Investigation of August 14, 1990 Collapse of Precast Concrete Beams At Airside Building, Midfield Terminal Project, Greater Pittsburgh International Airport, Allegheny County, Pennsylvania

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

December 1990

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Occupational Safety and Health Administration
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EXECUTIVE SUMMARY

A construction worker was killed and another worker was seriously injured on August 14, 1990, when several precast concrete beams, a column and hollow core concrete planks collapsed during the construction of the Airside Building of the Midfield Terminal Project at the Greater Pittsburgh International Airport, Allegheny County, Pennsylvania. At the time of the accident, the erection and placement of the precast hollow core concrete planks at the roof level in the southeast arm of the Airside Building was underway.

Representatives from the OSHA Area Office in Pittsburgh, Pennsylvania arrived at the accident site within 6 hours. The Office of Construction and Engineering from the National Office in Washington, D.C., was requested to provide assistance in determining the cause of the accident.

Based on eyewitness accounts, observations of the collapsed structure, concrete core test results and structural analysis, the Occupational Safety and Health Administration concludes that:

1. The cause of the collapse was the failure of the precast roof beam marked RB-35 placed along column line B-20 between column lines 17 and 16 due to inadequate development length of #7 bottom rebars.

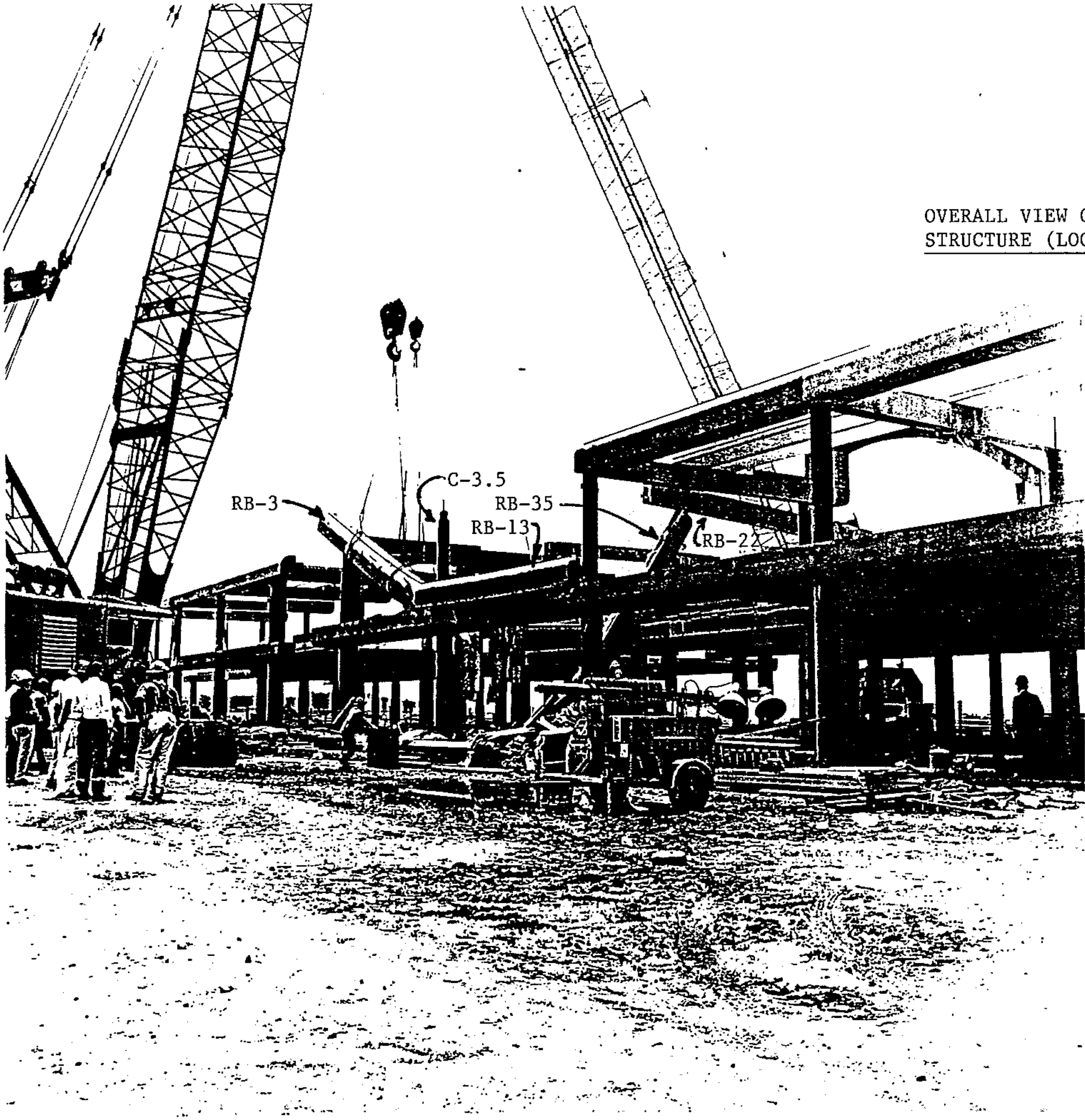
2. Other design and construction deficiencies observed during the investigation and noted in the report did not contribute to the collapse.

1.0 INTRODUCTION

On August 14, 1990, at about 9:30 a.m., several precast concrete beams, a column, and hollow core planks at the roof and concourse levels of the southeast arm of the Airside Building of the Midfield Terminal Project at Greater Pittsburgh International Airport, collapsed during the erection and placement of hollow core precast planks of the roof level. One construction worker, on the concourse level below, died due to the falling debris of the collapsed beams and planks. Another construction worker was seriously hurt. Figure 1.01 is a photograph taken after the collapse had taken place.

Personnel from the OSHA Area Office arrived at the scene 6 hours after the accident and collected evidence in the form of photographs and videotapes. The OSHA Office of Construction and Engineering, from Washington, D.C., was requested to provide assistance in the investigation of the accident. The purpose of the assistance was to determine the cause of the accident. Representatives from the Office of Construction and Engineering visited the site on August 16, September 18, and October 25, 1990, to gather relevant information for the investigation, and to conduct the joint interviews of the designer of the precast elements and the structural engineer of record.

The OSHA investigation involved eyewitness accounts; interviews of the designer, engineers, and quality control and precast fabricator personnel; observation of the collapsed structural elements; material property tests of the concrete beams and structural analysis to determine the cause of the accident. Throughout the course of the investigation, the Office of Construction and Engineering worked together with the personnel of Pittsburgh OSHA Area Office. The late Harlan B. Jervis, OSHA Compliance Officer, made significant contributions to this investigation.



OVERALL VIEW OF THE COLLAPSED STRUCTURE (LOOKING TOWARD SOUTH)

2.0 CONDUCT OF THE INVESTIGATION

A copy of structural drawings prepared by the structural engineer of record, and a copy of the Project Manual containing technical specifications prepared by the project architect and engineer were provided by the construction manager of the project to the OSHA Area Office. These documents were then forwarded to the OSHA Office of Construction and Engineering in Washington D.C. for review. As specified in the above contract documents, the contractor was responsible for the engineering design of the precast structural members. Selected shop drawings showing design and detailing of the precast members which failed in the collapse were also obtained from the construction manager of the project. A guideline of "The precast concrete erection sequence" prepared by the designer of the precast elements was also forwarded to the OSHA Office of Construction and Engineering through the same channel.

Core samples from the failed precast roof beam RB-35 were taken to determine the concrete strength by an independent testing laboratory. The same laboratory was also employed by the construction manager of the project, to verify the development and splice lengths of embedded reinforcing steel and concrete cover of rebars in the precast elements which had either been erected prior to the accident or were ready to be installed.

Interviews of eyewitnesses and engineers were conducted to obtain accounts of the collapse, to identify the mode of failure, to determine the construction activities preceding the collapse and the design and casting procedures of the precast elements.

A structural analysis was conducted to compute stresses in the failed members at critical locations due to the erection loads occurring immediately preceding the collapse. Structural analysis also included checking critical member stresses from roof planks scheduled to be placed and supported by the beam prior to placement of the topping and the pour strip.

The conclusion regarding the cause of the failure was based on all the above information.

3.0 DESCRIPTION OF THE COLLAPSE

The construction site is located at the Midfield Terminal Project at the Greater Pittsburgh International Airport, Pittsburgh, Pennsylvania. The building under construction, called the Airside Building is a structure consisting of precast concrete beams, columns and precast prestressed hollow core concrete planks. The plan of the building is of X shape with four arms and a center core at the junction of the four arms. Figure 3.01 shows the key plan of the Airside Building and identifies the location of the collapsed structure.

At the time of the accident, on August 14, 1990, numerous construction activities of the project were underway. The activity in the vicinity of the accident was centered near column lines B-19, B-20 and B-21 between column lines 16 and 17 where the precast roof planks spanning over roof beams RB-14 and RB-35, and RB-35 and RB-36 were being placed. Figures 3.02 and 3.03 show the partial roof erection plan and the second floor (concourse level) erection plan of the southeast arm of the building. These plans were prepared by the designer of the precast structural frame , showing the shop identification marks of the precast members. Throughout this report, all the elements will be referred to by the same identification marks as they appear on the shop drawings. Figures 3.04, 3.05 and 3.06 are the partial roof, second floor (concourse level) and foundation plan of the same area as contained in the set of the structural drawings prepared by the structural engineer of record, (as discussed in chapter 2 of this report). These drawings were furnished to the precast elements designer to be used as a general guide to the basic framing system as well as the precast member sizes.

In the southeast arm between column lines B16 to B25, a majority of the columns and beams of the concourse level and roof level including the spandrel beams were erected, as shown in figures 3.07 and 3.08, prior to the day of the accident. The concourse level floor planks had been placed in position in many areas and the erection of the roof planks was in progress.

Prior to the accident, five roof planks were placed in position, two between column lines B19 and B20 and three between column lines B20 and B21 in the bay bounded by column lines 16 and 17. The erected roof planks are identified in figure 3.09. The roof beams marked RB-14, RB-35, RB-36, RB-22 and the spandrel beams RB-3 and RB-13 were already in place, as stated earlier.

As the placement of the sixth roof plank progressed, a sudden collapse of the roof beam marked RB-35 and perimeter spandrel beams marked RB-13 and RB-3 occurred, following the release of the hoist line that was attached to the sixth roof plank. The three roof planks supported by beams RB-14 and RB-35 and the three roof planks supported by beams RB-35 and RB-36 collapsed and dropped on the concourse level slab. The six planks are identified as #701A, #616 and #701 between column lines B19 and B20 and #615, #701A and #701 between column lines B20 and B21. Figures 3.10 to 3.13 were photos taken after the accident. Due to the impact of the fall of the roof planks, the concourse level precast planks collapsed and dropped to the ground. The extent of the concourse level planks which sustained damage due to the fall of the roof planks is from column line B19 to midway between column lines B20 and B21. This can be seen from figures 3.14 to 3.16. Column marked C3.5 at intersection of column lines B20 and 17 also sustained damage. The column marked C3.4A collapsed but remained attached to the beam RB-35 as shown in figure 3.17. The concourse level spandrel beams marked B3B, B6 and B18 also sustained damage as a result of the roof spandrel beams dropping and resting on them.

Beam RB-35

The roof beam RB-35 failed at two locations. Figures 3.19, 3.20, and 3.21 are photos of the two failed location after the collapse. One location was at the vertical plane where the depth of the beam changed from 1'-11 7/8" to 3'-1 7/8" at a distance of 6'-0" from the center of column marked C3.5. This location is identified as "c"-"c" on the elevation view of the shop drawing on figure 3.18. A flexural type failure occurred resulting in a separation along this vertical plane, the bottom portion of the beam separated at the beam depth change whereas the top portions of the failed segments were held together by the top reinforcement of the beam. The other failure was at a location adjacent to the vertical plane where the depth of the beam changed from 12" to 1'-11 7/8" at a distance of 1'-2" from the center of column marked C3.5. This location is close to the mark "a"-"a" on figure 3.18, on this location a complete separation of the end concrete piece with the remaining portion of the beam had occurred. This exterior end section of concrete (approximately of 1'-6" long) was presumably crushed during the failure process.

Beam RB-13

Spandrel beam RB-13 was supported by the roof beam RB-35 at the column C3.5, and beam RB-14 at the column C.3.5A (Column at the intersection of column lines B19 and 17). The failure of the exterior end concrete piece of beam RB-35, as described earlier, resulted in the loss of the end bearing support for beam RB-13 at column C.3.5. The spandrel beam RB-13, 30'- 0" long, dropped vertically and rested on the concourse level spandrel beams. The concourse level spandrel beam was damaged resulting in spalling and cracks. See figures 3.14 and 3.15. The roof beam RB-14, that supported the other end of the beam RB-13, also sustained damage at the concrete bearing.

Beam RB-3

The end of the spandrel beam RB-3 at column marked C3.5 dropped and rested on top of the concourse level spandrel beam marked B-18. The other end of beam RB-3 remained to be supported on roof level at column marked C6.8. Figures 3.13 and 3.14 show the spandrel beam after the collapse. The beam sustained extensive damage resulting in spalling and cracks. The loss of support of the spandrel beam RB-3 at column C3.5 occurred due to the loss of the end section of beam RB-35 during the collapse.

Beam RB-22

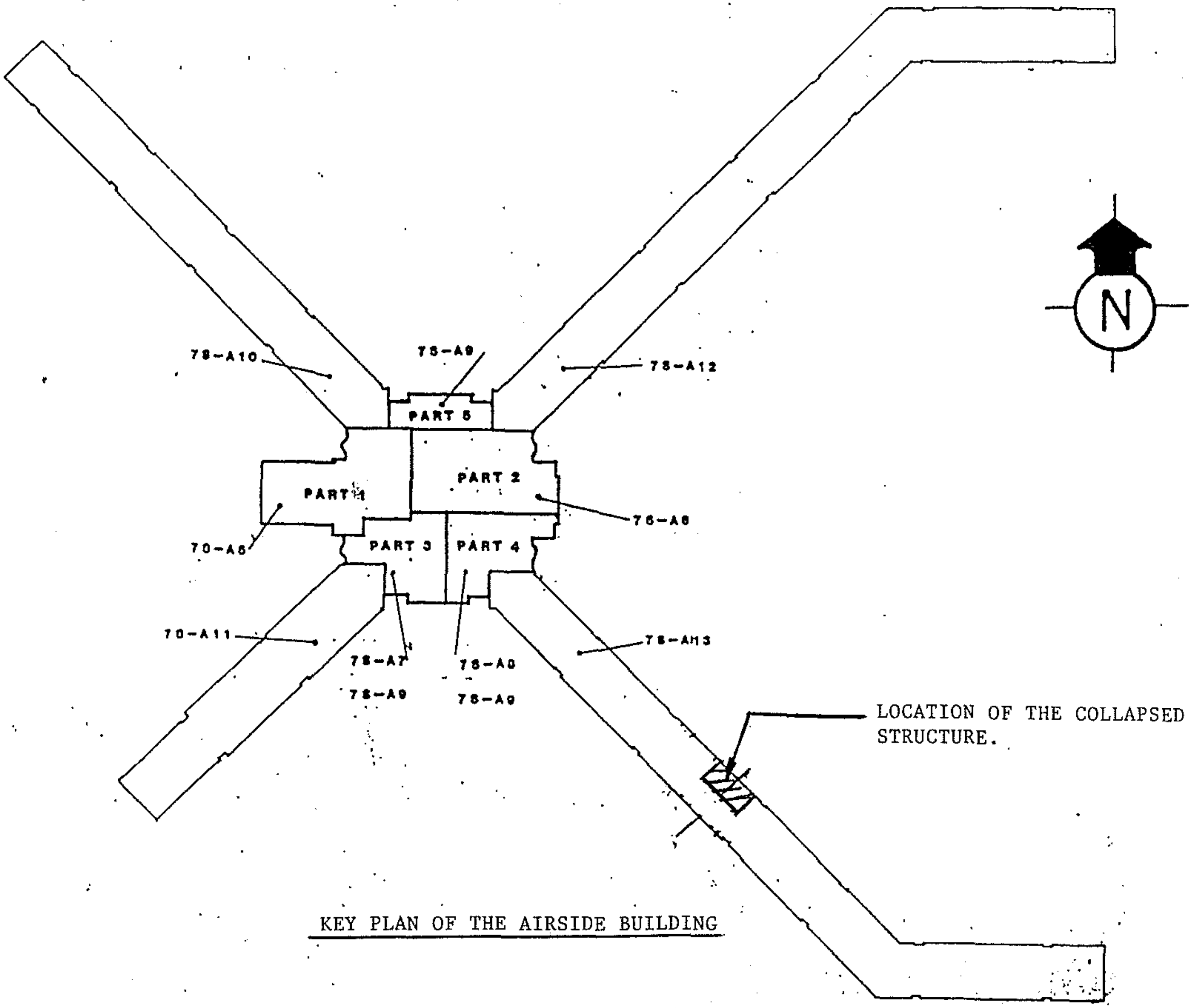
As illustrated in figure 3.22, beam RB-35 remained attached to column C.3.4A and ended in a tilted position. The left end of beam RB-22 supported by beam RB-35 near column C.3.4A, was raised higher but continued to rest on top of beam RB-35 at its edge after the collapse. One of the four #9 rebars connecting RB-35 and RB-22 snapped while the three others were bent as a result of the collapse. The 3" diameter holes in beam RB-22 for the 4-#9 rebars were grouted. These rebars are identified as 2-#902 and 2-#904 in figure 3.18. The other end of the beam RB-22 remained connected to the supporting beam (also identified as RB-35 in the Erection Plan by the precast designer).

Column C3.4A

Column marked C3.4A which was supporting the interior end of beam RB-35 failed at the top and bottom ends. There were eight rebars extending from the top of the column into the 11" x 11" opening in beam RB-35. See figure 3.23 for the reinforcement. As a result of the collapse, the column remained attached to beam RB-35 with its eight rebars bent and skewed as shown in figure 3.22. The base of the column, prior to the collapse, was resting on the concourse level beam marked B-2. The steel base plate of the column had four oversize holes through which four threaded rebars were to be fastened with washers and nuts. It is not known if the nuts and washers were placed and tightened. The base of the column was rotated and separated. See figures 3.22, 3.24 and 3.25 for the configuration of the column and beam RB-35 after the collapse. With the exception of some spalling and cracks at the top edge of the column, deformations were not observed along the length of the column. At the base of the column, concrete had spalled as a result of the column rotation.

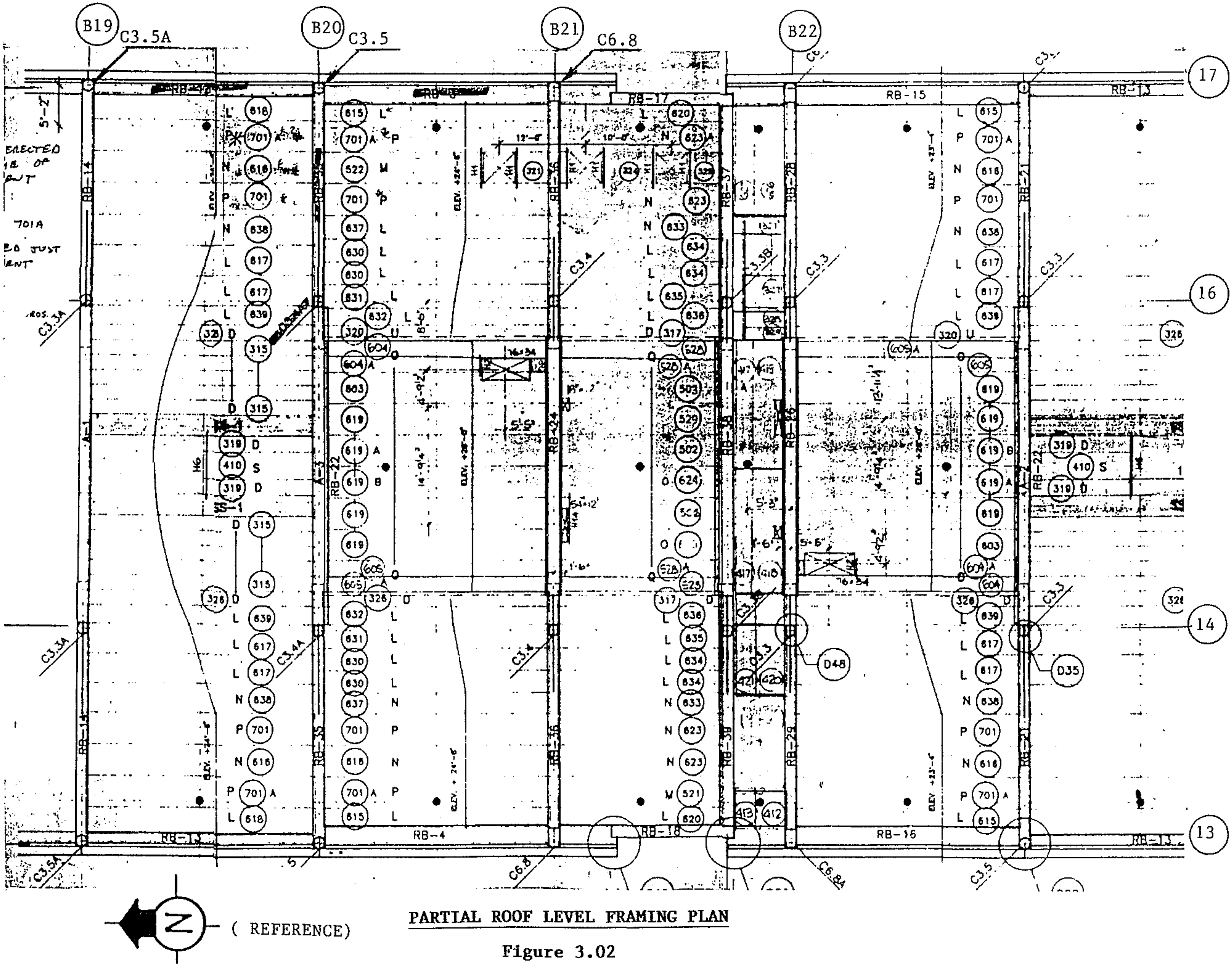
Column C3.5

The column marked C3.5 remained in position after the collapse as shown in figure 3.14. The top of the column, areas at the level of beam RB-35 bearing elevation ,the concrete had spalled, however, the 11"X11" grouted piece showed only minor spalling.



