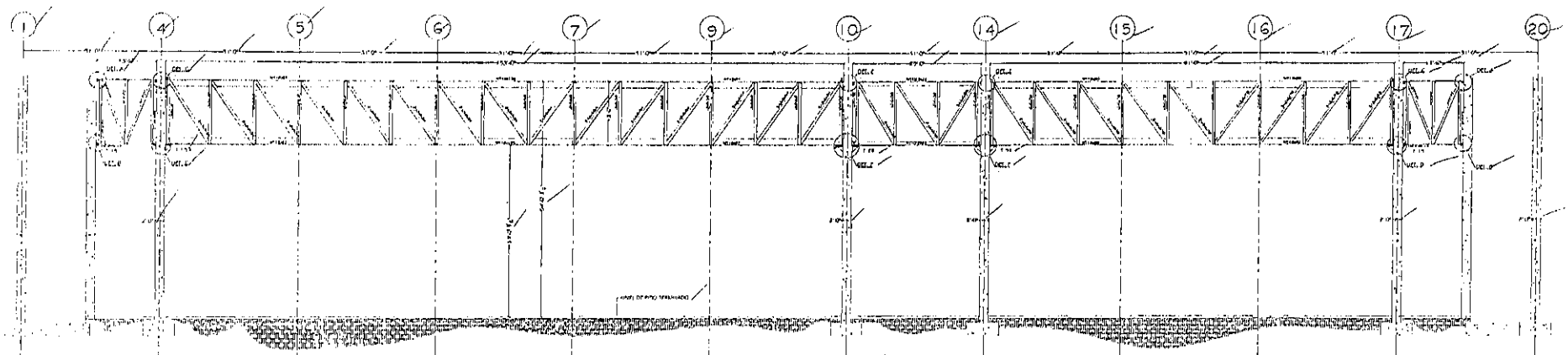
COLUMN DETAILS AND SECTIONS

15A

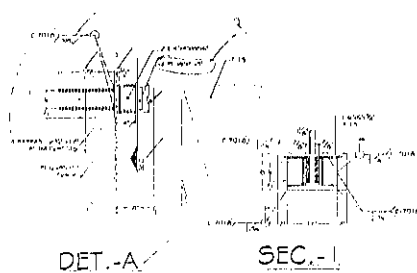
FIGURE 5





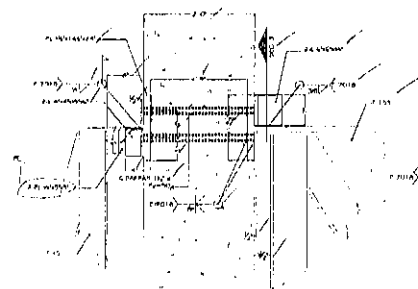


SECCION EJE-K

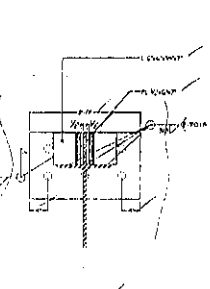


DET.-A

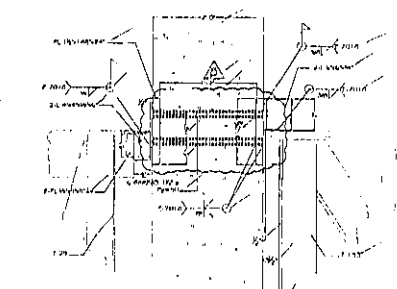
SEC.-1



DET.-C

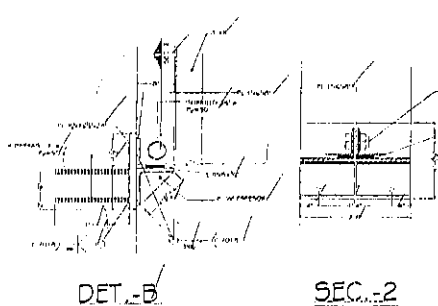


SEC.-3



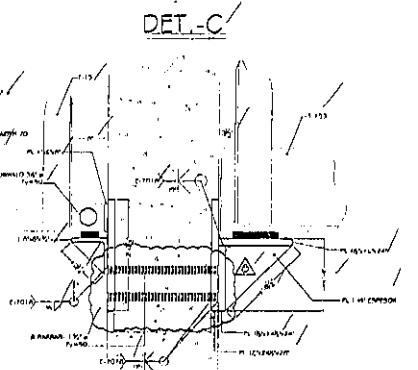
DET.-E

APPROVED SUBJECT TO GOVERNMENT INSPECTION
 AS INDICATED
 DESIGNED BY: [Signature]
 CHECKED BY: [Signature]
 JOSE CON...
 SEP 14 2004