KEY PLAN OF THE AIRSIDE BUILDING

Figure 3.01



PARTIAL ROOF LEVEL FRAMING PLAN

Figure 3.02

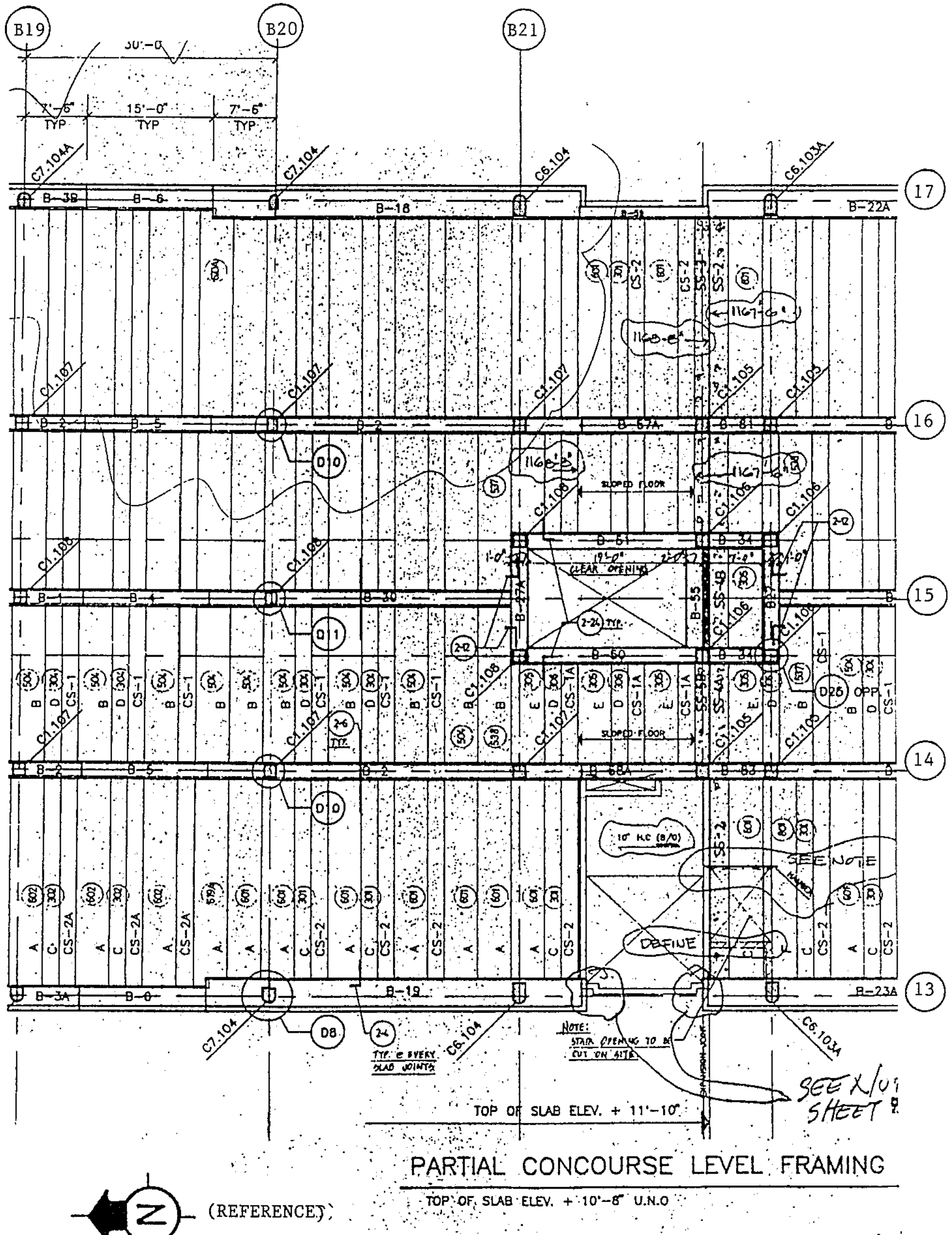
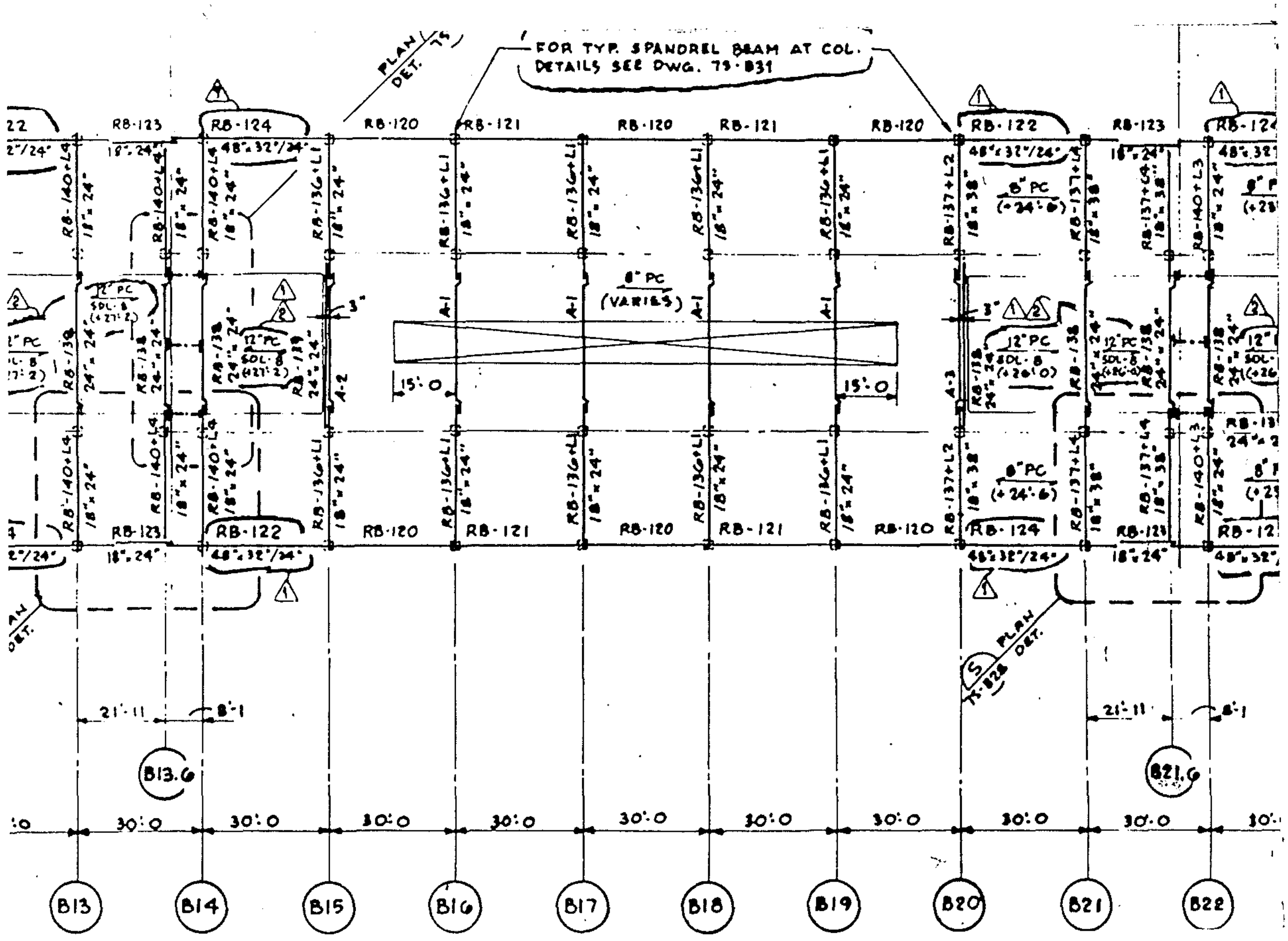
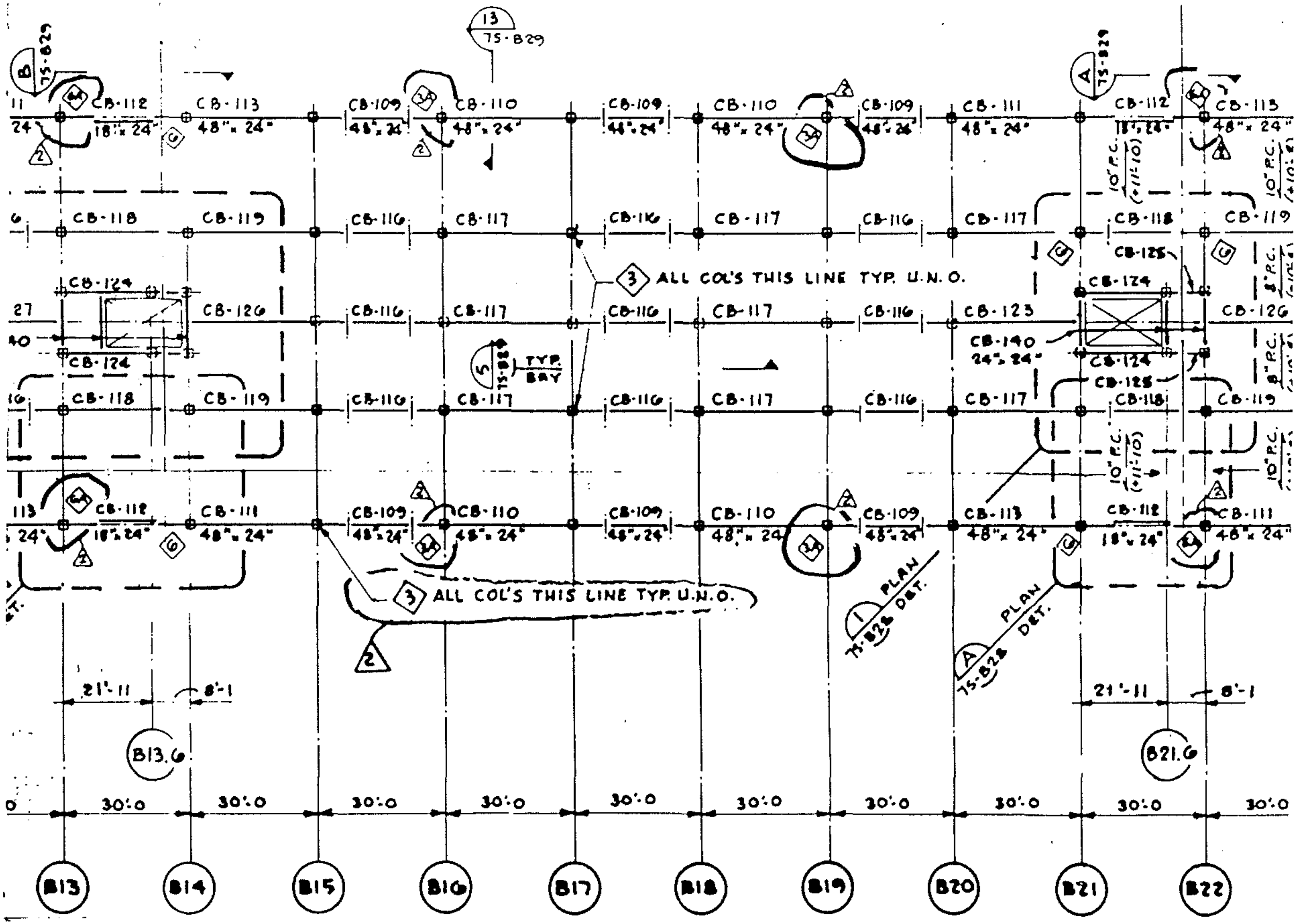

B19
B20
B21
17
16
15
14
13
TOP OF SLAB ELEV. + 11'-10"
PARTIAL CONCOURSE LEVEL FRAMING
TOP OF SLAB ELEV. + 10'-8" U.N.O
(REFERENCE)

Figure 3.03



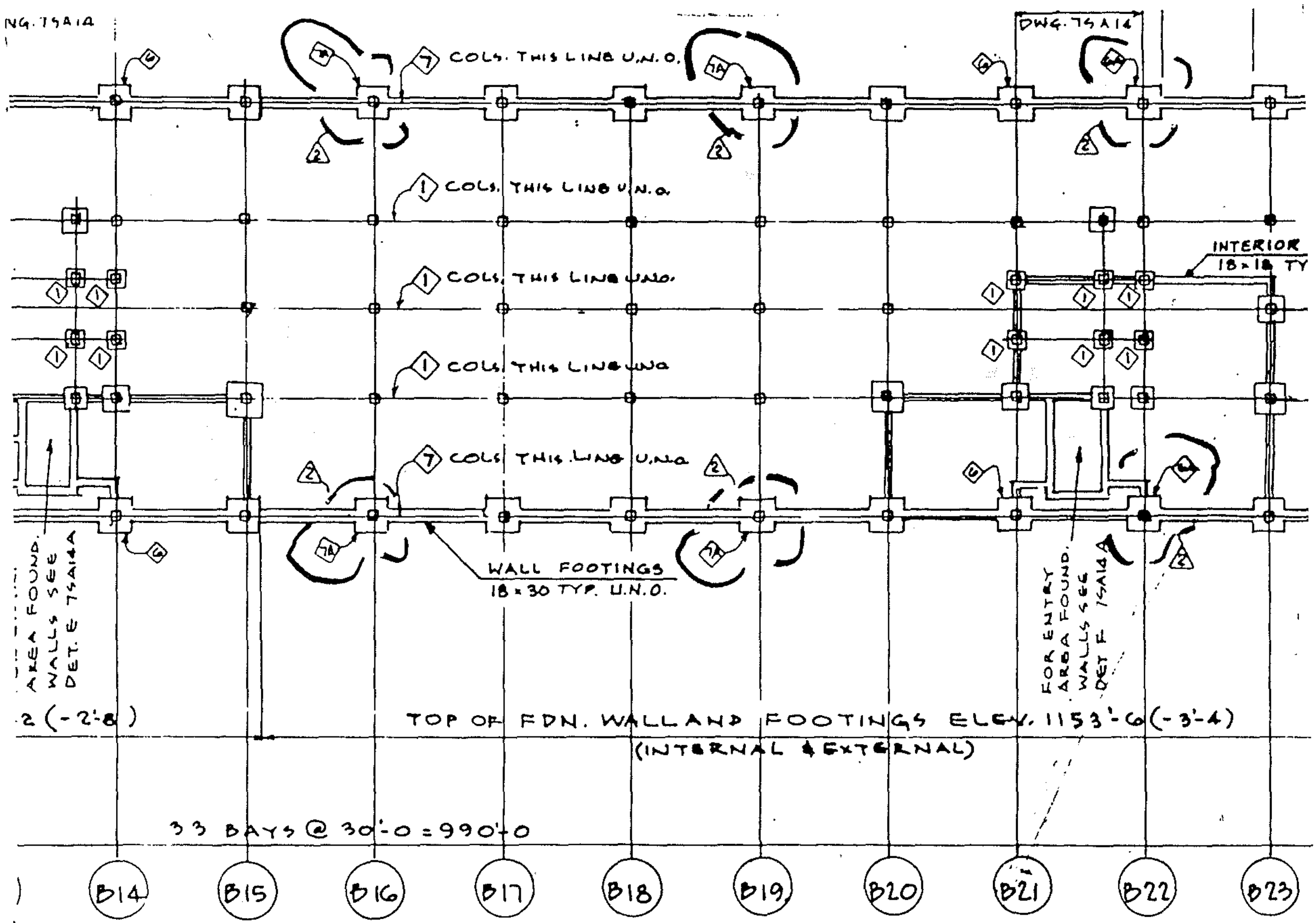
PARTIAL ROOF LEVEL PLAN

Figure 3.04



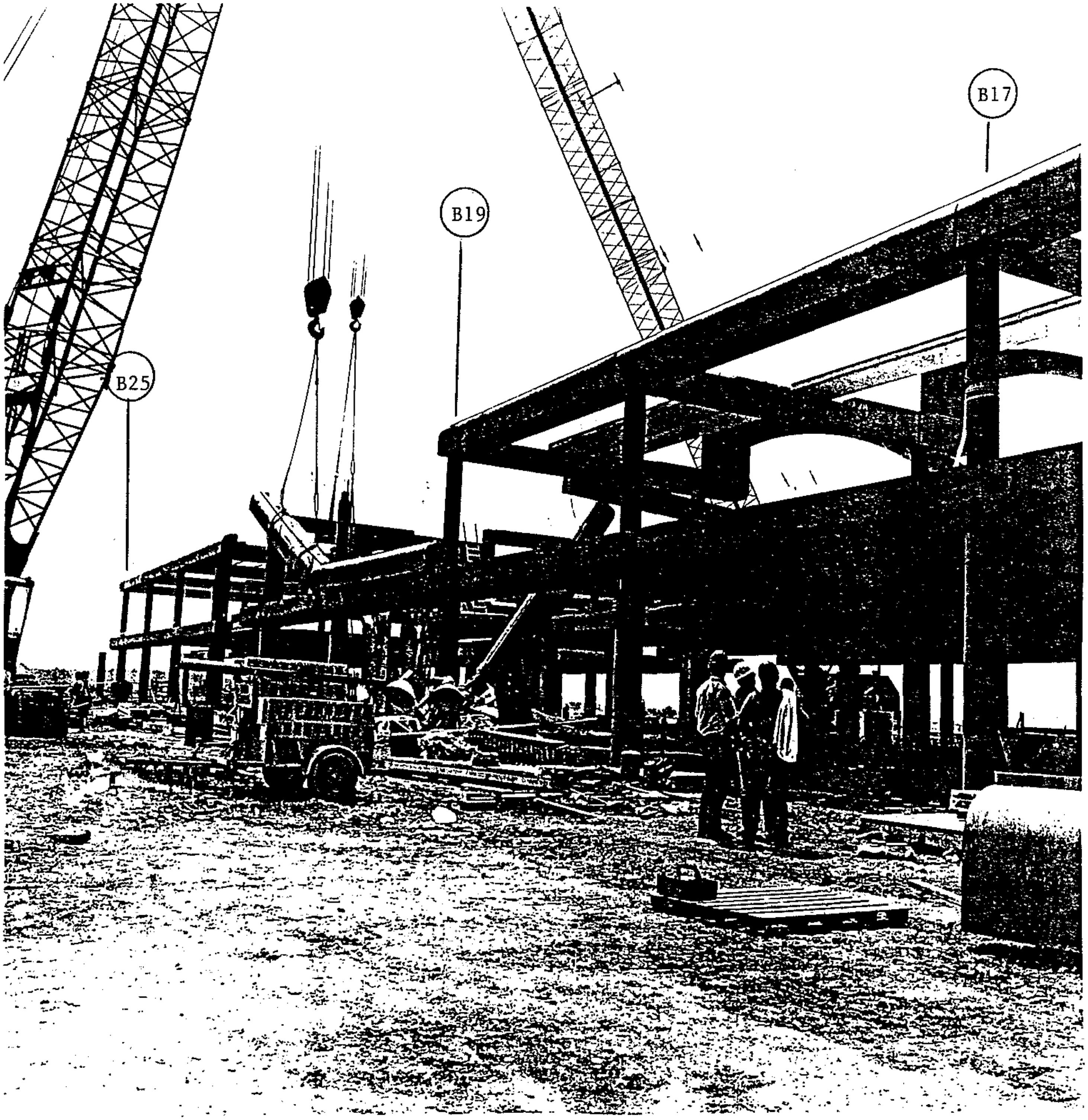
PARTIAL CONCOURSE LEVEL PLAN

Figure 3.05



PARTIAL FOUNDATION PLAN

Figure 3.06

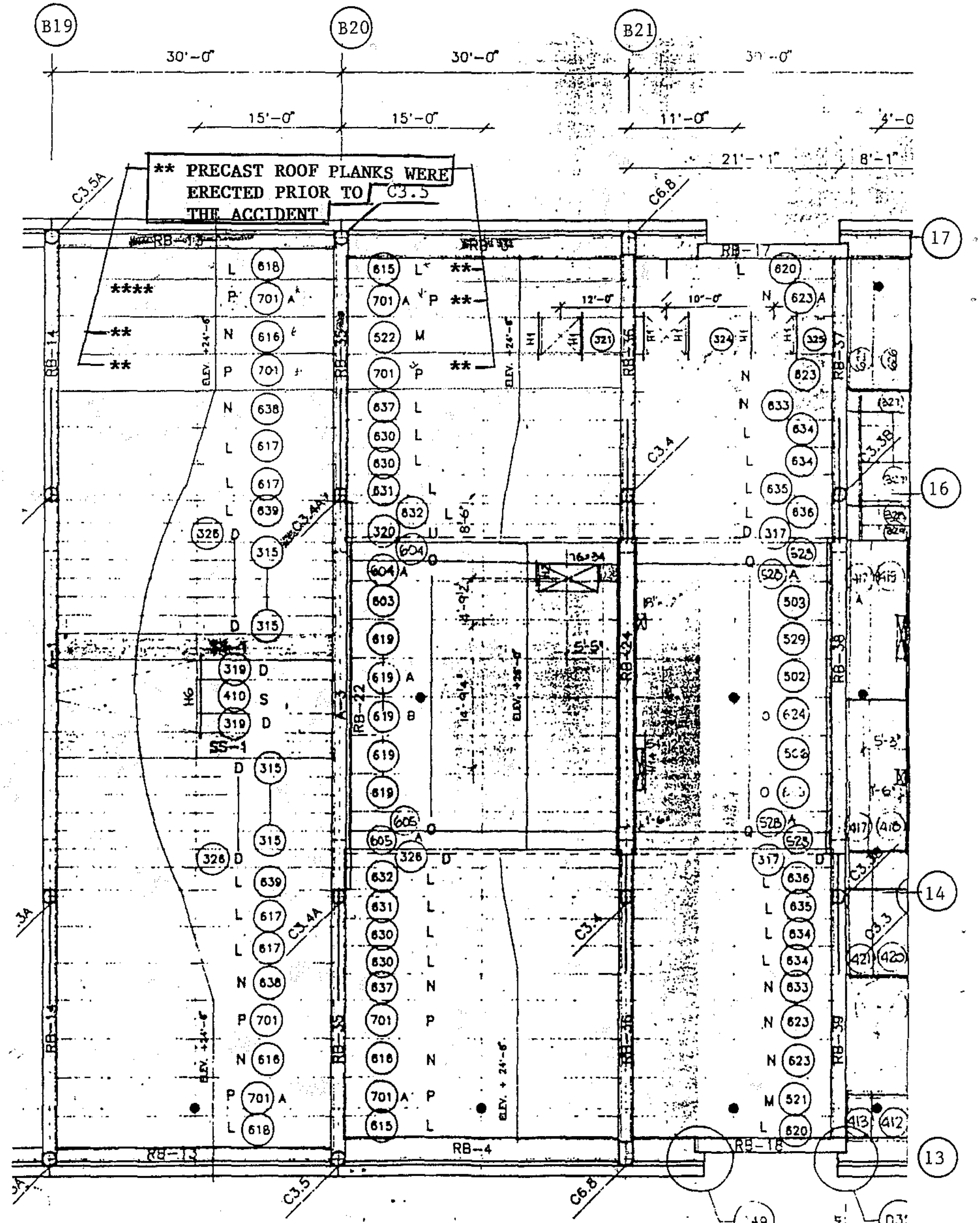


OVERALL VIEW OF THE
COLLAPSED STRUCTURE.

FROM COLUMN LINE
B17 TO B25

SOUTHEAST ARM FROM COLUMN LINE B14 LOOKING TOWARD NORTH.

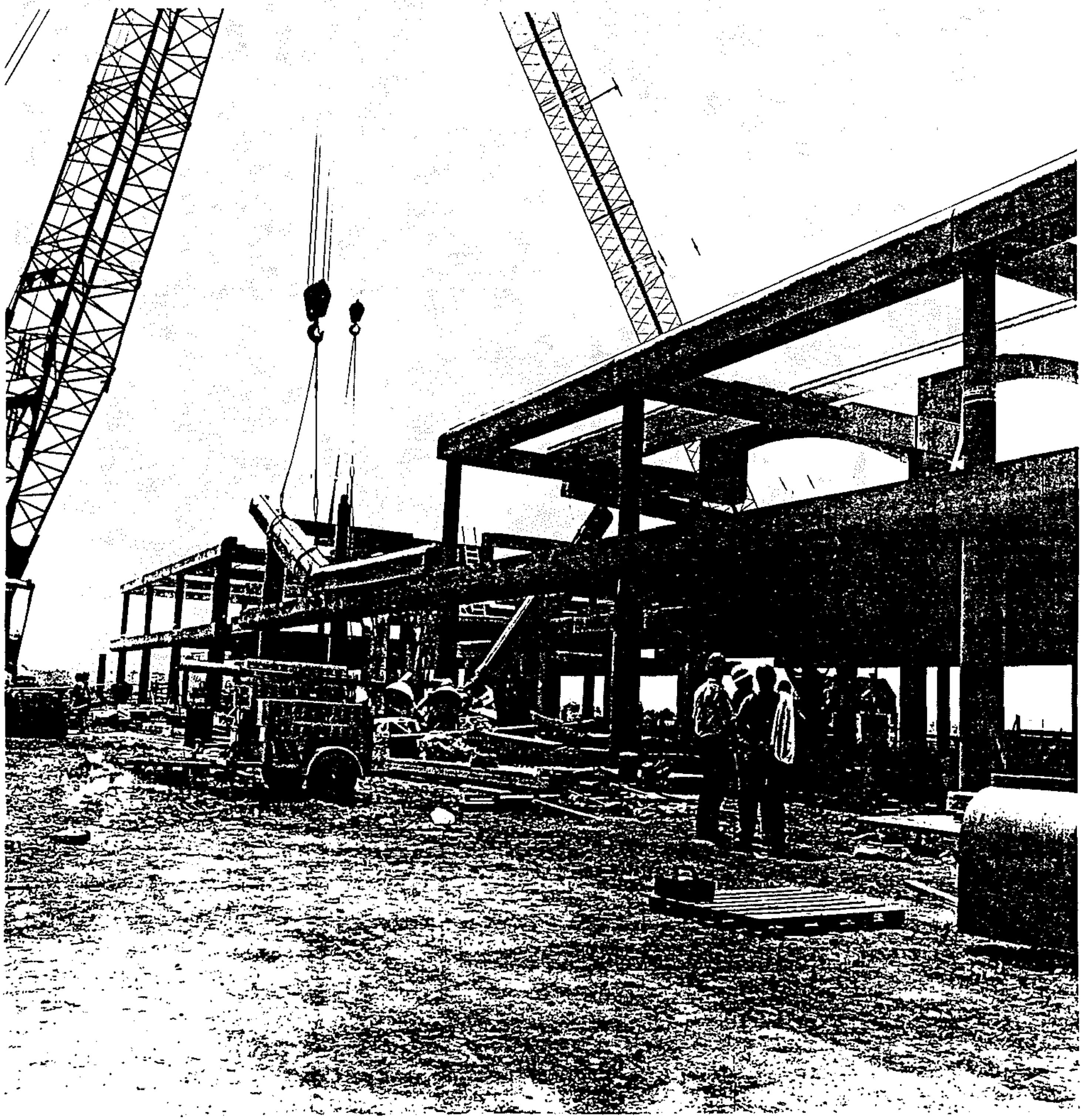
Figure 3.08



PARTIAL ROOF LEVEL FRAMING PLAN

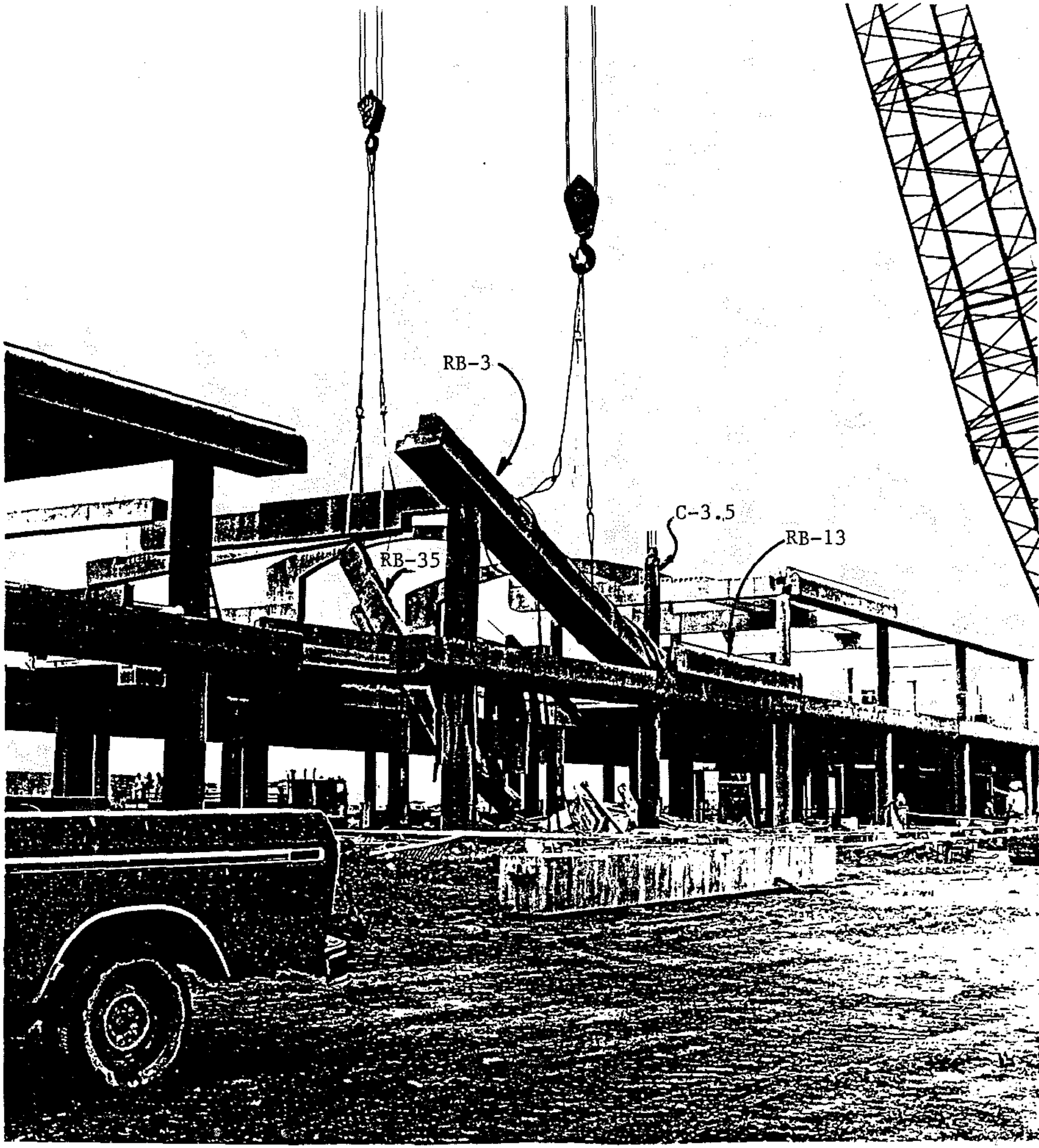
**** THE PRECAST ROOF PLANK ERECTED IMMEDIATELY PRIOR TO THE COLLAPSE

Figure 3.09

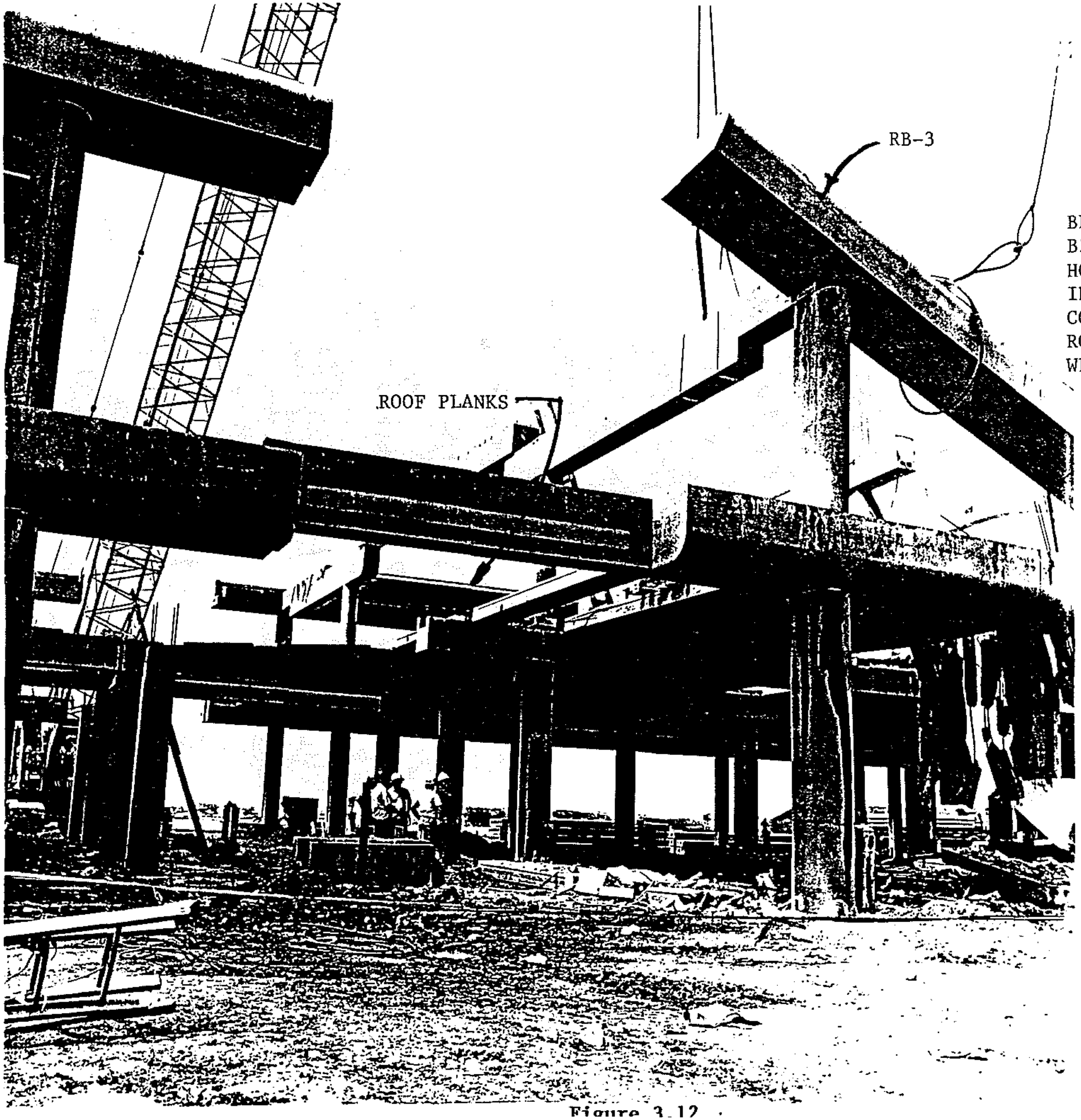


SCENE OF THE ACCIDENT SITE, LOOKING TOWARD SOUTH.

Figure 3.10

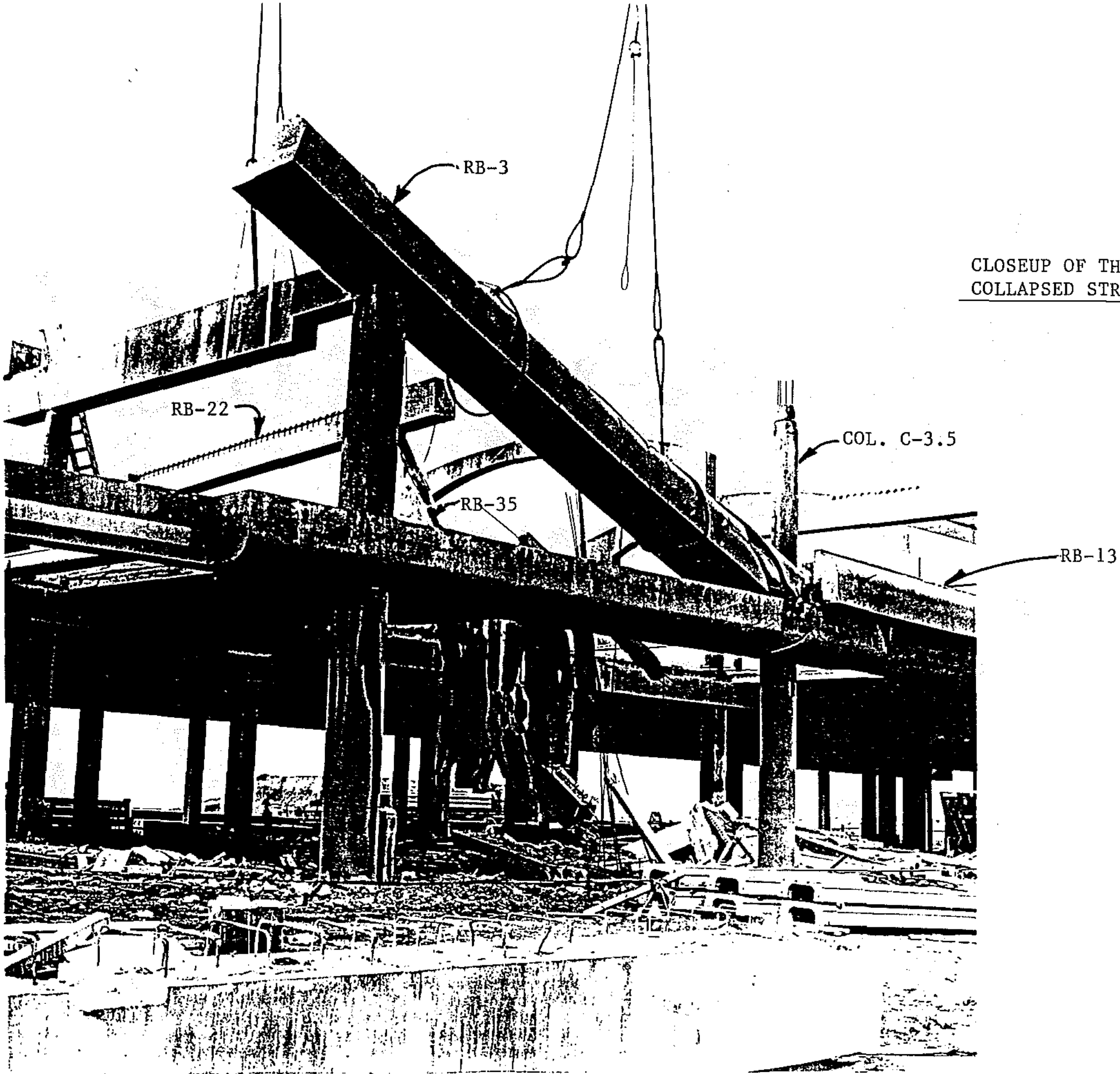


OVERALL VIEW-LOOKING TOWARD WEST



BETWEEN COL. LINES B20-B22, SHOWING HOLLOW PLANKS WERE INSTALLED BETWEEM COL. B20-B21 AT THE ROOF LEVEL OF THE WEST EXTERIOR BAY.

Figure 3.12



CLOSEUP OF THE
COLLAPSED STRUCTURE

LOOKING TOWARD THE COLLAPSED
RB-35 AT THE COL. LINE B20

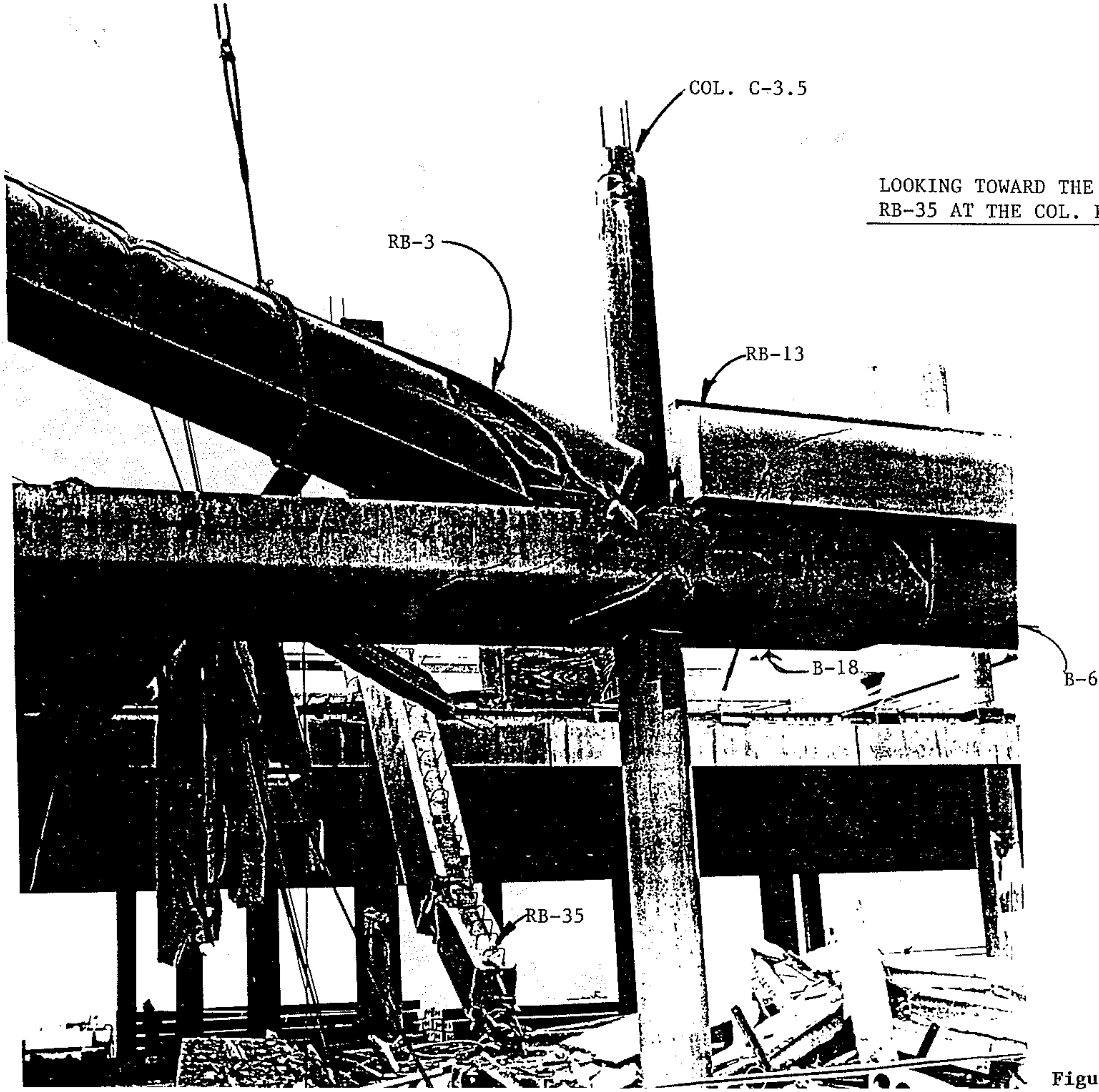
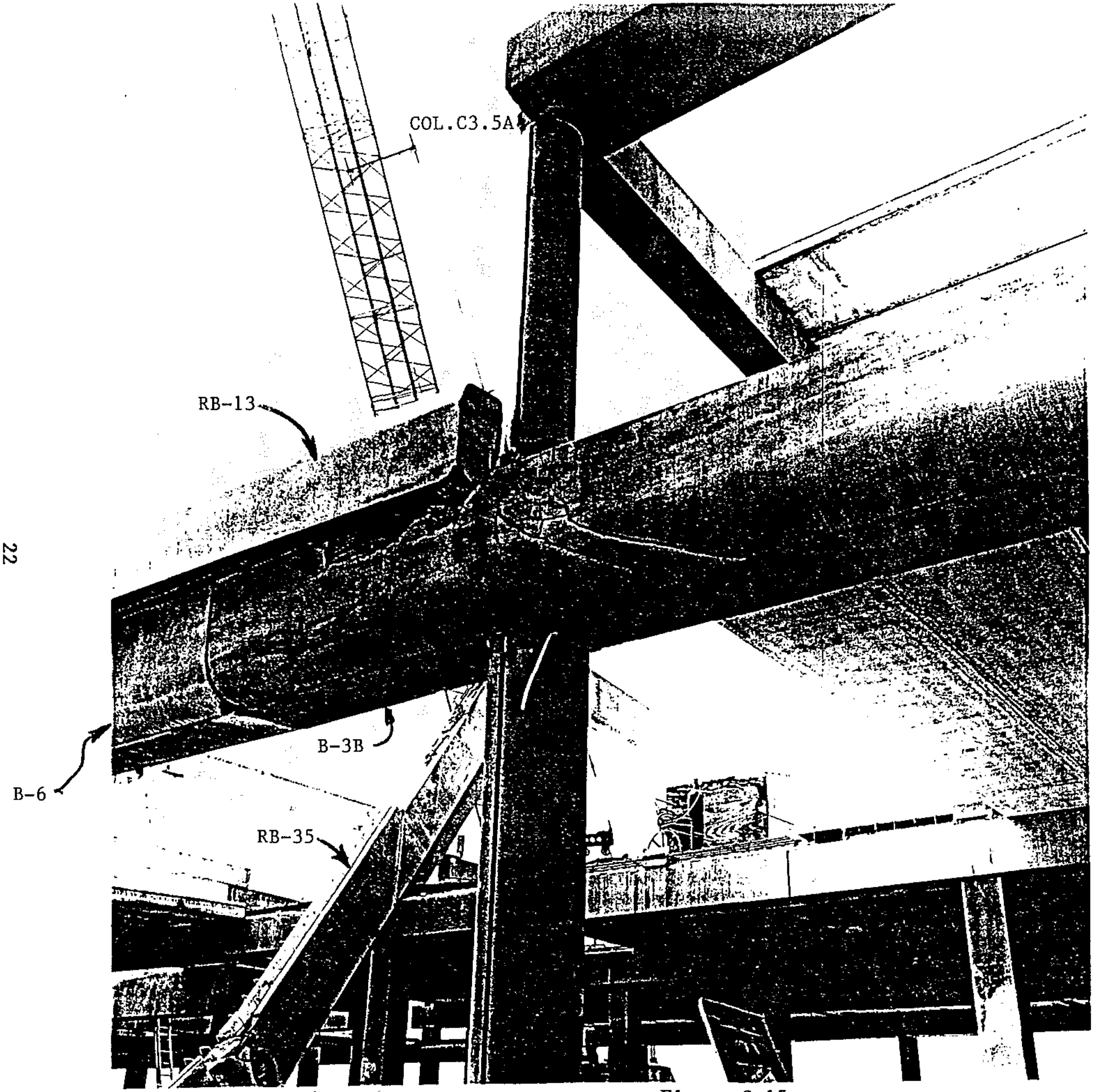