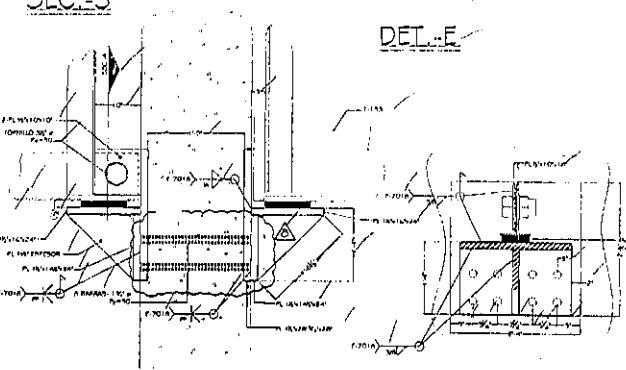


DET.-B

SEC.-2



DET.-D

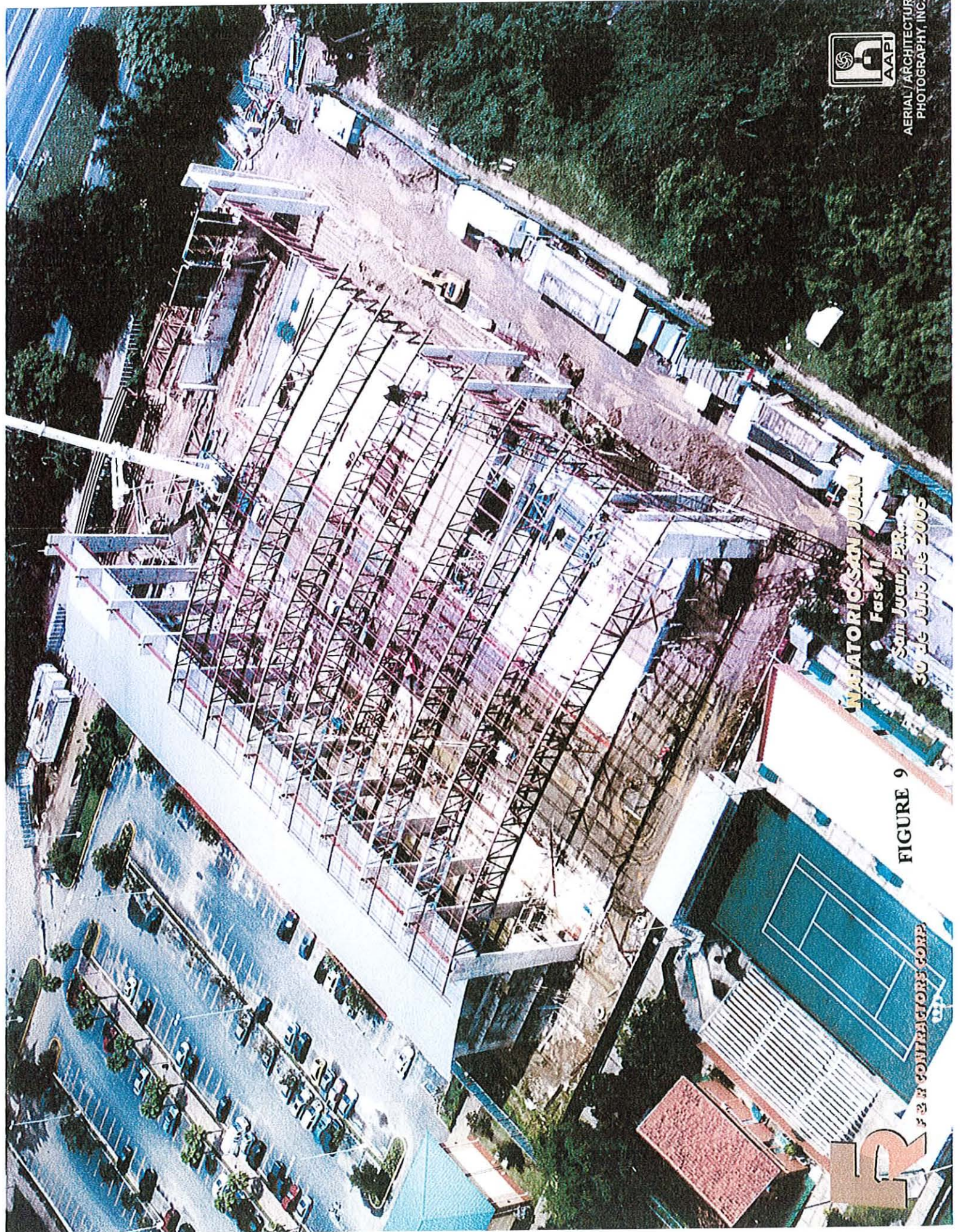


DET.-F

SEC.-4

10	Santiago Metal	TRUSS GIRDER ALONG AXIS-K
IP-3561		
25/01/08	S&R CORP	NATATOR O. CAHUAN
1 A.C.	N/A	NO. 5282
IMP. MATERIAL	IMP. MATERIAL	IMP. MATERIAL

FIGURE 8



AERIAL / ARCHITECTURE
PHOTOGRAPHY, INC.

AUDITORIO SAN JUAN
Fase II
San Juan, P.R.
30 de Julio de 2005

FIGURE 9

P & R CONTRACTORS CORP.