Figure 3.14



CLOSEUP AT THE COLUMN LINE B-19

Figure 3.15

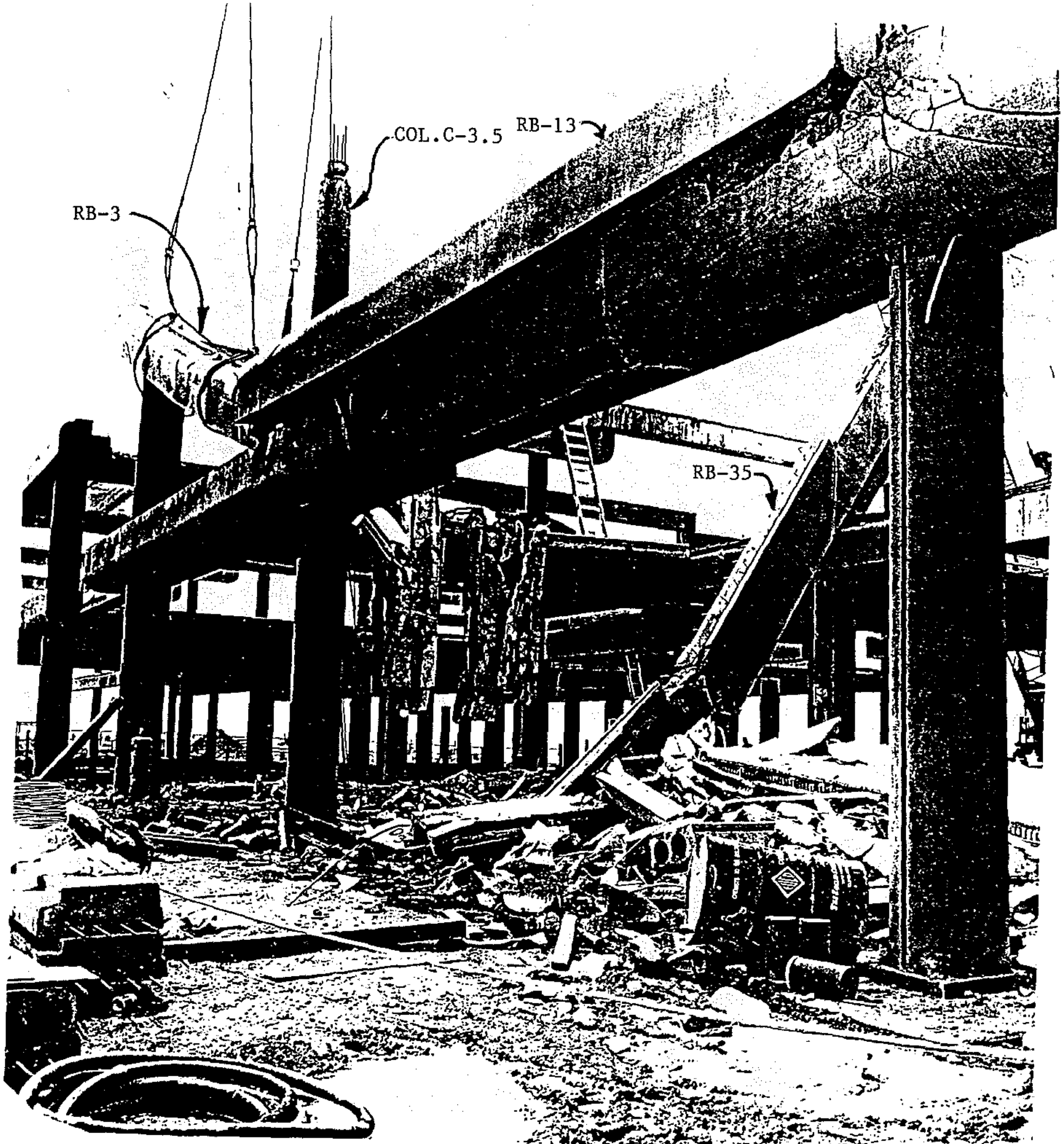
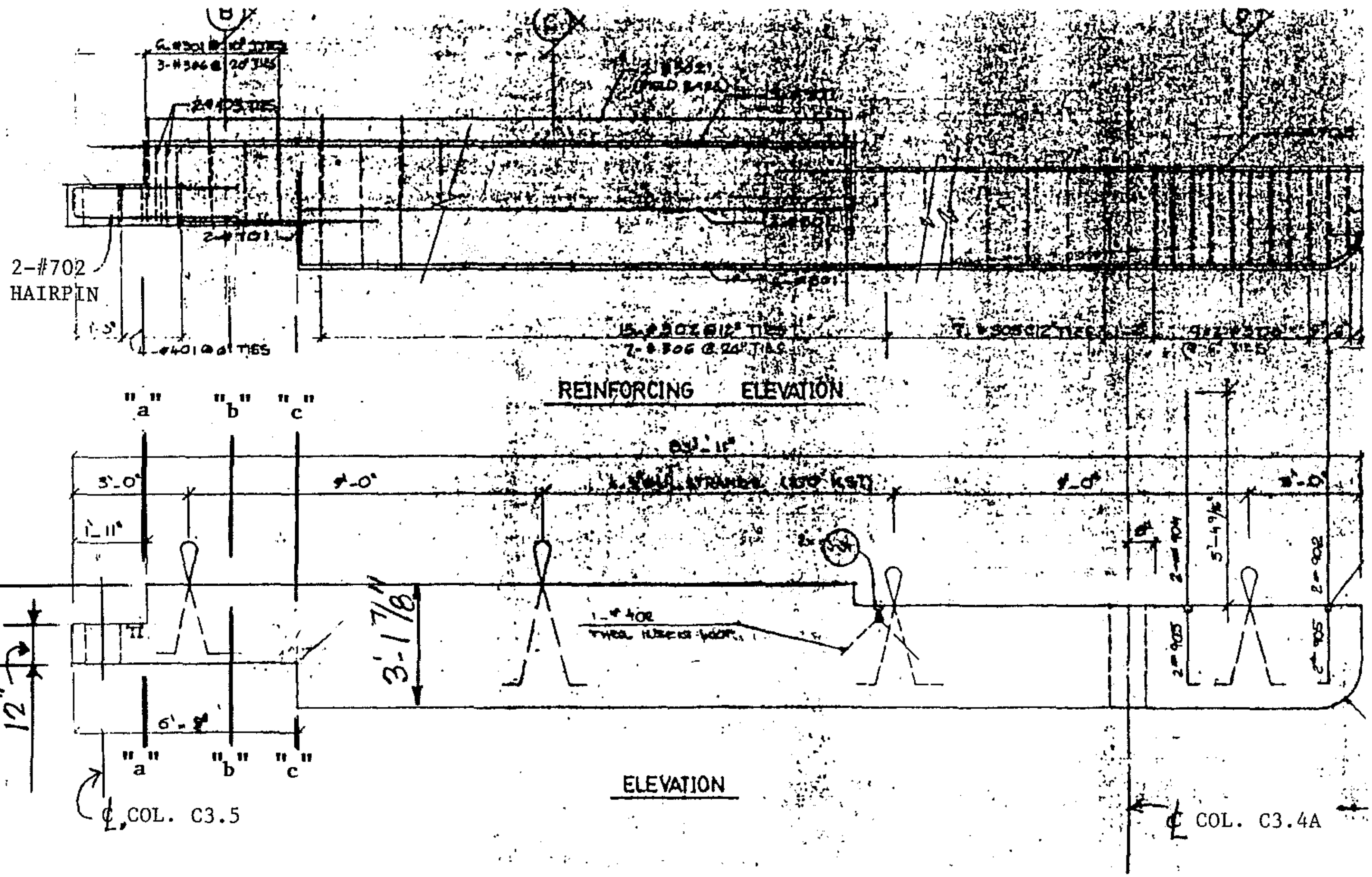


Figure 3.16

LOOKING FROM COLUMN
LINE B-19 TOWARD SOUTH

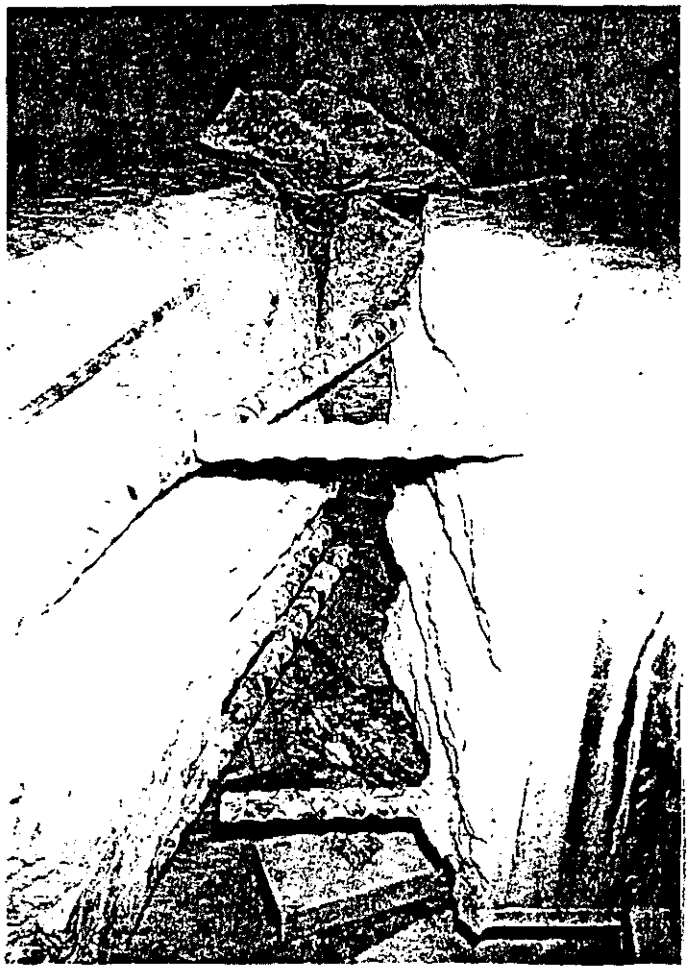
Figure 2.17

POSITION OF BEAM RB-35
AND COLUMN C-3.4A
AFTER THE COLLAPSE
(LOOKING TOWARD SOUTH)



BEAM MARK RB-35

Figure 3.18



ONE FAILED LOCATION OF BEAM RB-35 (AT 6'-0"FROM THE CENTER OF COLUMN C3. SHOWING THE END OF THE 2-#701 BOTTOM BARS.

SAME LOCATION , DIFFERENT VIEW.

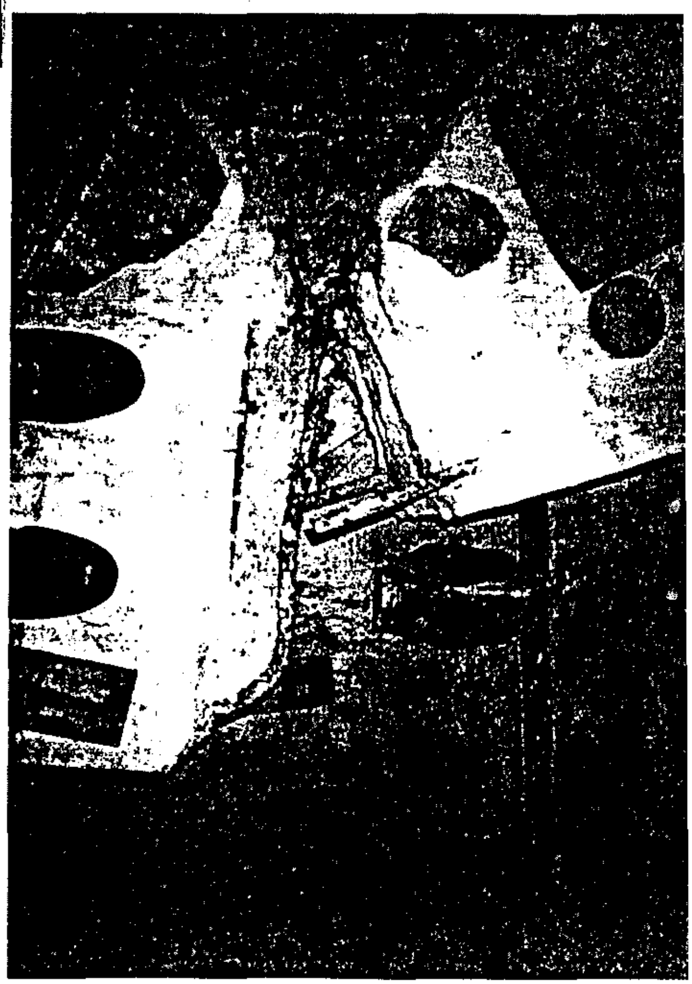
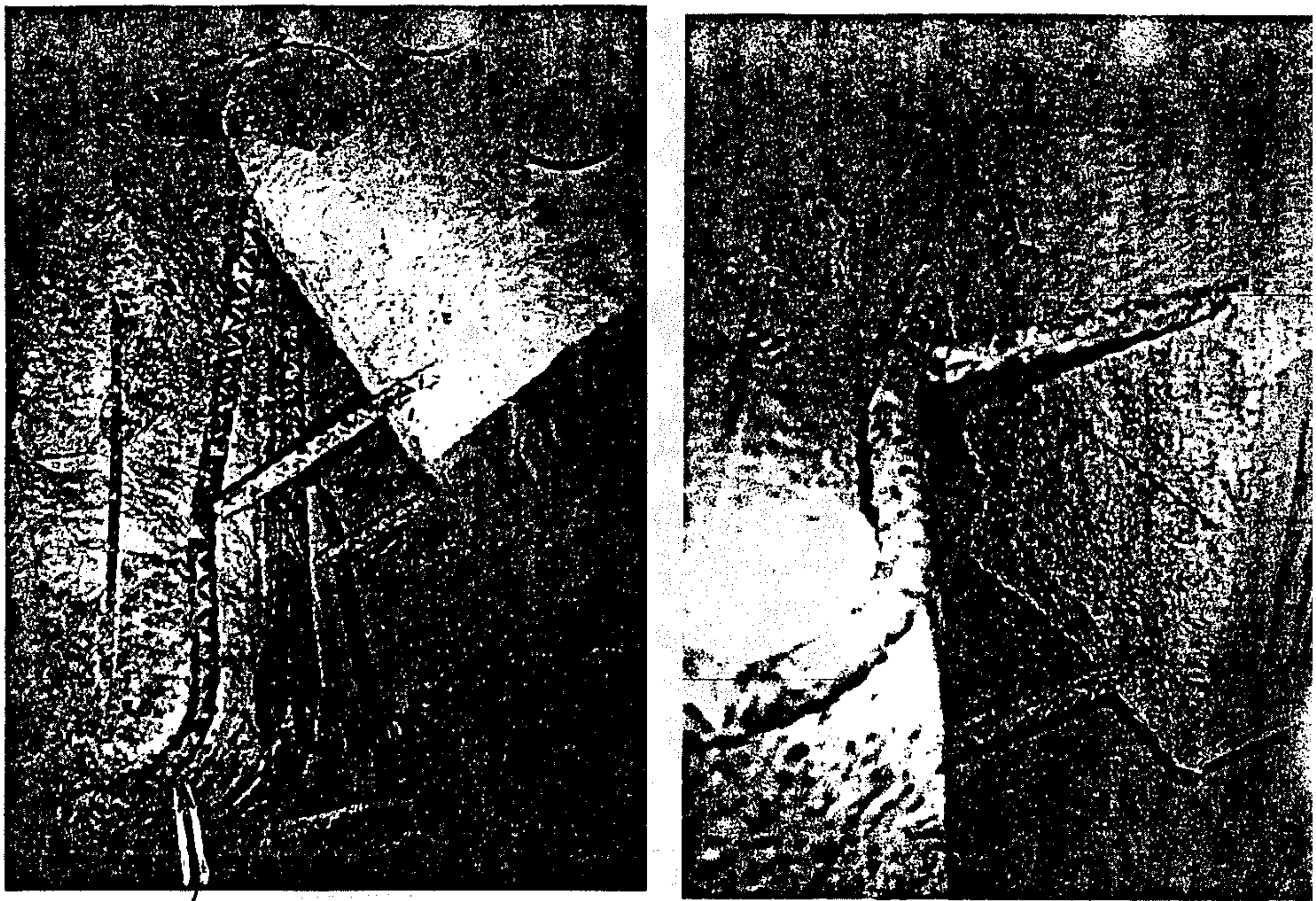
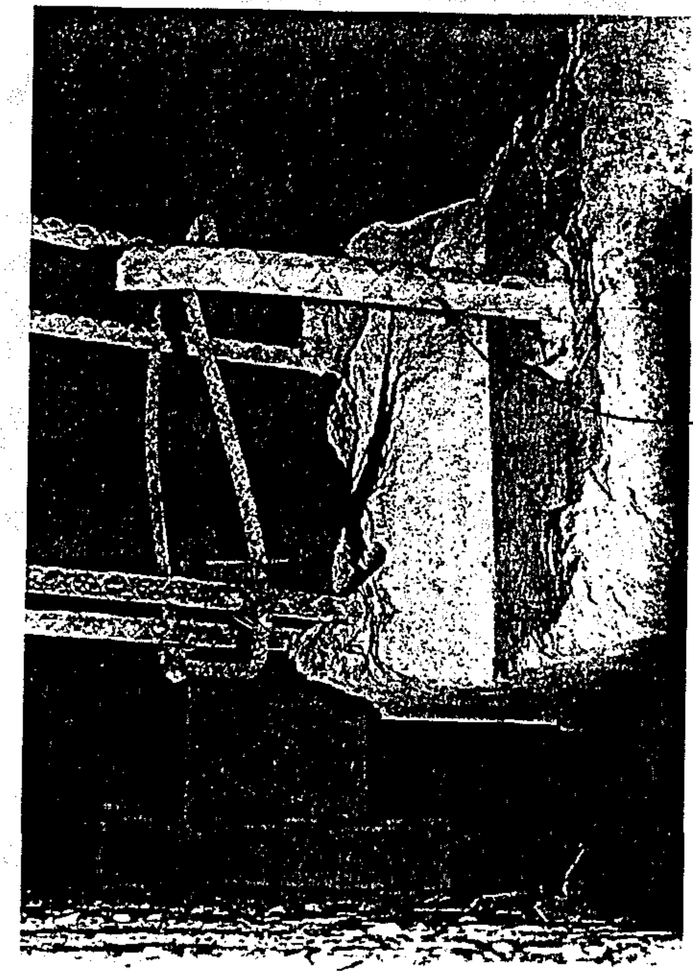


Figure 3.19



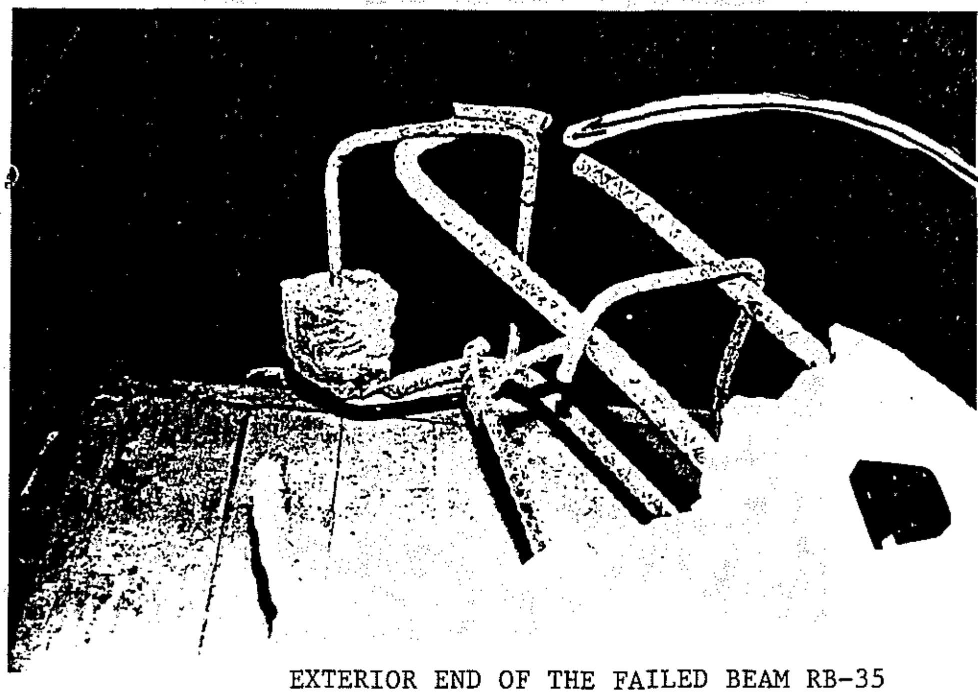
CLOSEUP VIEW OF THE 2-#701 REBARS AT THE FAILED LOCATION,
ALSO SHOWING THE 4-#801 BEAM BOTTOM REBARS BENT UP AT DEPTH CHANGE.

Figure 3.2.0



THE CENTER #9 TOP
BAR EXTENED 12" FROM
THE CONCRETE FACE.
TWO OTHER #9 BARS
WERE FLAME CUT AT THE
FACE.

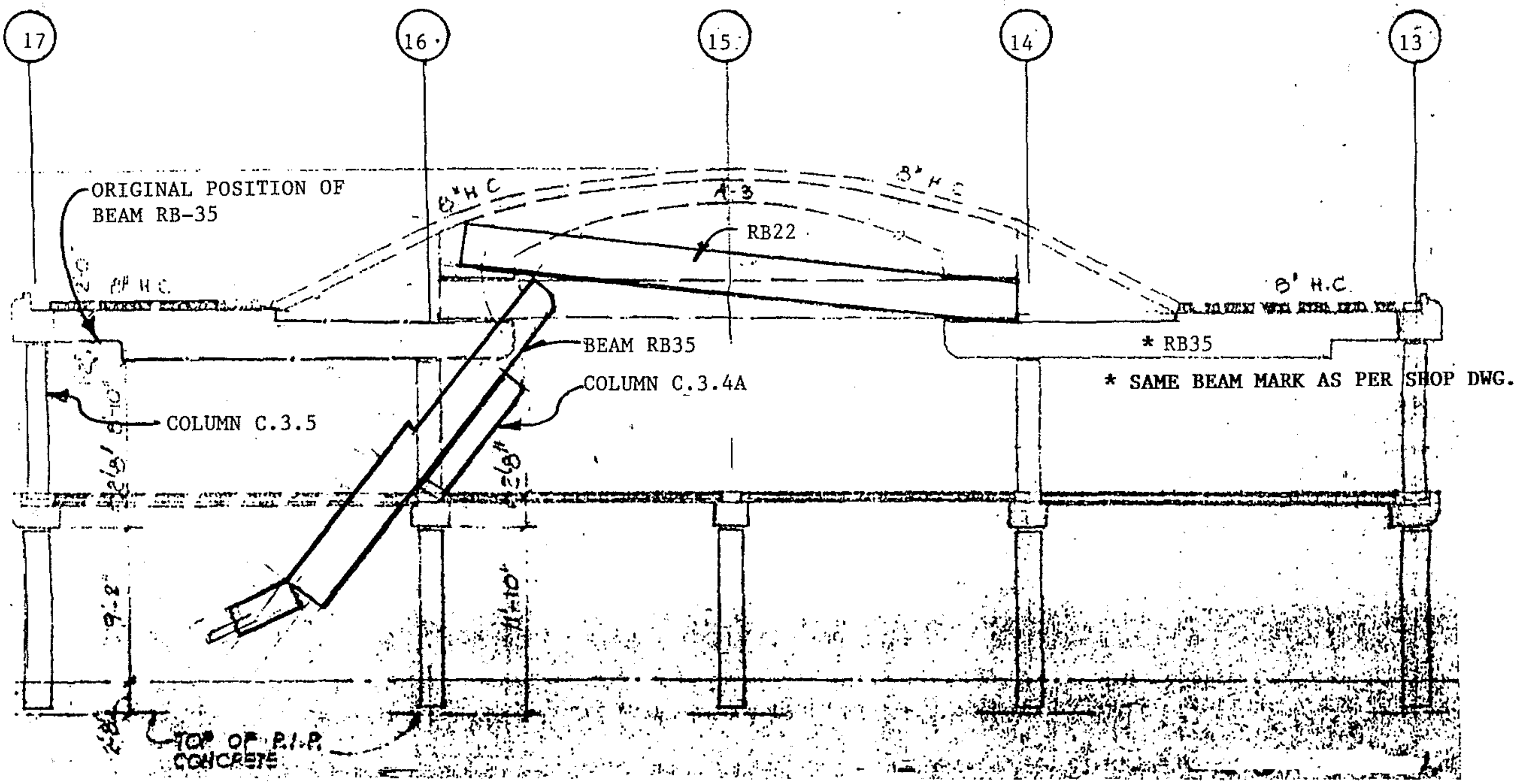
OTHER FAILED LOCATION OF BEAM RB-35,
AT THE DEPTH CHANGE FROM 12" TO 1'-11 7/8".
CONCRETE CRUSHED DURING THE COLLAPSE.



EXTERIOR END O
RB35,SHOWING
2#702 HAIRPIN-
BARS FRACTURE
NEAR THE EXTER
BENT CORNERS.

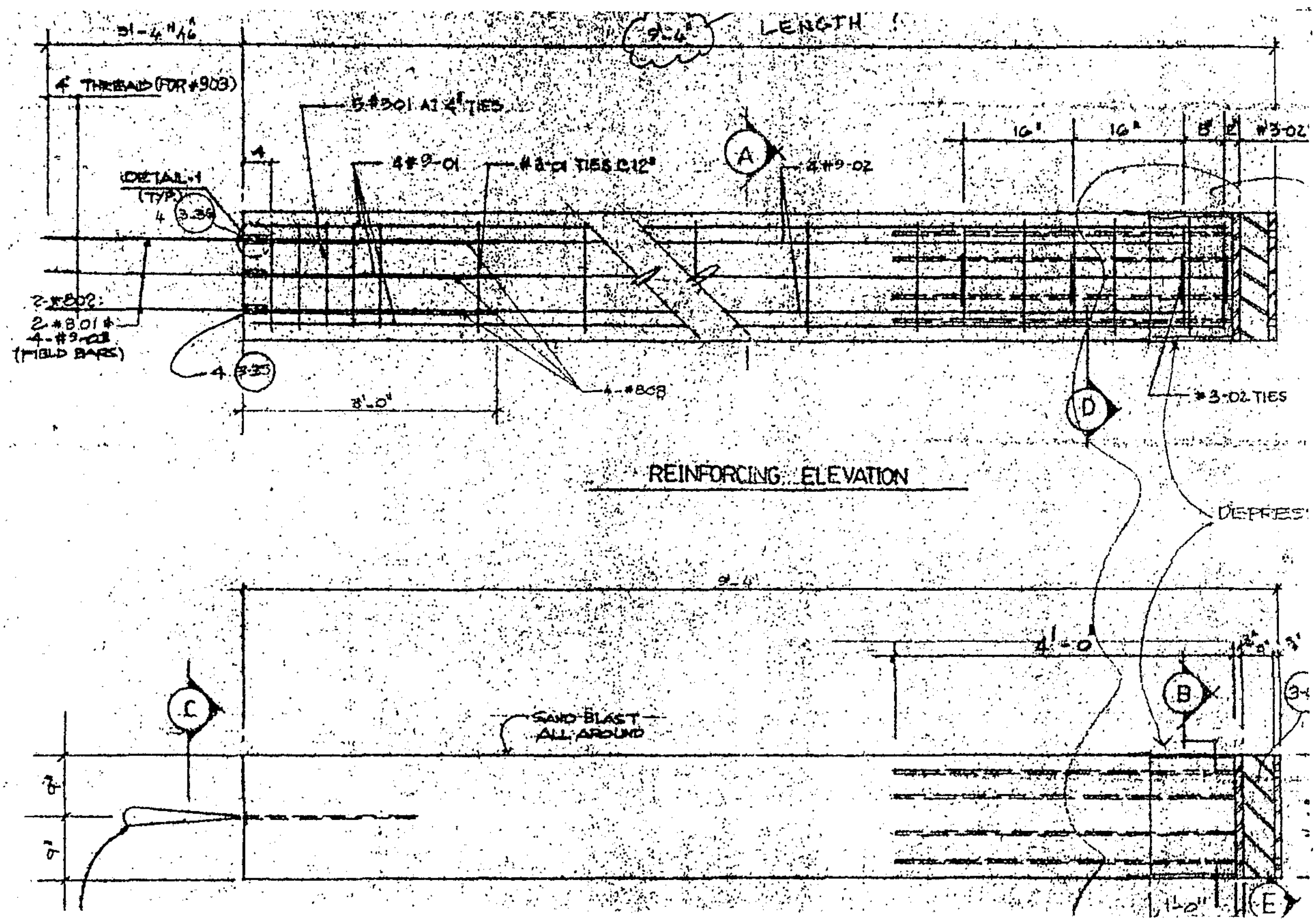
EXTERIOR END OF THE FAILED BEAM RB-35

Figure 3.21



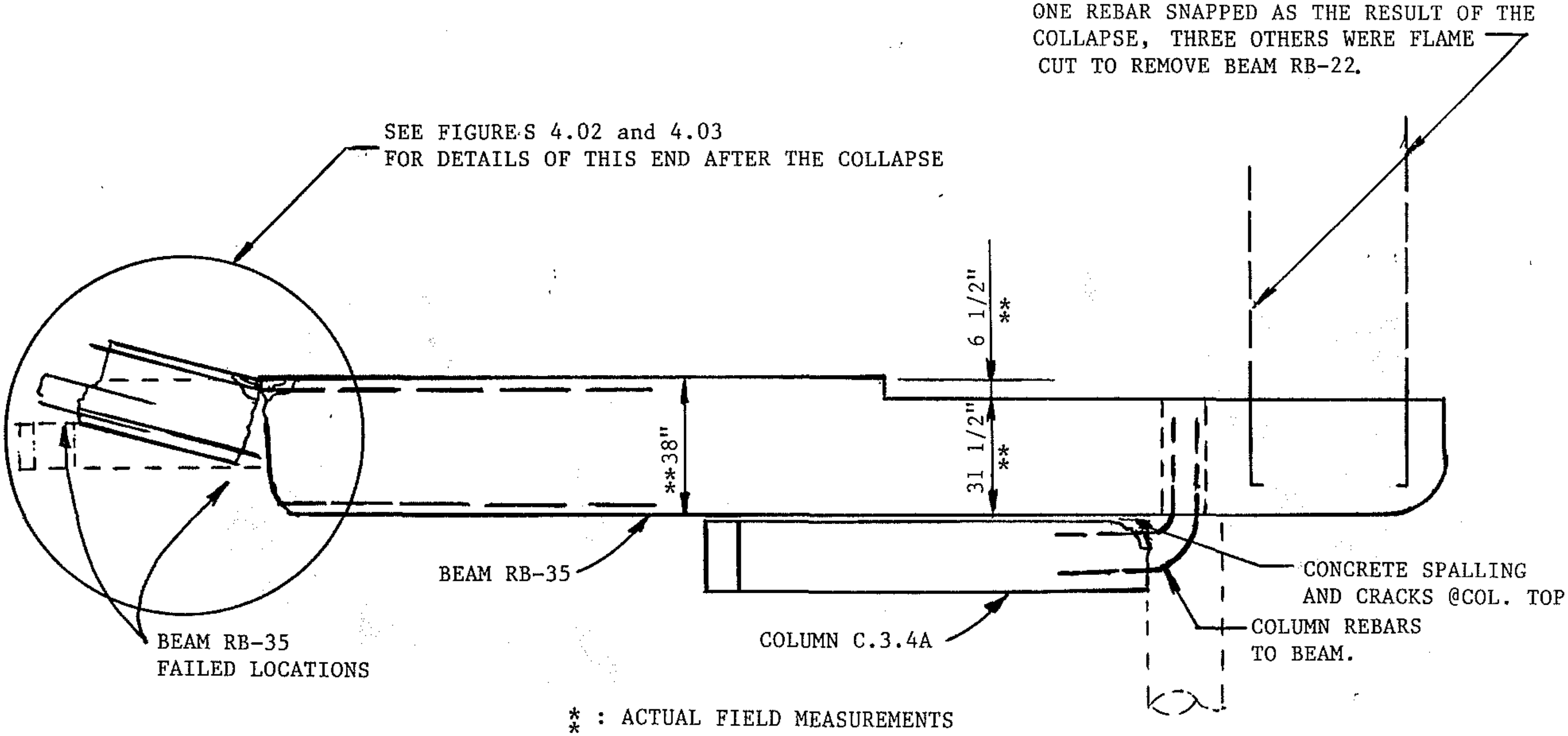
CROSS SECTION AT COLUMN LINE B20 AFTER THE COLLAPSE
LOOKING SOUTH

Figure 3.22



COLUMN MARK C.3.4A

Figure 3.23



CONFIGURATION OF ROOF BEAM RB-35 AND COLUMN C.3.4A AFTER THE COLLAPSE

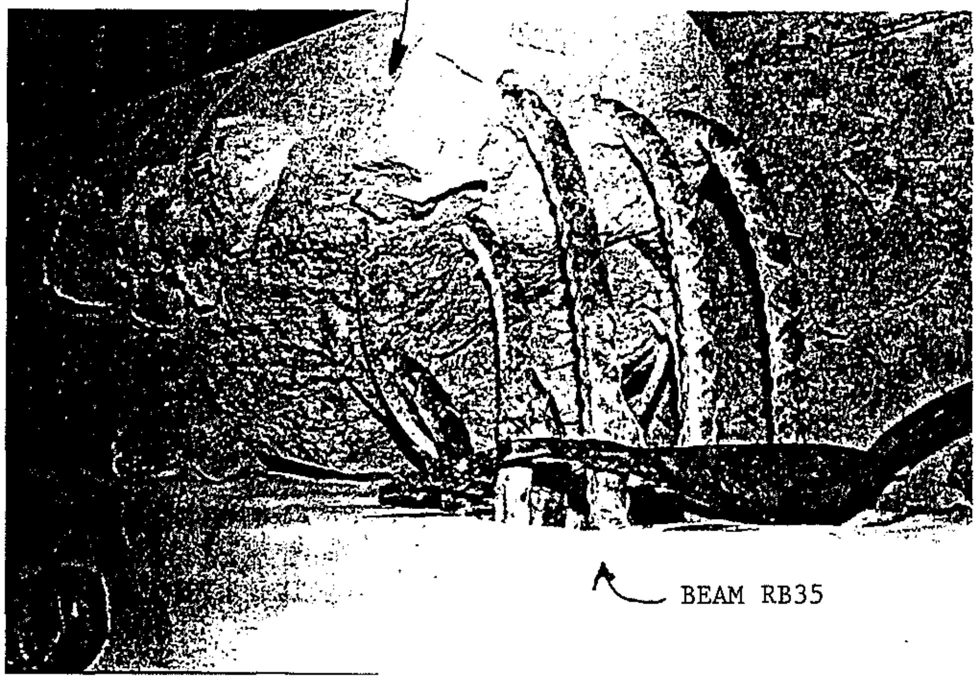
Figure 3.24

BEAM RB35

COLUMN C.3.4

COLUMN REINFORCING BARS BENT 90°

COLUMN C.3.4A



BEAM RB35

THE FAILED BEAM RB35 AND COLUMN C.3.4A

Figure 3.25

4.0 INTERVIEW STATEMENTS OF WORKERS AND ENGINEERS, AND FIELD OBSERVATION

Interview Statements

During the course of this investigation, construction workers in the vicinity of the accident site, employees of the quality control company, personnel of the precast concrete manufacturer, the designer of the precast elements and the structural engineer of record were interviewed by the OSHA team. The purpose of the interviews was to determine the sequence and mode of the collapse and to obtain general information regarding design, review, manufacturing and erection of the precast members. Twenty-one interviews were conducted with the employees associated with the field operations at the construction site who were in the general area of the accident site at the time of the collapse. Five interviews were held with personnel of the quality control company and the fabricator of the precast elements. Two separate interviews were conducted with the precast element designer and structural engineer of record. The highlights of the interviews are as follows.

On the day of the accident, the placement of the roof precast planks was underway. Two planks had been placed between column lines B19 and B20 bounded by column lines 16 and 17 prior to the accident. The precast plank placed immediately before the collapse was plank marked #701A, as identified in figure 3.09. Three planks between column lines B20, B21, 16 and 17 had also been installed and supported by the failed roof beam RB-35 at the time of the accident. Description of the failure is given in section 3.0.

The witness statements of the employees in the vicinity of the accident indicate that the failure was preceded by a loud noise described as a "cracking noise" or a "snapping sound". None of the witnesses could indicate the actual sequence of failure of the precast elements. It could not be ascertained with any degree of accuracy which precast member failed first. The witnesses did not observe any distress in any of the precast members prior to the failure. However, statements from three construction workers, including the ironworker responsible for unloading and hoisting the roof planks, indicated that the collapse occurred immediately following the release of the choker attached to the third plank spanning between roof beams RB-14 and RB-35.

One significant point of interest to this investigation was the placement of the reinforcing bars in the precast beams in accordance with the approved drawings. Five employees were interviewed regarding the placement of the rebars. The quality control technician, who was responsible for verifying the rebar placement (Witness #14), indicated that the usual practice was to inspect the reinforcing bars "for quantity and dimensions" before the beams were poured. The rebars were inspected to ensure that they were in the general area location shown on the approved plans, unless specific dimensions and locations were given for the bars in which case they were so placed. He believed that the location of the two bars marked #701 for beam RB-35 was not specifically dimensioned on the approved drawings and, therefore, these rebars were placed in the general area. As per his statement, the same was true for the bars marked #501 for beam RB-35. Employees of the precast fabricator also stated that the 2-#701 rebars were installed in the general area of where it was shown on the drawing due to lack of specific dimensions indicating the location of the bar. Interview statements from the designer of the precast elements and structural engineer of record, however, indicated that in their judgement there was adequate information regarding the location of the rebars. The precast element designer and the structural engineer of record believed, as per their statements, that the location of the 2-#701 rebars were clearly shown on the approved shop drawings. The precast designer also stated that there were no inquires made by the quality control personnel or the precast manufacturer during the fabrication of the beam.

Observation of the Collapsed Structure

Figures 3.10, 3.11, 3.16 and 3.17 show the general view of the collapsed structure. Subsequent to the collapse, the failed precast beam RB-35 and column C.3.4A were removed and stored at the Trans World Airline (TWA) warehouse in the airport complex, the remaining damaged precast beams and columns were stored at the job site. Engineers from the OSHA Office of Construction and Engineering, Washington D.C., made three visits to examine the damaged members. Critical dimensions relating to the fabrication of beam RB-35 were obtained. The following is the brief summary of the observations and measurements.

Beam RB-35

An examination of the collapsed beam marked RB-35 revealed a number of inconsistencies with the approved shop drawing. Figures 3.18 and 4.01 show the elevations and sections of the precast beam, as approved for the project to be manufactured in the precast concrete plant. Figures 3.24, 3.25, 4.02 and 4.03 show the partial elevation and sections of the damaged RB-35, based on actual field examination. The following observations were made.

- The width and depths of the beam conformed to the specified dimensions of the drawing.
- There were two vertical planes of failure, as is shown in the elevation in figure 3.24.
- The 1'-6" exterior end section of the beam was nonexistent. It is believed to have been crushed during the collapse.
- The center to center spacings of the four #8 bent bars marked #801 were 2 1/2", 4" and 6 1/2". See section A-A on figure 4.03.

- The concrete cover from the center of longitudinal bars #801 to the north and south face was 2 1/2".

- Two #7 bars marked #701 were 6'-0 long.

- From the vertical plane of failure where the depth of the beam changed from 24" to 38", two rebars marked #701 were protruding 8" on the north face and 8 1/4" on south face toward the deeper section. See figure 4.02 and 4.03.

- From the other vertical plane of failure, where beam depth changed from 12" to 24" these two #701 bars were protruding approximately 3" on both faces toward the shallow end. See figure 4.02.

- The two hairpin bars marked #702 were fractured at near the exterior bent corners of the bar as shown in figures 3.21, 4.02 and 4.03.

- The three #9 top bars marked #901 were observed to be flame cut, one at a distance of 12" from the plane where the beam depth changes, and two at the face of the plane itself as shown in figures 3.21, 4.02 and 4.03. These rebars were reported by the construction personnel to be cut in order to facilitate the placement of spandrel beams marked RB-3 and RB-13. These two beams were framed into and supported by the beam RB-35. Field observations were made to the beam connection on top of the column at the opposite side of this grid line (at the intersection of column lines 13 and B20) and the cutting of the similar top rebars of the roof beam, also marked RB-35 was noted.

Column C3.4A

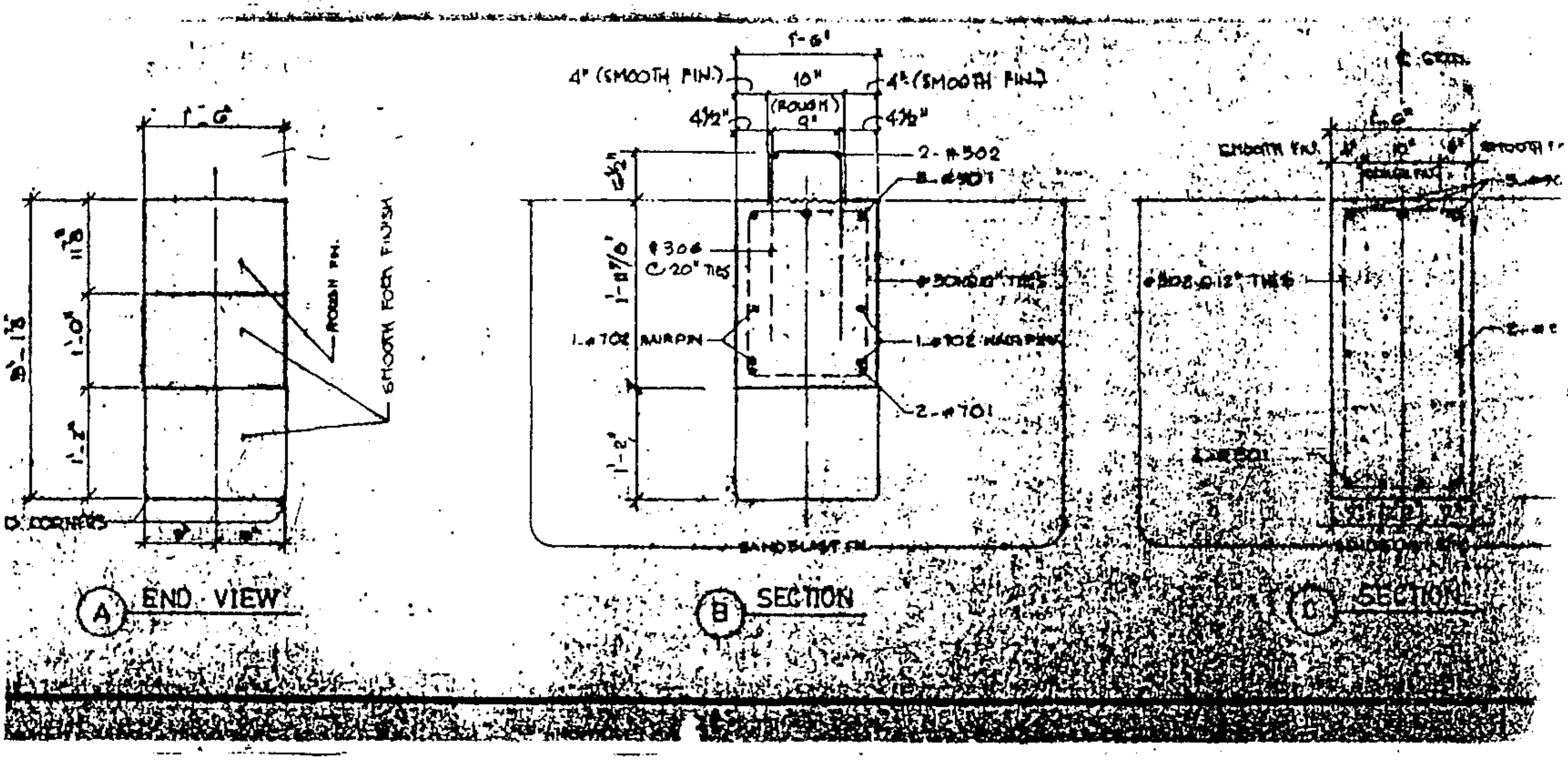
Column C3.4A collapsed with the precast beam RB-35, with its top end remained attached to the beam. Figures 3.24 and 3.25 show the attachment of the column top end to the beam following the collapse. All top eight bars were bent 90° and the column longitudinal axis was parallel to that of the beam. None of the bars were missing. There was spalling of concrete near the top of column on the side closest to the precast beam. Figure 4.04 shows the bottom base plate of the column. Two holes in the plate indicate slight deformations around the hole circumference.

Column C3.5

Figures 4.05 and 4.06 are copies of the approved shop drawings showing the top and bottom ends of the precast column C3.5. Figure 4.07 is a photograph taken after the accident. All eight #9 top bars extending above the beam bearing elevation were present. No deformations were noticed in those bars. The 11" x 11" grout had some minor spalling and some rebar did not seem to have desired cover. At beam RB-35 bearing, concrete had spalled. The four holes in the bottom base plate showed deformation around their circumference.

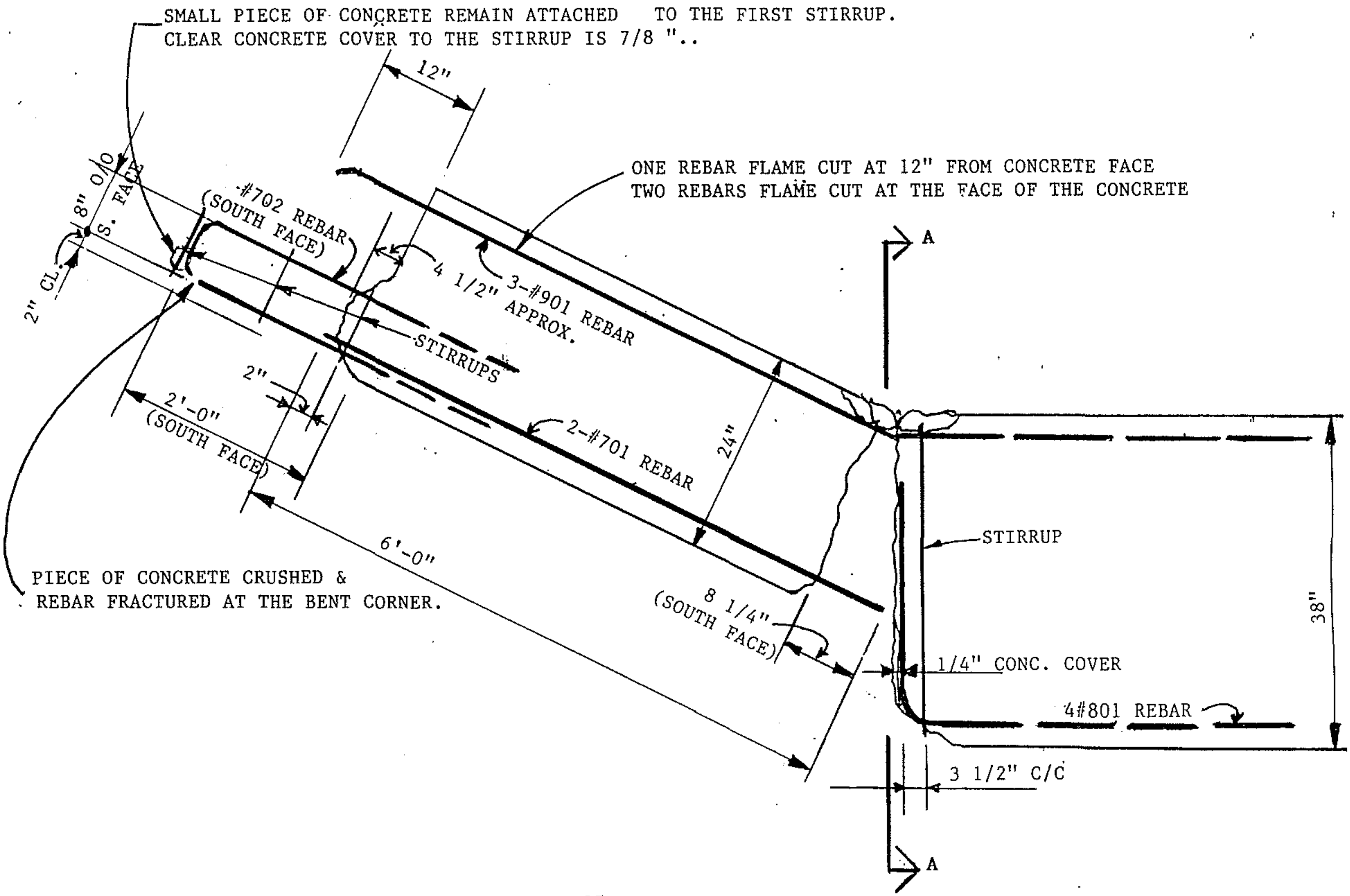
Beam RB-22, RB-3, RB-13

Both bearing ends of beams RB-3 and RB-13 had extensive damage as the results of the failure as shown in figures 4.08 and 4.09. Observation was not made of beam RB-22. However, a description of the failure is given in section 3.0.