FIGURE 10



FIGURE 11



FIGURE 12



FIGURE 13



FIGURE 14

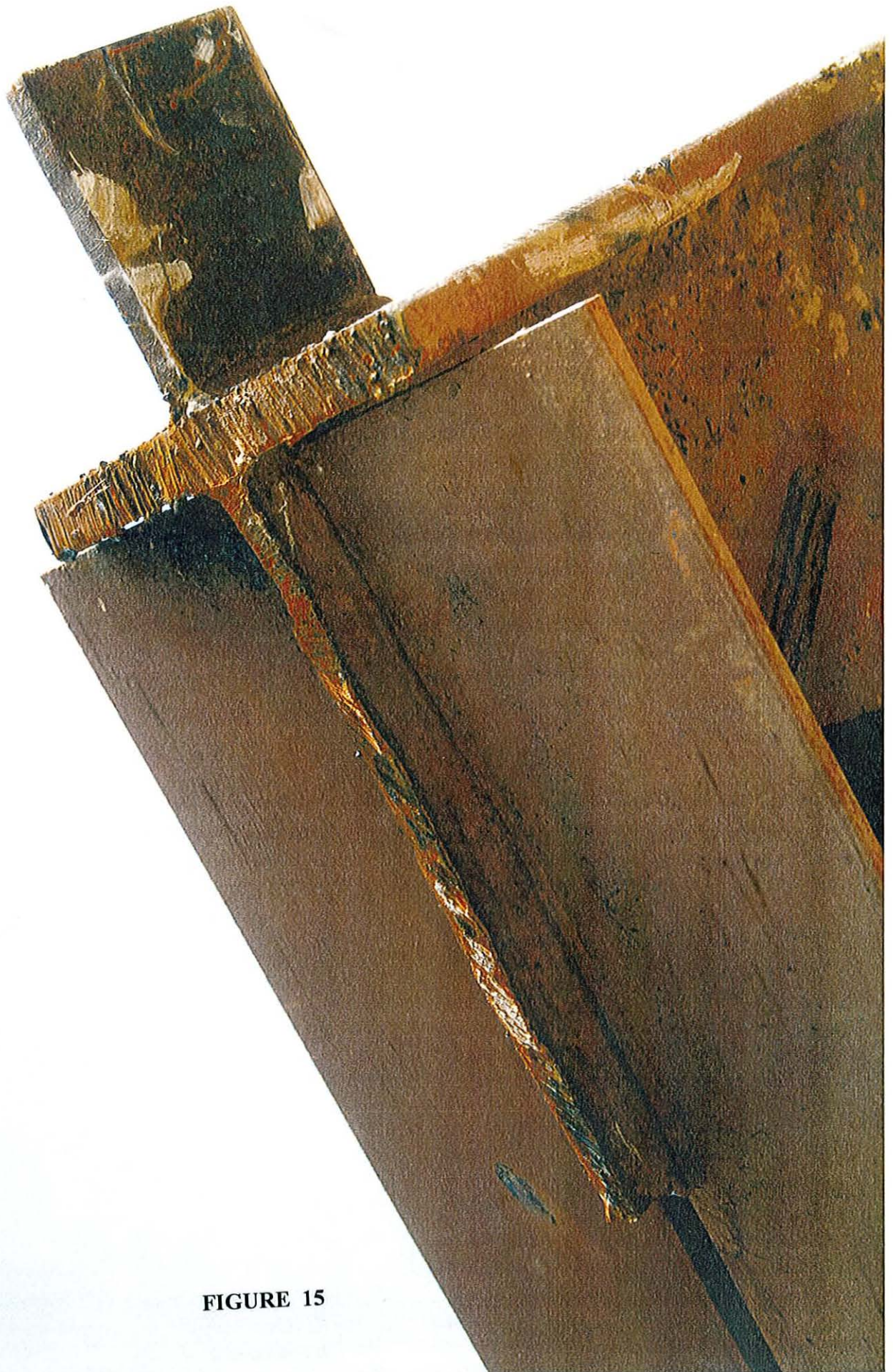