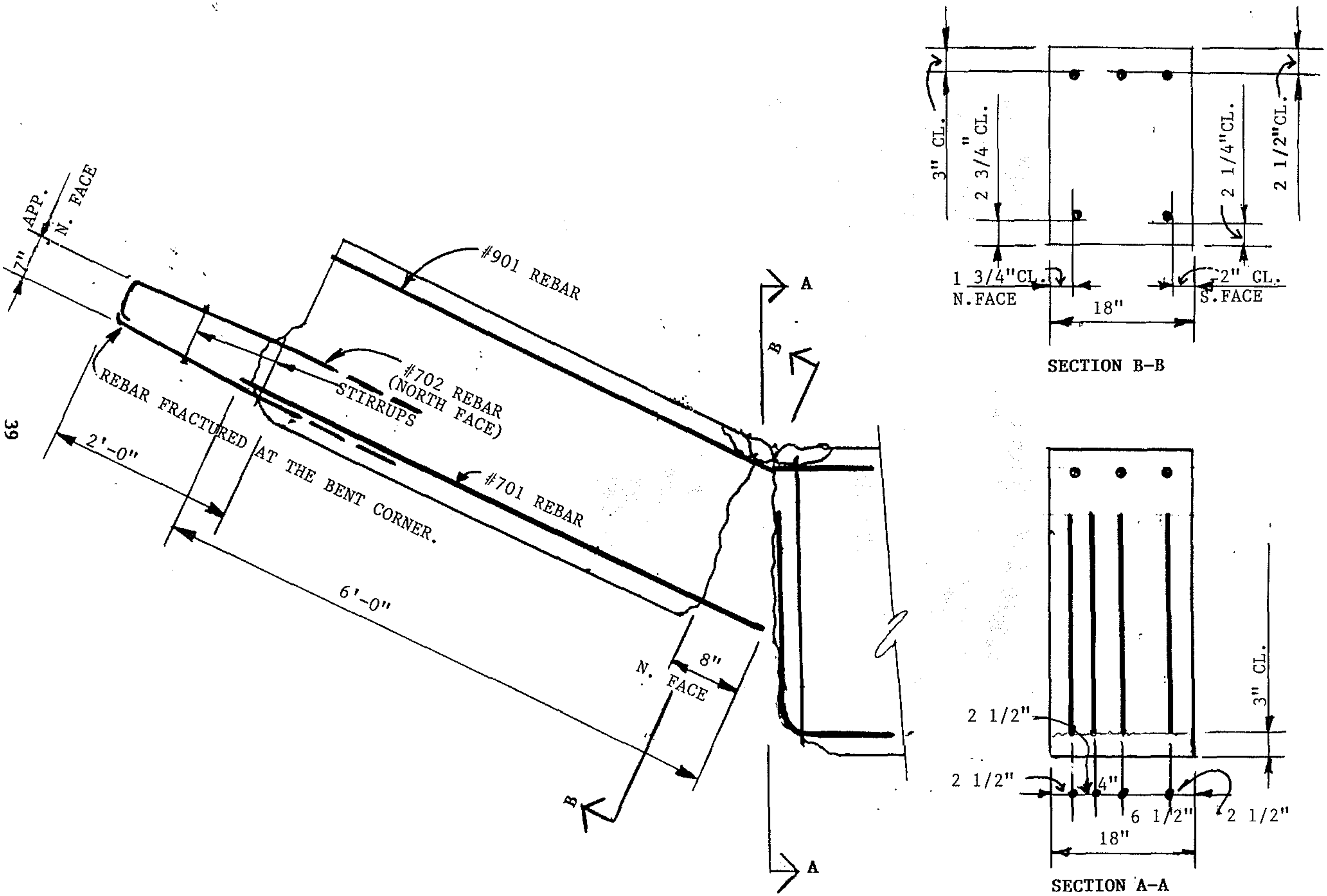
BEAM RB-35

Figure 4.01



EXTERIOR END OF BEAM RB-35 AFTER THE COLLAPSE
1 OF 2

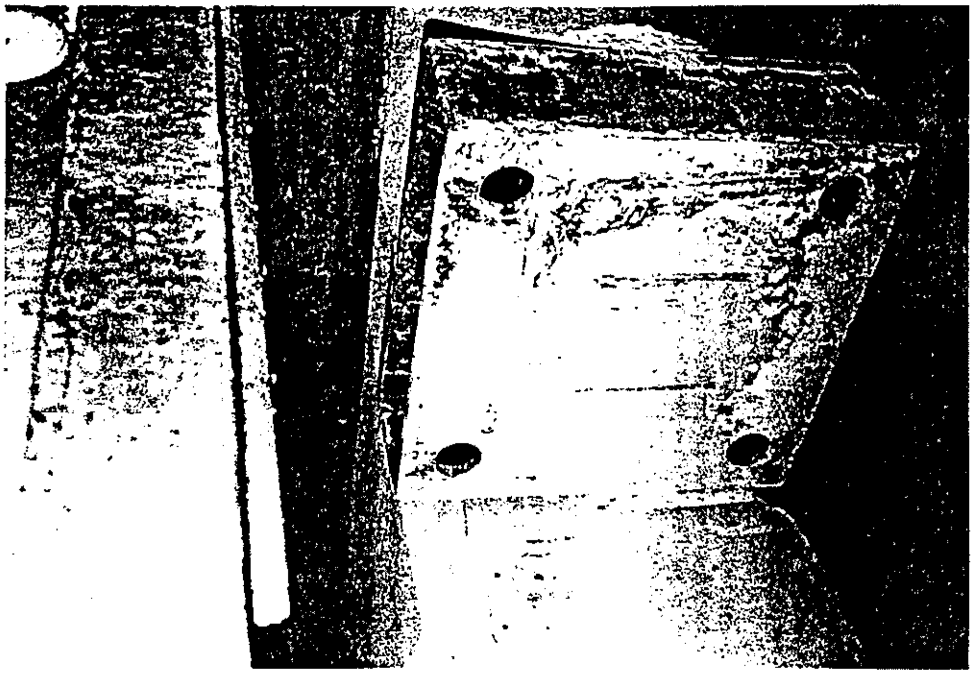
Figure 4.02



EXTERIOR END OF THE BEAM RB-35 AFTER THE COLLAPSE

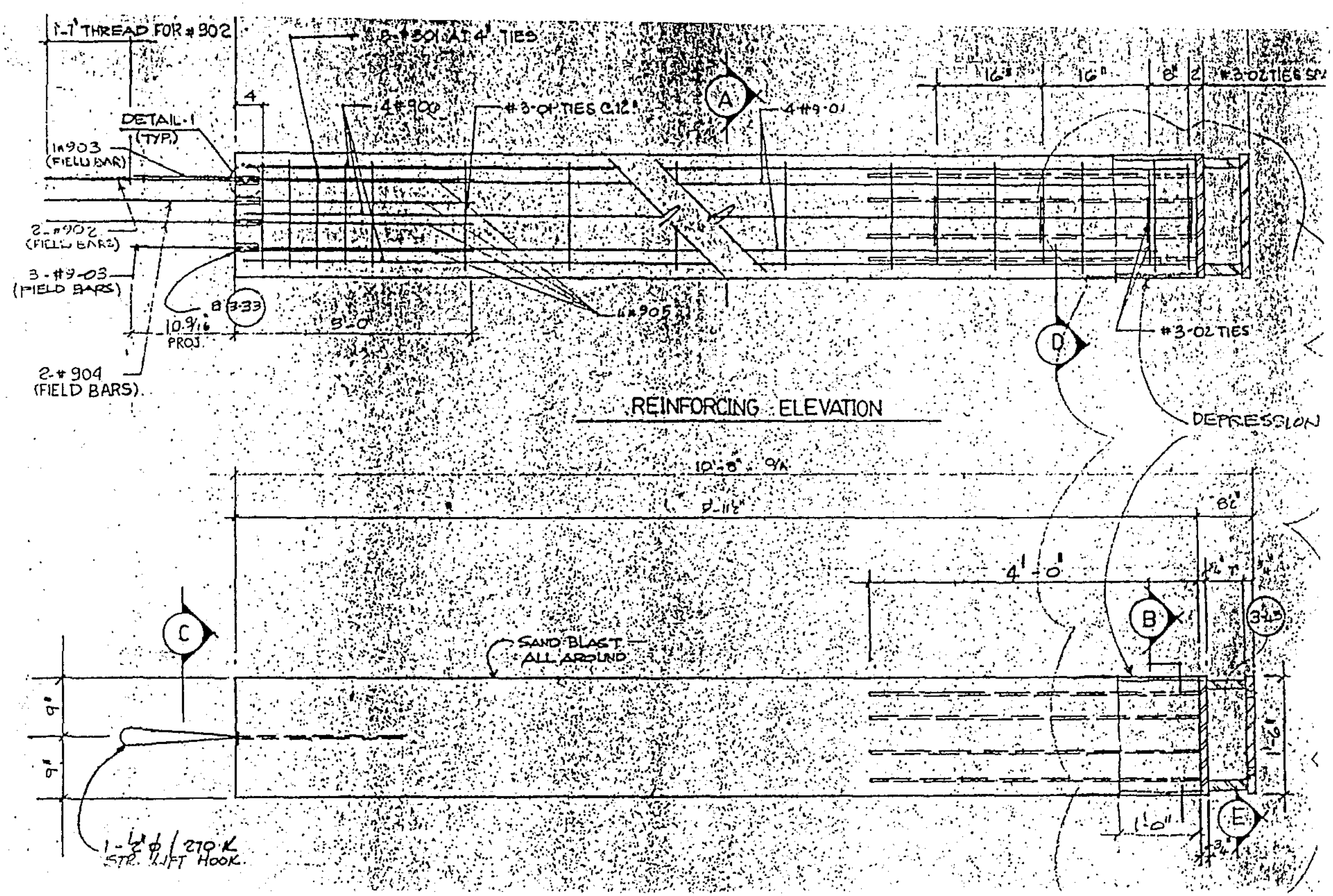
2 OF 2

Figure 4.03



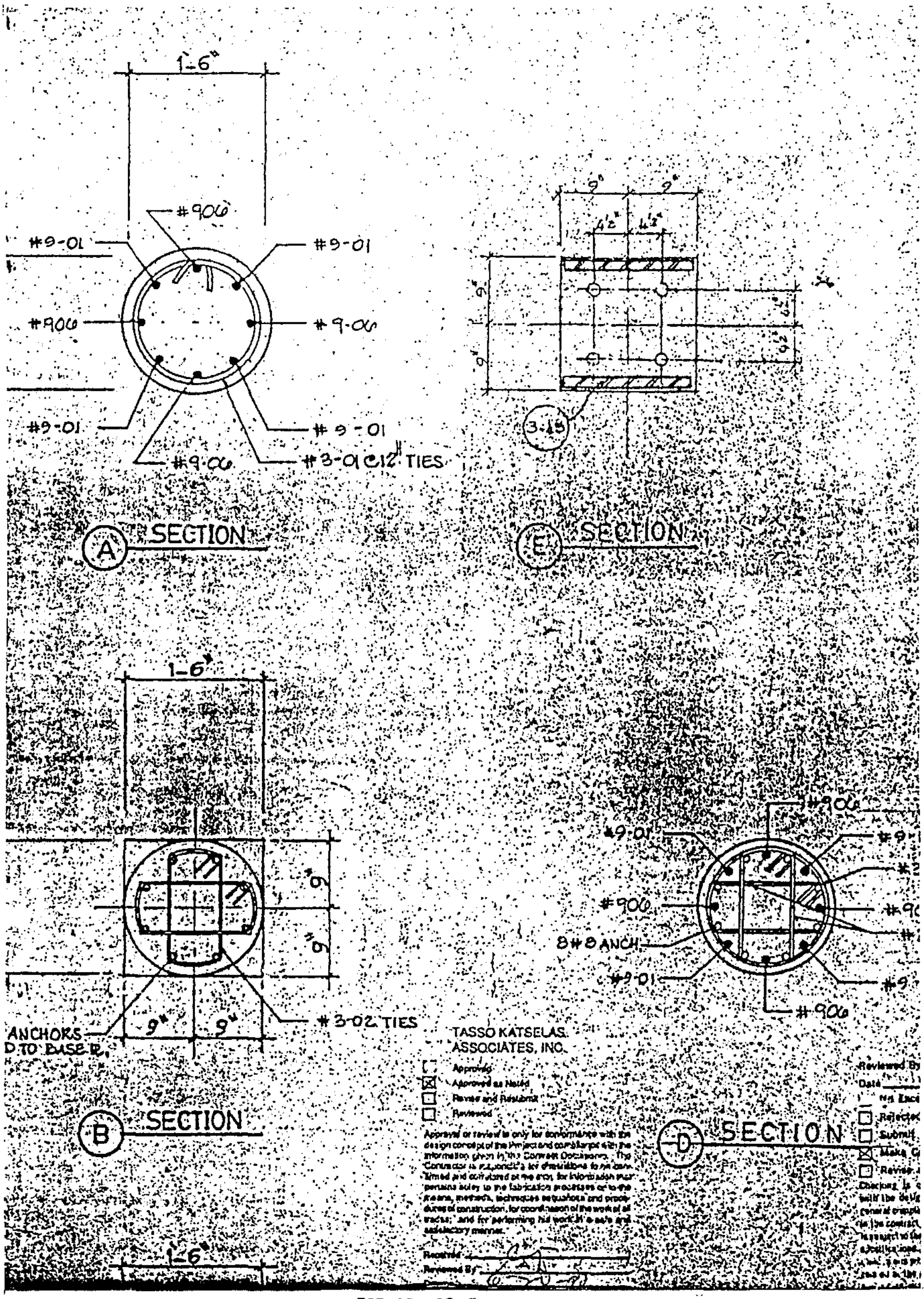
BASE PLATE OF THE FAILED COLUMN C.3.4A

Figure 4.04



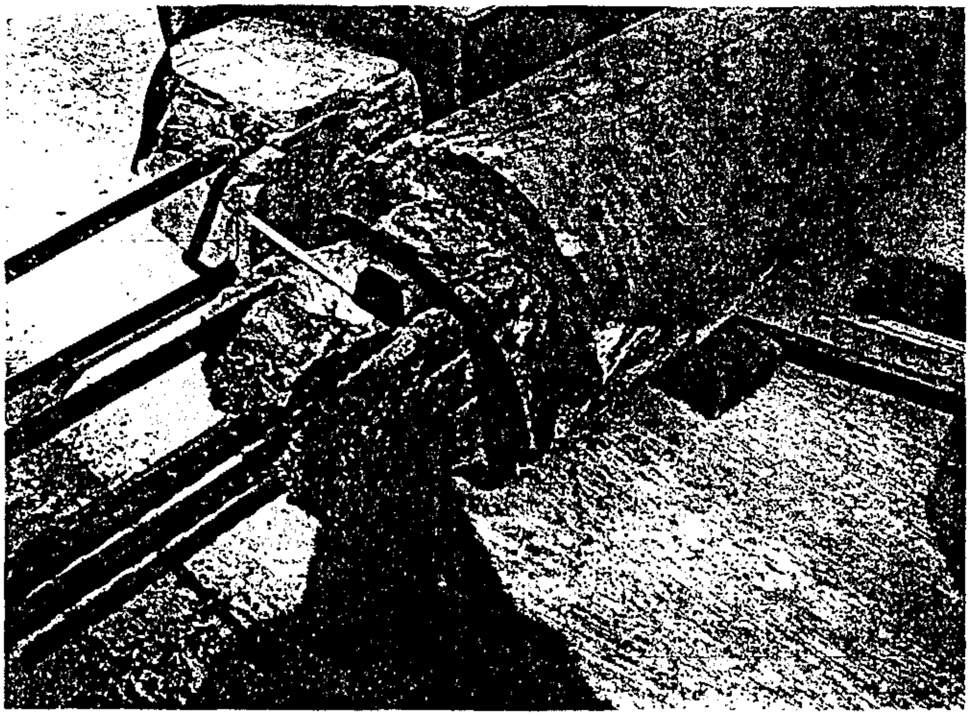
COLUMN MARK C.3.5

Figure 4.05

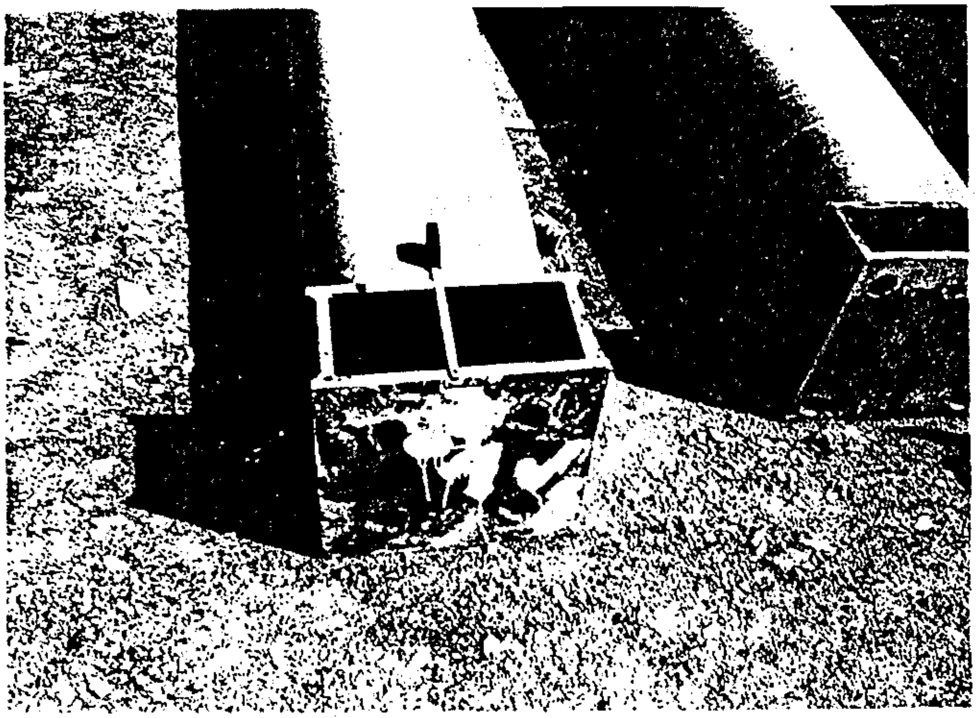


COLUMN C3.5

Figure 4.06



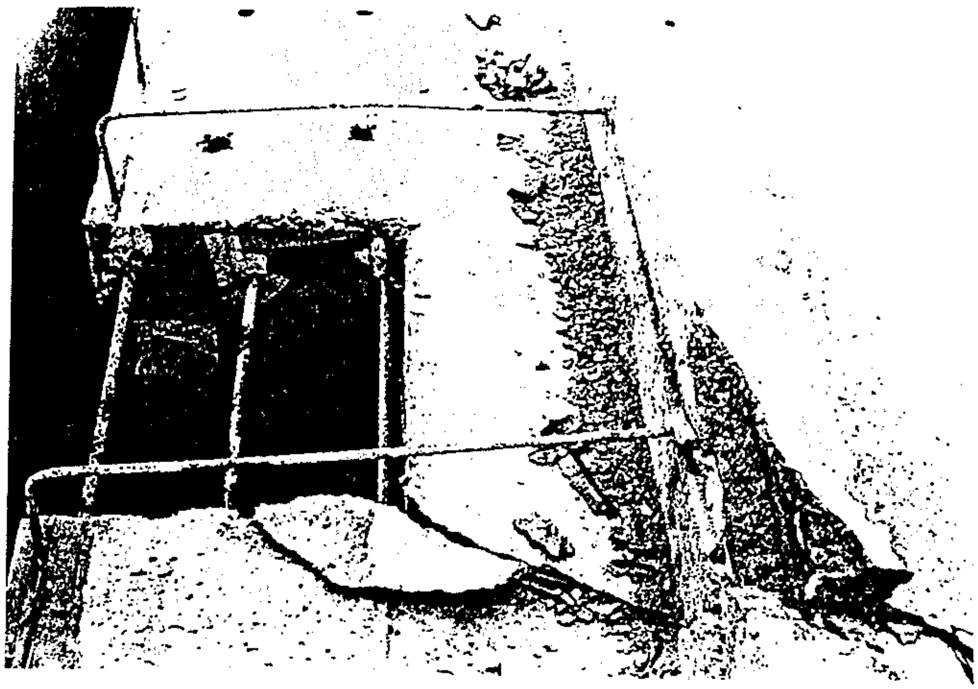
TOP OF COLUMN C.3.5



BOTTOM OF COLUMNC.3.5

Figure 4.07

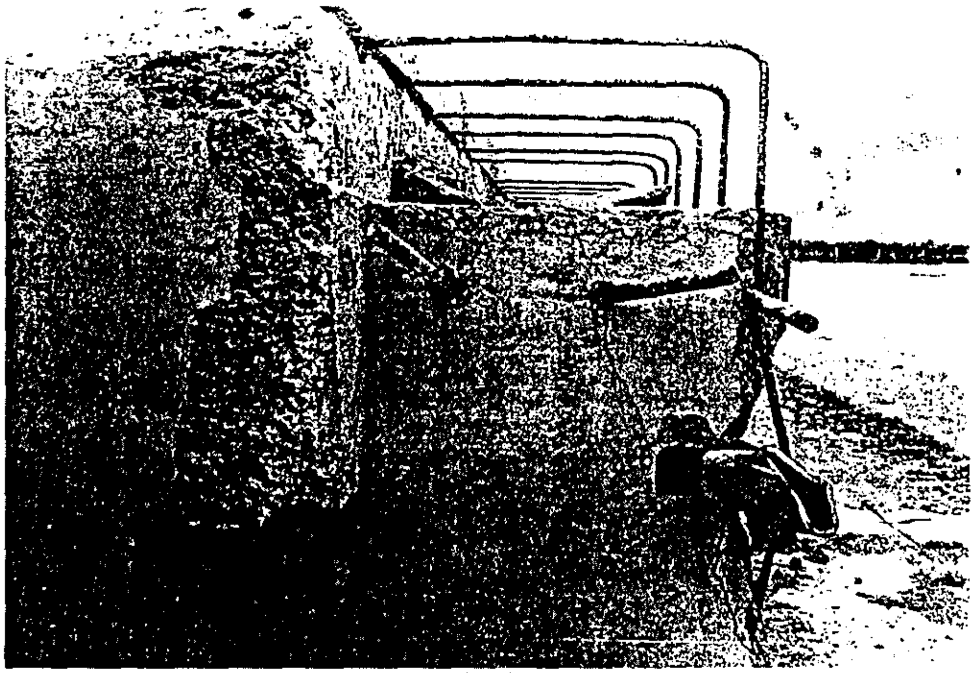
NORTH BEARING END



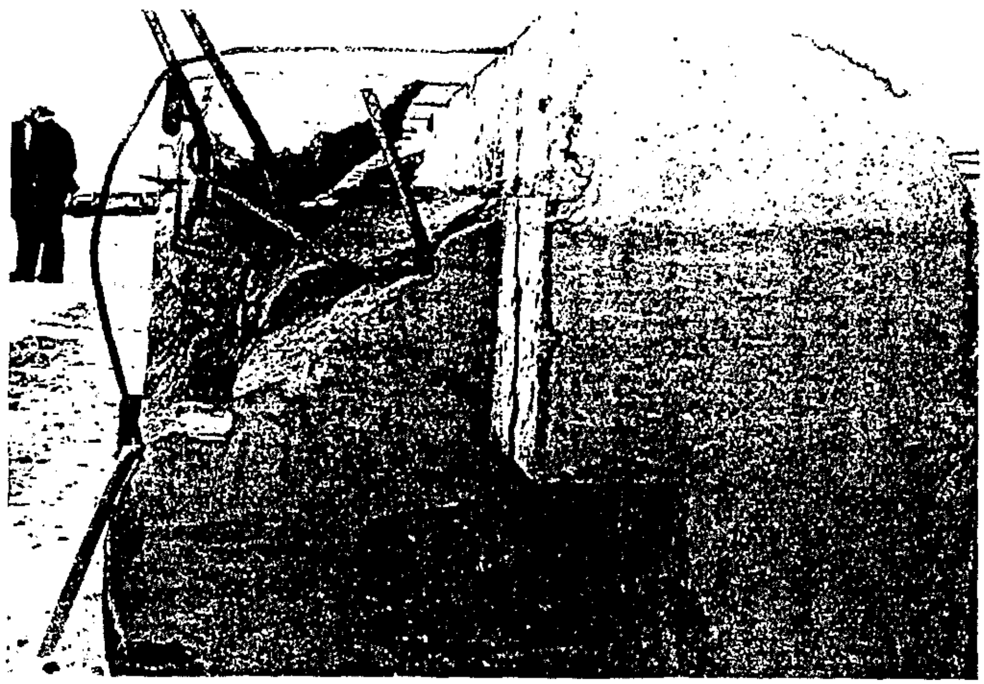
SOUTH BEARING END AT COLUMN B21 & 17

BEAM RB-3 AT THE CONSTRUCTION SITE

Figure 4.08



NORTH BEARING END



SOUTH BEARING END

BEAM RB13 AT THE CONSTRUCTION SITE

Figure 4.09

SUMMARY OF INTERVIEW HIGHLIGHTS

Eyewitness No.	Location of the Eyewitness at the Time of the Accident	Highlights of Comments
#1 - Journeyman Ironworker	Near the Ringer (*) on the concourse level. (*) Ringer - refer to Ringer Crane.	o Heard a snap; o Turned and saw beam close to him was down. o Saw one worker come down with the plank and landed on the top of the plank. o Saw another worker hanging from a hook. o Went further away from Ringer, looked up and saw the third worker - dead.
#2 - Journeyman Ironworker	On the roof plank near the Ringer crane.	o Worked with other worker, landed plank. o Took the chokers out from plank. o Building shook or shifted like an earthquake. o Still holding onto his choker, knew something was wrong. o In a matter of seconds everything caved in.
#3 - Inspector	West side of the building, toward the Ringer on concourse level.	o Was trying to pick up the rebar. o Looked up and saw the building starting to come down. o Saw one worker fall and other grab onto the cable on the crane. o Beams fell basically the way they were sitting.

#4- Journeyman Ironworker - Foreman	On concourse level, right underneath the roof plank that was being set.	o Heard a cracking noise. o Worker above him in the process of landing plank on the roof level. o Was showing the deceased what was to be done. o Heard a noise, ran out the other way.
#5 - Journeyman Ironworker	On the ground by the tool trailer, just came down off the ladder.	o Heard a loud crash. o Turned around, saw one worker hanging on the hook. o Saw top floor collapsed and 2nd floor with it and landed on ground.
#6 - Journeyman Ironworker	On the side of the Ringer (right across from the accident scene).	o Saw men releasing the choker. o Saw one worker holding onto to the choker and the other worker was coming down with the plank. o Was on the other side did not feel or hear anything.
#7	On ground floor underneath the concourse plank that collapsed.	o Heard sound like big tri-axle dump noise - screeching, crumbling sound. o Turned his head, saw the planks coming down. o Ran out from underneath. o Saw the deceased up on the second floor. o Saw another man pinned under. o Saw other worker was riding down with the hook.

#8 -Ironworker Apprentice	On top of the arch one bay away from the accident. (20 feet away).	o Noticed a plank coming overhead, stepped out of the way and let it go by. o Climbing to top of the arch from roof beam. o Saw two workers receiving the plank. o He was grouting and his back was to the accident scene. o Heard concrete smashing through. o Seemed everything coming down like dominos.
#9 -Ironworker Foreman	Between column B19 & B20 on line 15 facing west.	o Heard a crack. o Looked around and saw RB35 falling and planks following. o Running for the edge of the building.
#10 - Ironworker	On the truck.	o Unloading the third plank from the truck. o Saw connectors cut loose the plank. o In full view, saw beam RB35 collapse. o Saw one worker trying to catch the choker but fell to the ground. o Other worker caught the choker, swinging in the air.
#11 - Ironworker Foreman	On the side away from the Ringer crane.	o Heard a snap. o Looked up and saw plank moving and then down. o One worker came down with the plank, the other was hanging on the hook. o Worker had cut loose of plank, and the plank was set. o About 20-25 feet away from the beam that came down.

#12 - Crane Operator		o Operating the crane that was setting the slab (plank). o Just landed a slab (plank). o Ironworker took the cable off. o Saw one worker reach and grab the choker. o The other worker missed the choker and fell with the concrete.
#13 - Laborer (For Precast Fabricator)	Not at the scene of the accident.	o Assigning workload to workers. o Answering questions or referring questions to others. o Assisting in laying out of cages. o 2- #701's of RB35 were installed by appearance on drawing due to lack of measurements. o Measurements on drawings are not thoroughly checked by him.
#14 - Q.C. Technician (For Inspection Company).	Not at the scene of the accident.	o Inspect rebar cage according to drawings for quantity and dimensions. o Inspection of rebar cage in forms for clearance and embedment etc. o Rebar measurements/locations were those specified on the drawings. o No specific location for the 2 -#701 bars, therefore, these rebars were inspected to a general area location and not to a specific measurement. o Was never given the location and measurements by the supervisor or anyone else concerning the 2 -#701 bars. o Same situation for 2 -#501 bars.

#15 - Foreman (For Precast Fabricator)	Not at the scene of the accident.	o In charge of basic outline of fabrication rebar cage measurements, placements, etc. o Coordinate workers, showed them how to set-up stands, rebars, ties, etc.
#16 - Precast element designer	Not at the scene at time of the accident.	o Prepared the design and shop drawing of the precast elements. o Submitted design to general contractor and structural engineer of the record for review. o Correct and re-submit design for final approval if required. o Had prepared one page erection procedure for the sequence of erection. o No inquiry was made to him about the location of the 2-#701 rebars. o Believed the development length of the rebar was dimensioned. o Believed the development length of all other RB35 were correct. o Believed the quality control engineer was at fault. o Believed the construction company did not follow the erection sequence. o Was not satisfied with the quality of the grout placement in the column. o Since the accident, re-evaluated the design and decided to reinforce the beam with additional rebars or steel plates to the sides of the beam.

#17 - Structural Engineer of the Record	Not at the scene at the time of the accident.	o Engineer of his company checked the precast elements design drawings, (not just review). o Only checked the design with the completed structure. Did not check for construction loads during erection stage, checked for service loads only. o Believed the development length of the rebar was noted on the drawing. o Did not question the location of the 2-#701 bars when reviewing the beam. o Believed the design of the beam was adequate if properly built. o Believed the lap length of bars #701 and #702 did not meet the ACI Code. o Re-evaluated the beam design and decided to reinforce the beam with additional rebars, or steel plates at sides of the beams.

5.0 LABORATORY TESTING

Soon after the collapse, a private testing laboratory was employed by the construction manager of the project to conduct the tests to aid the investigation of the collapse. The purpose of the testing was to determine the strength of the concrete of the failed RB-35 beam. The laboratory was also requested to conduct field inspection of embedded reinforcing steel in precast concrete structural members. The field inspection was undertaken to verify the end locations of the critical steel reinforcements of all roof beams that had been erected prior to the accident and of those beams ready to be erected.

Eight four-inch diameter (4"ϕ) cores were taken at various locations of the failed beam RB-35. These locations are shown in figure B-1 of appendix B. Nine tests were performed for concrete compressive strength (two tests were made to Core #1). Eight of the test results showed that the concrete compressive strength are higher than the specified value of 5000 psi. One core tested showed the compressive strength of 4980 psi. The average compressive strength of these nine cores are 5427 psi. Three cores were also subjected for splitting tensile strength which had a mean value of 512 psi. The report with the test results is attached in appendix B.

The field inspection consisted of examining precast beams, columns, and arches by using "R" and "DR" meters to determine the presence and/or locations of reinforcing bars, their cut-off points, and the depth of the concrete cover. A review, of the preliminary reports of the field investigation, had concluded that there were a number of inconsistencies relating to the location of critical reinforcing steels. At certain locations of several beams, the development lengths and lap splice lengths did not meet the American Concrete Institute Building Code requirements, thus, questioning the structural integrity of the structural elements. These concerns were immediately brought to the attention of Allegheny County and the consultants involved by the Pittsburgh Area OSHA office. These consultants were requested to examine all the precast beams to verify their structural adequacy, in light of the discrepancies in the placement of rebars indicated in the field report of the private testing laboratory. Two meetings were held where discussions took place between the consultants, Allegheny County and OSHA officials requesting the consultants to undertake immediate steps to verify the structural integrity of all members and to take necessary corrective measures.

6.0 STRUCTURAL ANALYSIS

Structural analysis was conducted to determine the level of stress in various precast members involved in the collapse due to the loads imposed upon them immediately prior to the collapse and compare them to their actual available strengths. Reactions and internal forces at various locations were computed. The measured dimensions of the reinforcing steel and structural members were used to compute the available strengths. Provisions contained in American Concrete Institute publication #318-83 [1] were used to evaluate the available strengths.

Among all the collapsed precast elements, RB-35 was analyzed for various loading conditions. Determination of all the loads on the beam RB-35 at the time of collapse was done. There was a general consensus that there were only six precast roof planks placed on beam RB-35, between column line 16 and 17 at column line B20 prior to the collapse. Those planks were marked #701A, #616 and #701 between column line B19 and B20 and #615, #701A and #701 between column line B20 and B21 as noted on figure 3.09. The last plank to be placed was marked #701A closest to column line 17 between column line B19 and B20 and was detached from the crane. Besides the dead weight of the roof planks, beam RB-35 was also subjected to the reaction of the dead load of beam RB-22. The arch A-3 was not yet placed over beam RB-22. Further, the spandrel beam RB-13 transferred its dead load to beam RB-35 with an eccentricity of about 5" from the center of column C3.5. Thus, the beam was subject to additional load from RB-13 and moment due to the eccentricity. The reaction of spandrel beam RB-3 was coincident with the center of column C3.5. Therefore, no dead load from RB-3 was imposed on beam RB-35.

The point of application of the dead load reaction of beam RB-22 on beam RB-35 was considered as it was of significance due to its cantilever effect. The beam RB-22 bears at each end over RB-35 for a length of 5'-3". The calculated downward deflection at the edge of beam RB-35 was 0.013" due to is own dead weight, loads from spandrel beam and beam RB-22. The calculated deflection at the center of beam RB-22 was 0.06" also in the downward direction. Based on the above and on compatible nominal deflections, it was considered unlikely that the reaction from RB-22 could have occurred at the edge of beam RB-35. Therefore it was considered realistic to apply the reaction of beam RB-22 at the center of its bearing.

Five loading conditions were examined to compute the factored and unfactored bending moments and then to compare them with the available strength at critical sections along the length of the beam. The following were the five loading conditions. Load factors used for factored bending moment calculations were based on ACI 318-83.

Load Case 1: In this case, only the dead load of the beams RB-35, RB-22 and RB-13 were considered. Roof planks were not considered. This case was done to determine the stresses prior to any loading of the roof planks.

Load Case 2: In addition to the loads described in Case 1, the dead load of the precast roof planks placed prior to the collapse, was considered. This case is the actual loading condition immediately prior to the collapse.

Load Case 3: In addition to the loads described in Case 2, the reaction from the precast arch marked A1 was added. This case was considered to determine what the stresses would have been if the arch was placed over beam RB-22 prior to placement of the roof planks.

Load Case 4: In addition to the loads described in Case 3, dead load of the remaining precast planks scheduled to be placed on beam RB-35 was considered. Dead load of the roof planks which were to be placed along the sloped portion or on the arch or on RB-22 was not used.

Load Case 5: In addition to the loads described in Case 4, dead load of all the roof planks scheduled to be placed over the sloping member, arch and RB-22 were considered.

Out of the five load cases described above, load case #2 was the combination of loading which existed immediately prior to the collapse. Load case #1 was for the situation which existed prior to the placement of any roof plank. Load cases #3, #4 and #5 were for the situations that would have occurred if the failure had not taken place and the construction progressed.

A general purpose commercial computer program, STAAD-III [2], was used to analyze the plane frame at column line B-20. The support conditions of the beams and column were chosen to reflect the actual condition existing at the site before the collapse occurred. Beam RB-35 on column line B20 was considered continuous over the support at column C3.4A. The beam column joint at column C3.4A was considered rigid because there was continuous negative reinforcement in the beam over the column and two #9 top bars were placed through the grouted area. The column had longitudinal reinforcement extending into the 11" x 11" beam opening which was grouted before the roof planks were placed in position. Analysis was done for two different support conditions of RB-35 at column C3.5, one assuming hinged supports and one assuming fully fixed supports. In reality the actual support condition of RB-35 at column C3.5 would only be partially fixed because of lack of any "positive" connection between RB-35 and column C3.5. The far ends of the columns resting on the concourse level were regarded as hinged.

Three critical sections were considered to evaluate the various load combinations stated above. Those sections were a-a, b-b, c-c shown in figure 3.18. Section c-c was chosen because failure had apparently taken place at that section and the depth of the beam changed at that location. Section b-b, was chosen because the hairpin bars #702 were terminated at about that location. Section a-a, was chosen because of the change in depth of the section. Other sections were not considered of interest to the investigation.

The moment strengths of the beam at the three locations were computed based on details gathered from field observation and approved shop drawings. For section c-c, two bottom #7 bars were considered and the effective depth was taken as 20.8". It may be noted here that contrary to the approved drawing, see figure 3.18, where #701 bars were shown below the bottom bar of the hairpin, field observation indicated that the #701 bars were actually placed above the hairpins. This might have been caused by the depths of the shear stirrups marked #401 and #301 which would not facilitate the placement of bars as shown on the approved drawing. For section b-b, two bottom #7 bars were used with an effective depth of 21.7", based on field observation. For section a-a, two bottom #7 bars with an effective depth of 9.7" were considered. Based on the above and 5000 psi as the compressive strength of the concrete, the design moment strengths were computed.

The factored and unfactored bending moments at locations a-a, b-b and c-c due to the different loading conditions are given in table 6.1. Also included in the table 6.1 are the moment strengths at sections a-a, b-b and c-c, with and without capacity reduction factors.

Table 6.1

Factored Bending Moments Ft. kips
Load Factor = 1.4

	Moment strength Ft. kips ϕ = 0.9	Case 1	Case 2	Case 3	Case 4	Case 5
Section a-a	49.8	0.5	20	17.6	27	21.8
Section b-b	114.6	16.3	71.8	64.9	91.2	76.4
Section c-c	110.0	31.8	122.3	109.9	152.8	126.2

1. Support coordination of RB35 at column C3.5 assumed hinged.
2. Case 2 is the condition at the time of collapse.
3. Moment strength at sections based on flexural reinforcement only. Development length not considered. Discussed later.
4. All bending moments are positive moments.

Unfactored Bending Moments - Ft. kips
Load Factor = 1.0

	Moment Strength Ft. kips ϕ = 1.0	Case 1	Case 2	Case 3	Case 4	Case 5
Section a-a	55.3	0.33	14.3	12.6	19.3	15.6
Section b-b	127.3	11.7	51.3	46.3	65.1	54.5
Section c-c	122.0	22.7	87.4	78.5	109.2	90.2

1. Support condition of RB35 at column C3.5 assumed hinged.
2. Case 2 is the condition at the time of collapse.
3. Moment strength at sections based on flexural reinforcement only. Development length not considered. Discussed later.
4. All bending moments are positive moments.

Unfactored Bending Moments for Load Case 2 at Section c-c for Different Support Conditions

Load Factor = 1.0
Capacity reduction factor = 1.0

Support condition of RB-35 at column C3.5 hinged, bending moment = 87.43 ft.-kips

Support condition of RB-35 at column C3.5 fixed, bending moment = 70.3 ft.-kips

The moment strengths of sections a-a, b-b and c-c with the capacity reduction factor taken as 1.0 were all higher than the unfactored bending moments regardless of the assumption made for the support condition of RB-35 at column C3.5. This was true for all loading conditions. However, this was not the case when factored bending moments were considered and compared with the moment strengths of the sections a-a, b-b and c-c, including a capacity reduction factor of 0.9, as required by ACI 318-83. For load cases #2, #4 and #5 the factored bending moments were higher than the moment strengths at section c-c.

The unfactored bending moments at section c-c due to the load case #2 (the loading condition at the time of collapse) was 87.4 ft.-kips if the end of RB-35 at column C3.5 was assumed hinged. If it was assumed fixed, the unfactored bending moment was 70.3 ft.-kips. These two values represent upper and lower bounds. In reality, the actual moment would lie in between the two.

The development length of the bars marked #701 was examined. In order for the bars to be fully effective and to develop the full moment strengths of section c-c as shown in table 6.1, #7 bar should have a minimum embedment length of 21" as per ACI 318-83. As observed in the field, the actual development length of the #701 bars was 8" on the northface and 8 1/4" on the southface. An eight inch embedment length will only develop a bending strength of 42.4 ft.-kips with a capacity reduction factor of 0.9 and a bending strength of 47.1 ft.-kips without capacity reduction factor at section c-c.

The lap splice length between bars #701 and #702 was also examined at section b-b. For load case no. 2, the unfactored bending moment of section b-b was 51.3 ft kips and the factored bending moment was 71.8 ft kips. In the former case, the area of flexural steel needed was only 0.488 in^2 requiring a class B splice length of 27". In the later case, the area of flexural steel needed was 0.75 $in.^2$ which required a class C splice length of 36". The actual splice length observed was 32".

7.0 FABRICATION PROCESS, DISCUSSION AND CONCLUSION

7.1 Fabrication Process

A brief description is provided explaining the procedure for developing, reviewing and approving the precast design and shop drawings.

The contract drawings for the project, prepared by the Architect-Engineer team, called for the design and detailing of most of the precast members, e.g., columns, beams, arches and planks to be performed by the contractor of the Airside Building, subject to approval by the Architect-Engineer. The structural engineer of record provided the design parameters, e.g., design criteria, loads and geometry of the precast members. Specification sections no. 03415 and 03430 deal with the precast prestressed hollow core concrete planks and precast prestressed concrete structural frame, respectively. Specification section 03430 in subsection 02C states "Analyze and design precast units and connectors in accordance with the design criteria and the loads shown on the drawings" and "Each connection shall be designed and detailed by this contractor." In compliance with the requirements, the contractor obtained the services of a consulting structural engineer as its subcontractor to perform the design of the precast members and prepare the shop drawings for each of the precast members with the seal of a registered engineer in the State of Pennsylvania. The procedure employed by the contractor on the project to accomplish the tasks of designing, detailing, manufacturing and erecting the precast elements is described below.

The contractor obtained the service of a consulting structural engineer who designed and detailed the precast members and prepared the shop drawings. The shop drawings and the calculations were forward by the precast designer to the contractor who would send them to the construction manager of the project. The construction manager then forwarded them to the project architect. The project architect would, in turn, submit them to the structural engineer of record for his approval. The drawings and calculations would then either be approved or returned with necessary comments by the structural engineer of record. The approved drawings or the comments of the structural engineer of record were received by the precast designer through the same route through which the drawings were submitted. In case the drawings were returned disapproved, the precast designer would make necessary changes and resubmit them for approval through the same channels.

It is understood that the manufacturing of the precast elements was not undertaken unless the process of approval was completed and a final approved stamp of the structural engineer of record was placed on the shop drawings.

It was indicated by the structural engineer of record, during an interview with the Pittsburgh OSHA personnel, that the computations and shop drawings were not only reviewed, but also "checked" by his office. He, however, indicated that the checks were made only for the final service condition which would exist at the completion of the erection phase. The structural engineer indicated that he did not check the design for the erection loads at various phases of erection.

The precast manufacturer, having obtained the approved shop drawing, proceeded with the casting of the precast elements. An inspection company was employed by the contractor to assure the quality control of the elements produced in the plant. Among the duties of the inspection company was the task to check the placement of the rebars in the concrete forms before concrete was placed.

The precast designer provided a set of guidelines to the contractor to be used during the erection of the precast elements. The instructions are contained in a letter, see Appendix C.

7.2 Discussion

Chapter 6 of this report provided an analysis of beam RB-35 for the various loading conditions during erection. Case 2 was the loading condition which occurred immediately prior to the collapse. The analysis was conducted to compute the internal forces based on the actual loads supported by the beam without load factors. Similarly, limit state strength of the concrete beam was determined without using the capacity reduction factor. This procedure was undertaken to reflect the actual conditions to determine the cause of collapse. However, internal forces, and concrete beam strength were also computed by using the recommended load factor and capacity reduction factor to check the compliance with the design criteria of ACI 318-83.

A. Cause of Collapse

Structural analysis and field observations indicated that section c-c of beam RB-35 was deficient as it failed to develop the required flexural strength due to insufficient embedment length of the bottom bars marked #701. As the placement of roof planks progressed, section c-c was subject to increasing positive bending moment which exceeded its capacity and, hence, failure occurred. Due to the dead load of the planks, an unfactored positive bending moment of 70.3 ft.-kips or 87.4 ft.-kips was computed at section c-c depending upon whether the support condition of beam RB-35 at column C3.5 was assumed fixed or hinged. The flexural limit strength of the beam RB-35 without employing any capacity reduction factor, at section c-c was computed as 122 ft.-kips, provided the flexural bars were able to develop their full strength.

The flexural bars marked #701 were terminated near section c-c and, hence, must be embedded for a length of 21" to develop the full strength, as per ACI 318-83. However, field observation indicated an embedment length of 8" which reduced the flexural capacity of section c-c to 47.1 ft.-kips from 122 ft.-kips. The reduced flexural capacity of section c-c, due to the insufficient embedment length was less than the unfactored positive bending moment of the dead loads of the planks placed on the beam at the time of the collapse. If the support condition of RB-35 at column C3.5 was hinged, the actual capacity was 54% of the requirement. If the support condition was fixed, the actual capacity was 67% of the requirement.

B. Placement of Bar #701 on Beam RB-35

The proper placement of the bars marked #701 was, therefore, crucial because it affected the flexural capacity of the beam RB-35. See figure 3.18 for location of bar marked #701 as it appeared on the shop drawing. The precast elements manufacturer and the plant quality control personnel had stated in their interviews that the shop drawing did not specifically indicate the location of rebars #701 in beam RB-35, but, rather it had been shown in the general area of where it could be placed. They stated that the dimension line near one end of the #701 rebar was used as the dimension for locating the spacings of shear stirrup and not as the beginning of the bars marked #701. They indicated that due to this lack of clarity the bars were placed in the general area of where it was shown on the drawing. The precast element designer and structural engineer of record, differed with this view during their interview. They both stated that the location of the bars marked #701 was indicated on the shop drawing by the same dimension line as for the shear stirrups. As per the precast element designer and the structural engineer of record, this dimension line indicated the spacing of shear stirrups marked #401 and also the beginning of the bars marked #701. In their opinion, no additional information was needed for the placement of rebar marked #701 in the beam.

C. Lap Splice of Rebars #701 and #702

It must be mentioned here that the placement of the bar marked #701 had impacted the beam in two ways. One end of the bar provided the development length needed for the full flexural capacity at section c-c, and the other end provided the lap splice length between bars marked #701 and #702 which affected the flexural capacity of section b-b. The precast designer stated in his interview that if the bars were placed at the dimension line shown on the plan, a development length of 25" would be available, sufficient to develop the full flexural capacity of beam RB-35. However, the lap splice length on the other end would then be reduced to 15-1/2". A lap splice length of 15-1/2" is less than the required Class C splice length of 36", as per ACI 318-83. Correct splice lengths were critical to develop the full flexural strength at section b-b. However, the actual provided splice length of bar #701 was 32" due to the shortened development length of other end. Figure 7.01 illustrates the reinforcement requirements as per ACI 318-83.

D. Compliance with ACI 318-83

It is a generally accepted engineering practice to apply ACI 318-83 design criteria when determining the reinforcement requirements of concrete beams for the construction and service loads. At section c-c, it was determined that by using a load factor of 1.4, the factored bending moments for case 2, 4 and 5 exceeded the flexural capacity of the beam, computed with the capacity reduction factor of 0.9. For case load 3, the amount of steel was marginal, as per ACI 318-83.

E. Erection Sequence

The precast erector did not seem to have followed the erection sequence recommended by the precast element designer in its entirety. Two deviations from the recommendations were noticed and their possible impact on the beam RB-35 are discussed below.

"The precast concrete erection sequence" prepared by the designer of the precast elements, see appendix B, had specified that the placement of roof beams shall be completed prior to the erection of roof planks. Though the arch was not specifically mentioned in the erection sequence, the precast elements designer had stated in his interviews that the arch was included among all the beams to be erected before roof planks would be placed in position.

If the erector had considered the roof arch A-3 as one of the roof beams and followed the procedure, there would have been a slight reduction in the bending moment at section c-c. For load condition #2, the reduction in the bending moments would have been on the order of approximately 10%.

This reduced bending moment due to the cantilevered load of the arch would still have required section c-c to develop full flexural capacity, thus requiring 21" development length of the bottom bar #701. So the placement of the arch had little significance on the flexural requirement of beam RB-35 for load case #2.

Another deviation was the fact that the pour strips at the concourse level were not poured prior to the erection of precast elements, e.g., beams, arch, planks at the roof level. This would have resulted in the base of the columns at the concourse level becoming rigid instead of hinged as was the case at the time of the accident. However, it would have made little difference in the bending moment requirement at section c-c of beam RB-35 for load case no. 2.

7.3 Conclusion

The following conclusions by the Occupational Safety and Health Administration are based on the examination of the collapsed structure, review of interviews of eyewitnesses and consultants, and structural analysis:

(1) The roof beam RB-35 at column line B20 failed in flexural due to the inadequate development length of its bottom bars marked #701.

(2) The precast elements manufacturer and his quality control personnel had stated that the location of the two bottom bars marked #701 was not specifically dimensioned on the approved shop drawing. They further indicated that the bars were, therefore, placed in the general area of where they were shown on the approved plan.

(3) The precast designer and structural engineer of record stated in their interview that the location of the bottom bars marked #701 for beam RB-35 was dimensioned on the shop drawing. They indicated that the bars were not placed in accordance with the information contained in the shop drawing.