FIGURE 15



Truss Support

FIGURE 16

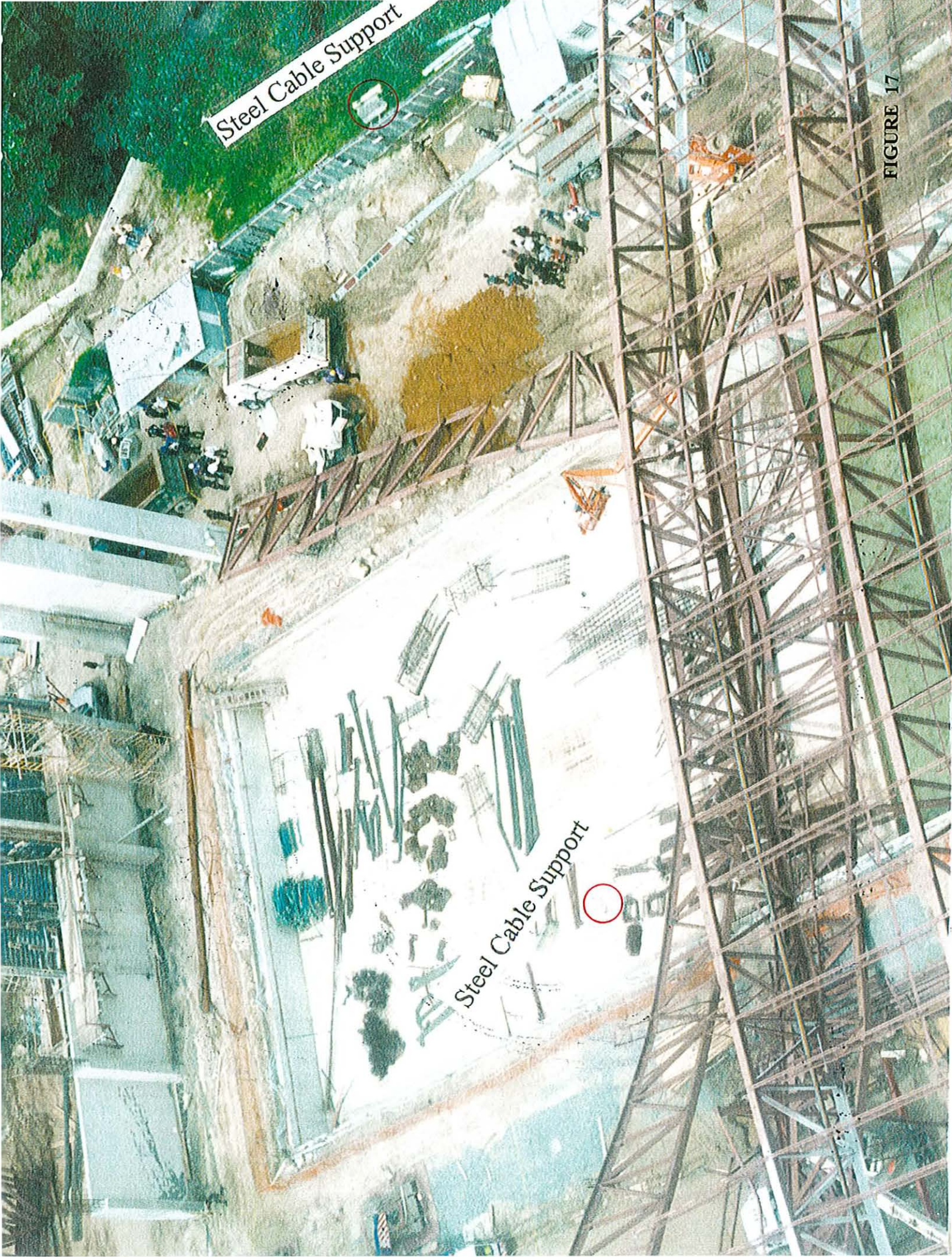


FIGURE 17

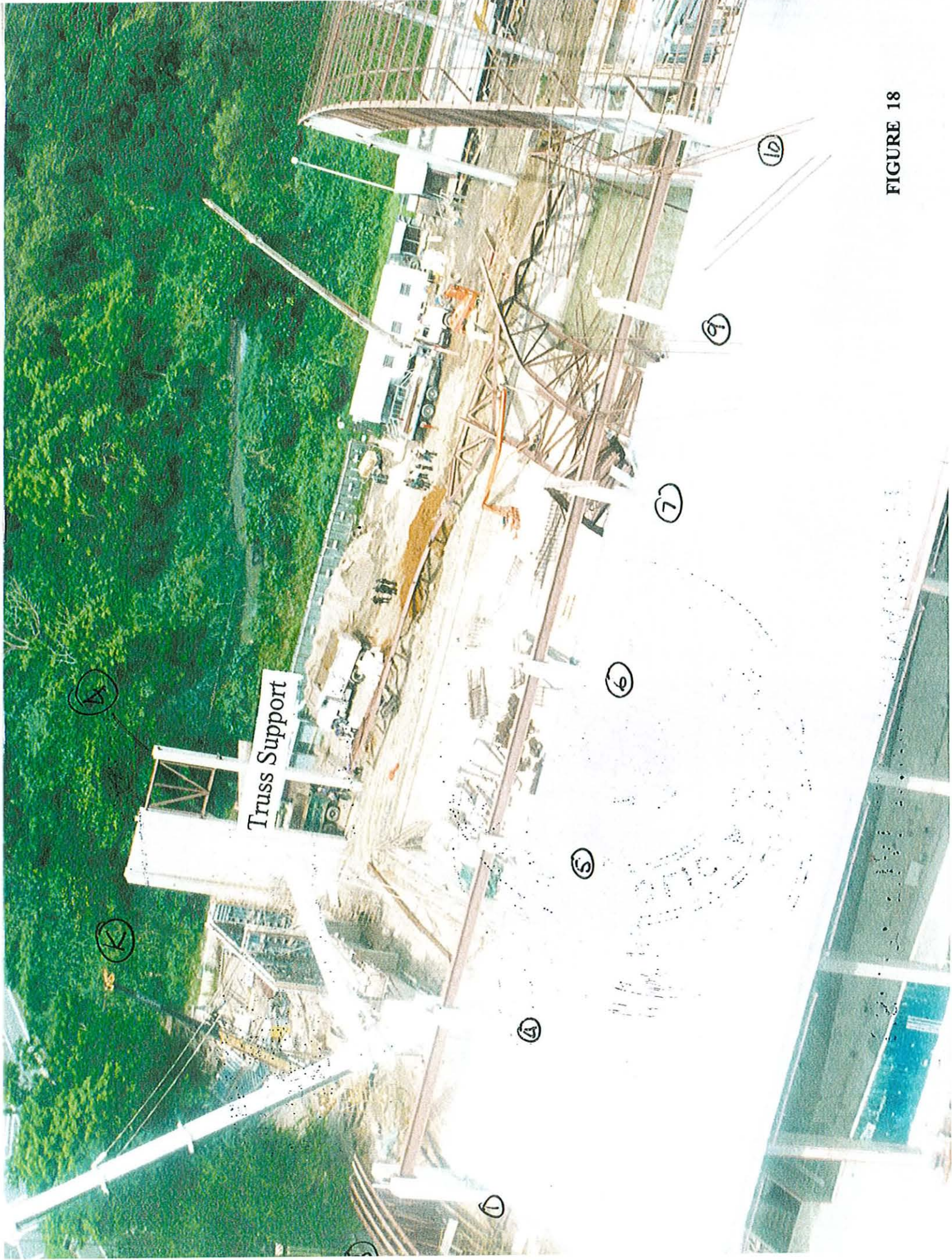


FIGURE 18



FIGURE 19



Steel Cable Support

FIGURE 20