(4) The flexural reinforcement of two #7 bars of beam RB-35 to support the construction loads during erection was marginal as per ACI 318-83.

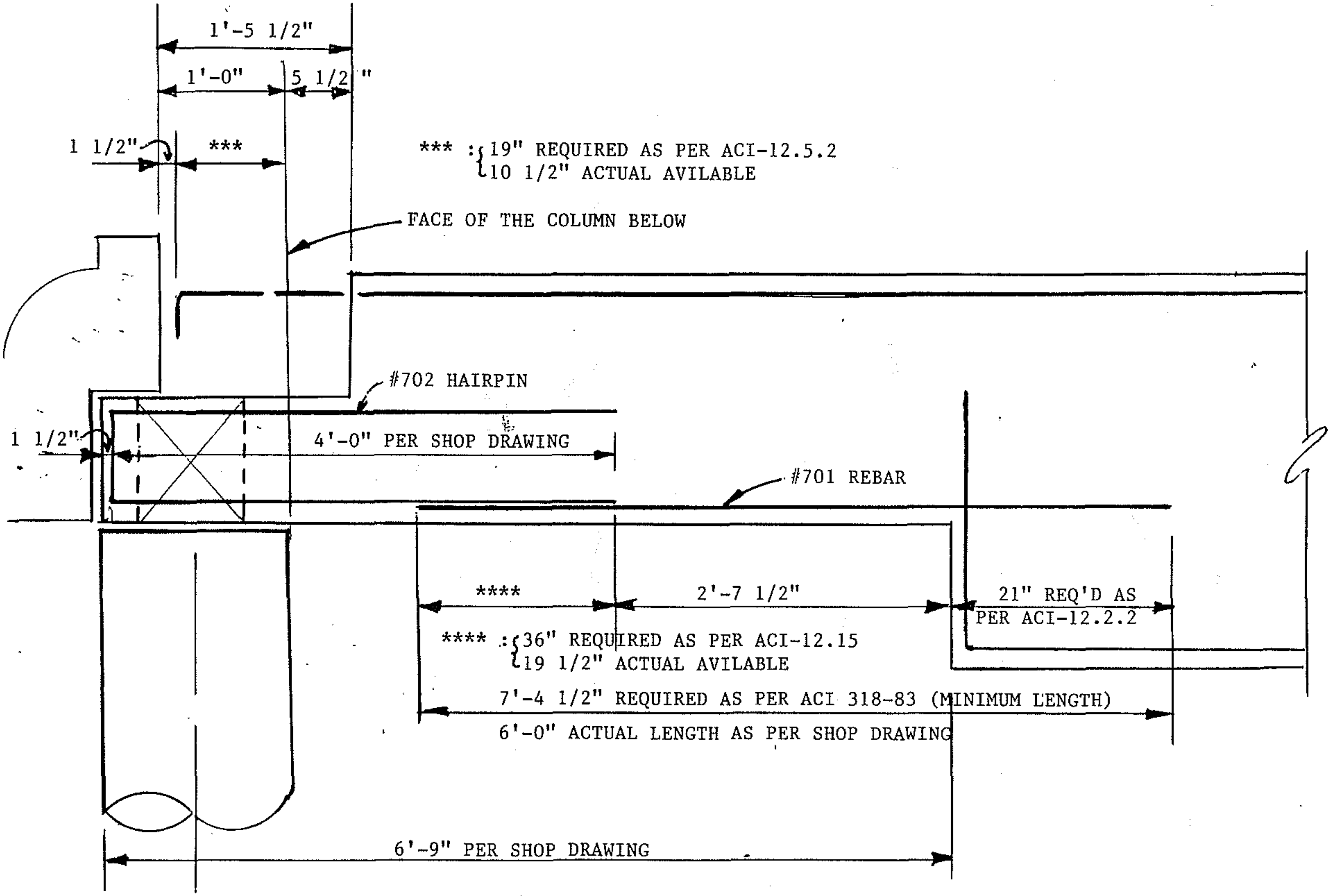
(5) The lap splice length of #7 bars marked #701 and #702 did not meet the ACI design criteria.

(6) Four #8 bars for the longitudinal bottom reinforcement of beam RB-35 were not properly spaced by the precast manufacturer.

(7) Field examination of the embedded reinforcing steel had indicated a series of inconsistencies in placement of bars in several precast elements.

(8) The precast erector proceeded with the roof erection without completing all the pour strips at the concourse level, as called for in the erection sequence.

(9) The structural engineer of record did not check the design of precast elements and the details of the shop drawing for the construction load.



REINFORCEMENT REQUIREMENT AS PER ACI 318-83

Figure 7.01

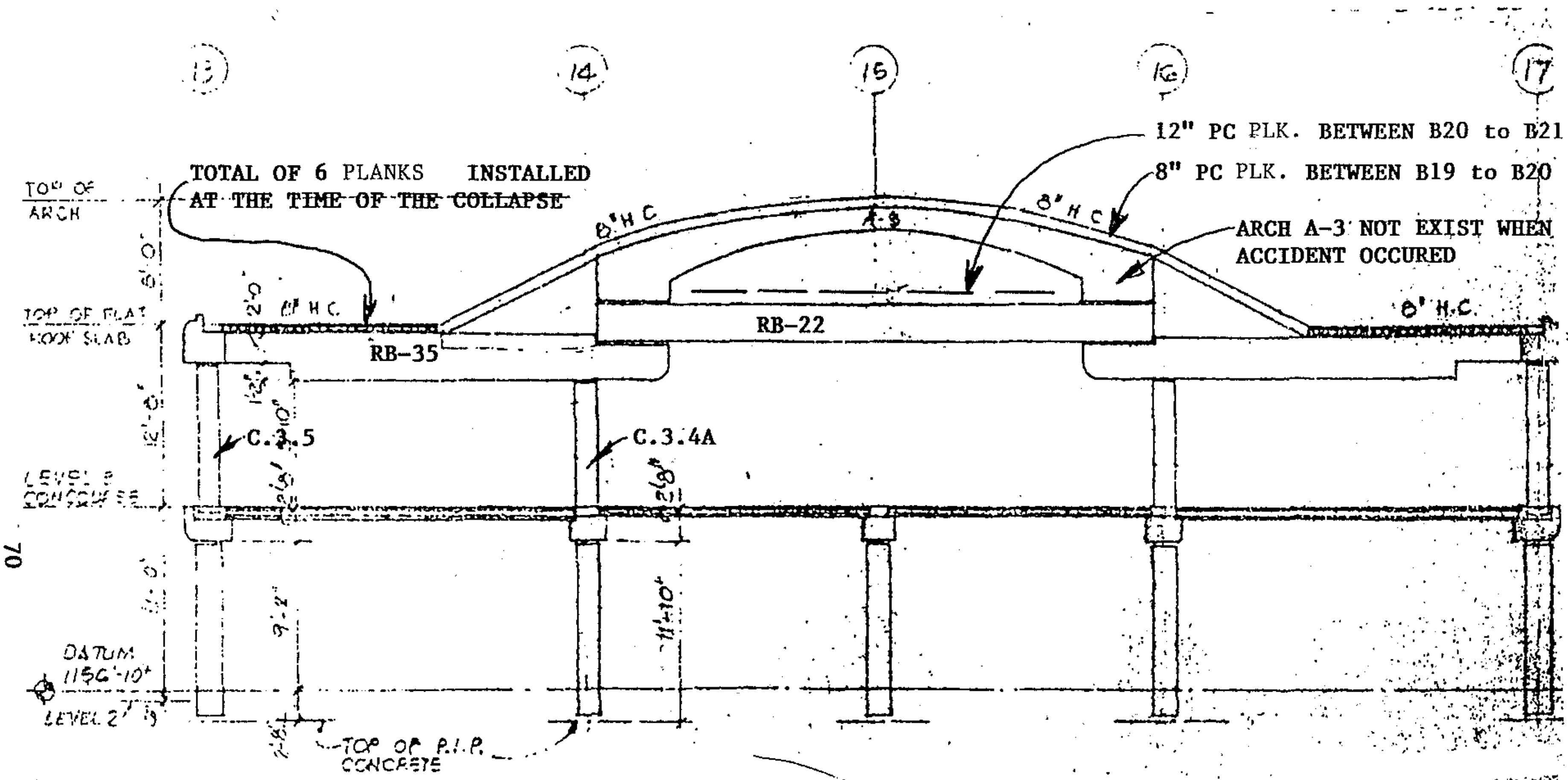
REFERENCES

[1] Building Code Requirements for Reinforced Concrete (ACI 318-83).

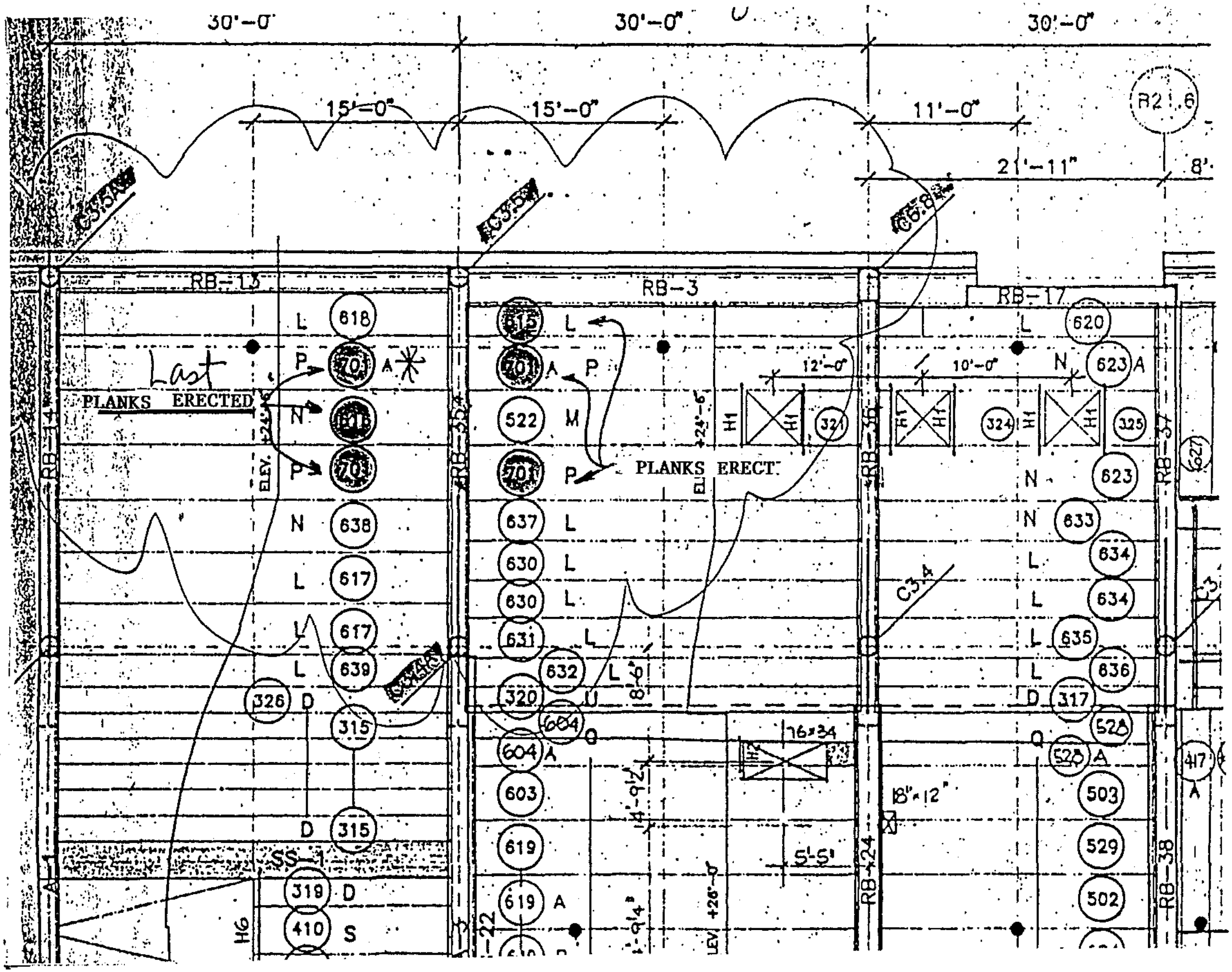
[2] "STAAD-III/ISDS Program User's Manual"
Research Engineers, Inc.
590 Lippincott Drive Marlton, NJ

APPENDIX A

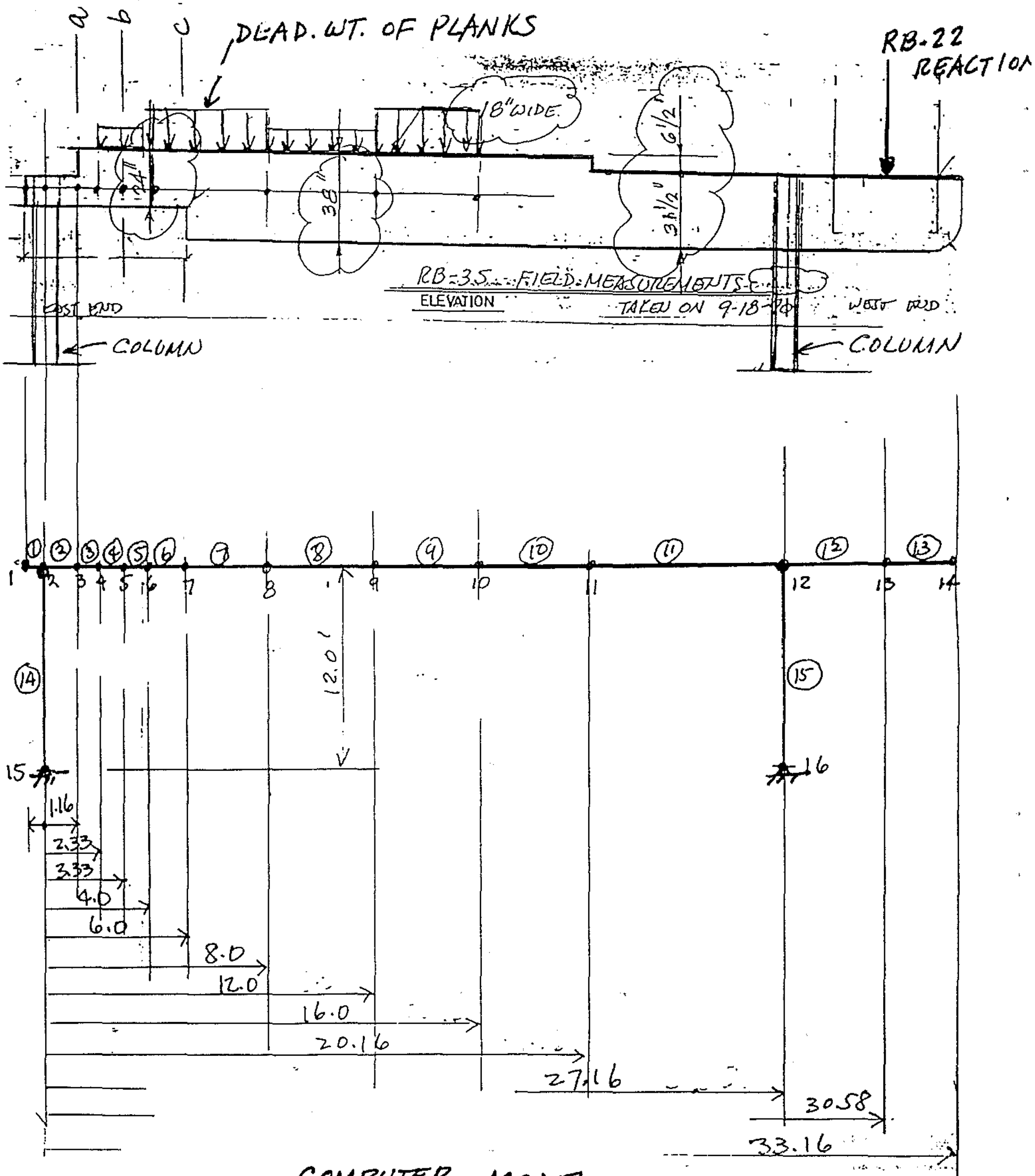
COMPUTATIONS



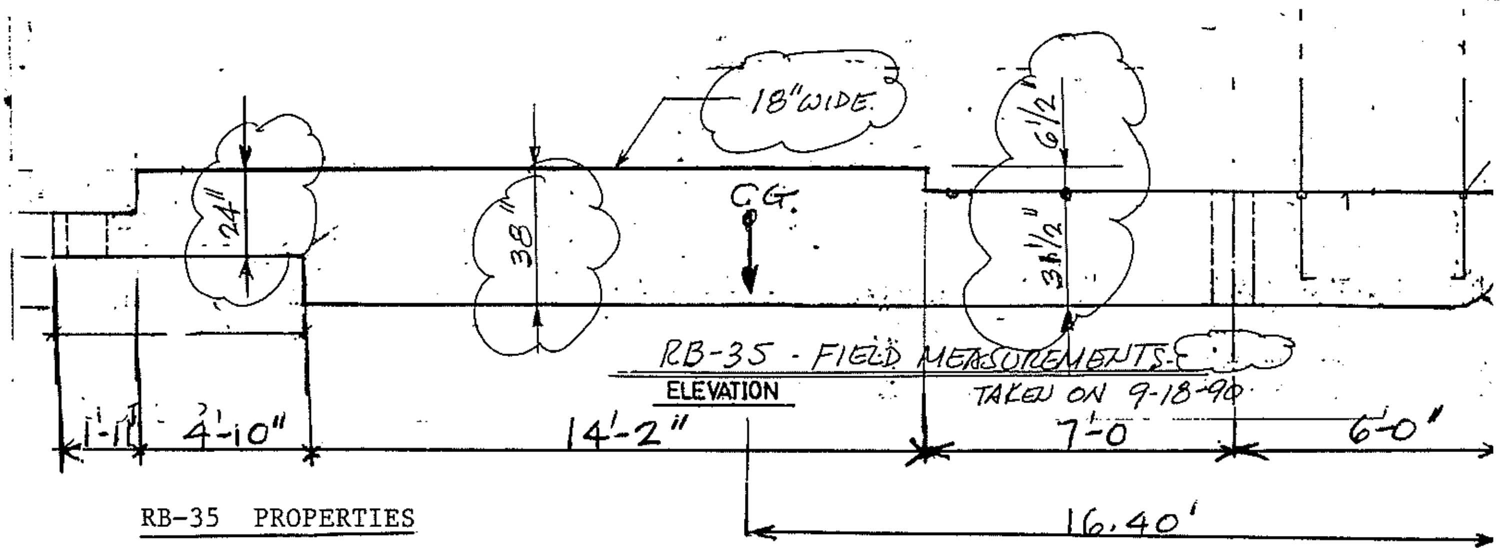
CROSS SECTION



6 PLANKS WERE ERECTED PRIOR TO THE ACCIDENT



COMPUTER MODEL



RB-35 PROPERTIES

SECTION 18"X12"
A=216 IN^2 $I=1/12\ (18)\ (12)^3=2592\ IN^4$ (218#/')

18"x24"
A=432 IN^2 $I=1/12\ (18)\ (24)^3=20736\ IN^4$ (435#/")

18"X38"
A=684 IN^2 $I=1/12\ (18)\ (38)^3=82308\ IN^4$ (689#/')

18"X31.5"
A=567 IN^2 $I=1/12(18)\ (31.5)^3=46884\ IN^4$ (571#/')

* DENSITY OF CONCRETE=145#/CU. FT.

TOTAL WEIGHT OF RB 35

$W=(218X1.92-0.92^2X1X145)+(435X4.83)+(689X14.17)+571X13$

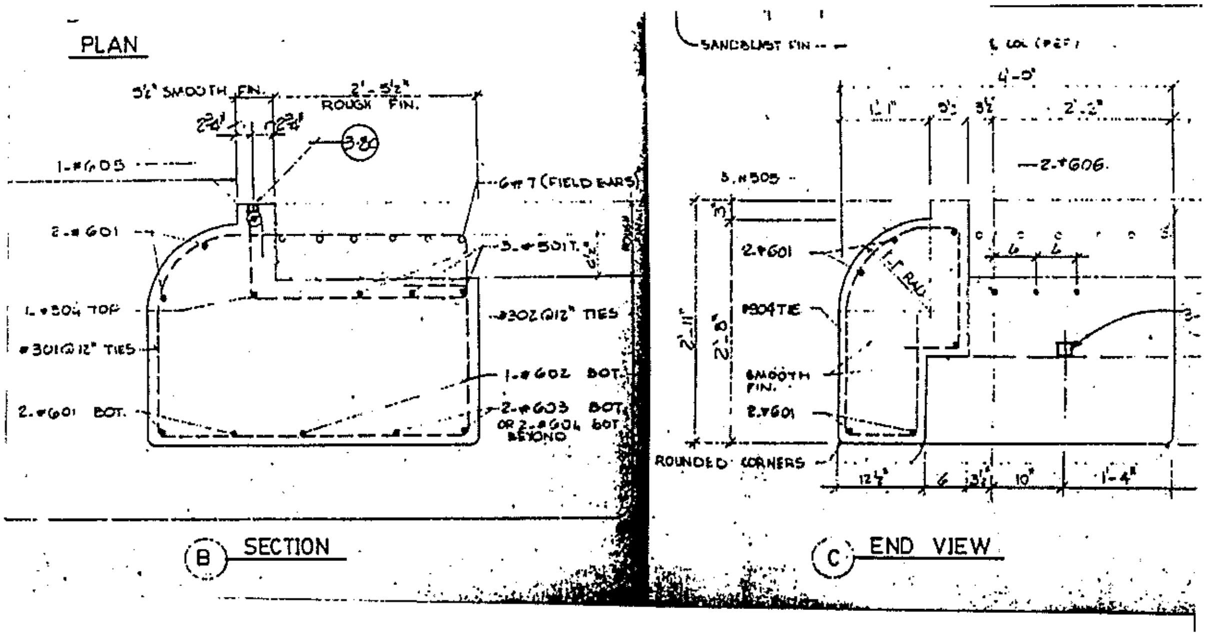
=296# +2101# +9763# +7423#

=19583#

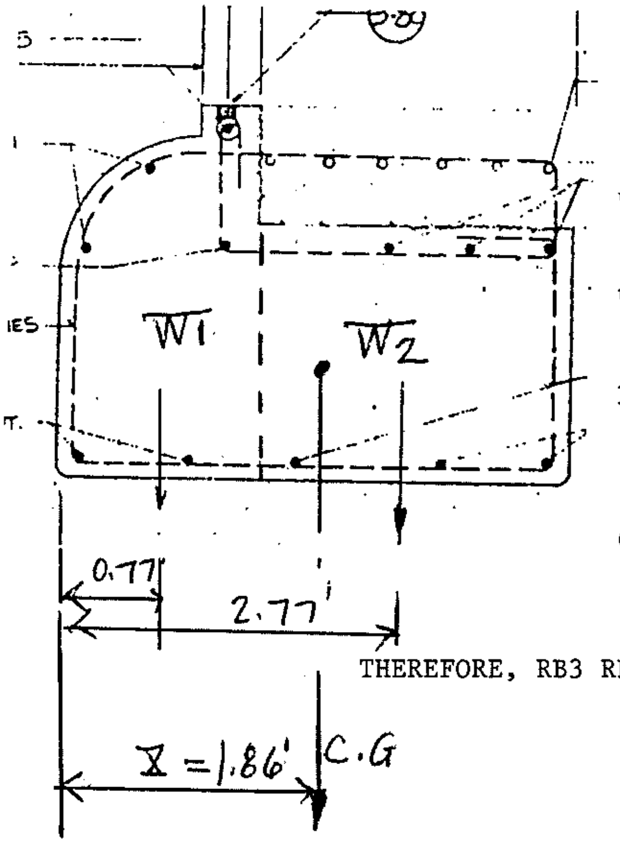
C.G. OF THE BEAM FROM RIGHT END

$$X=\frac{7423X6.5+9763X20.66+2101X29.58+296X32.96}{19.583}$$

= 16.40' From right end



RB-3/3A : TOTAL LENGTH= 38'



W1 = 1.54X2.67X1.45=596#/'

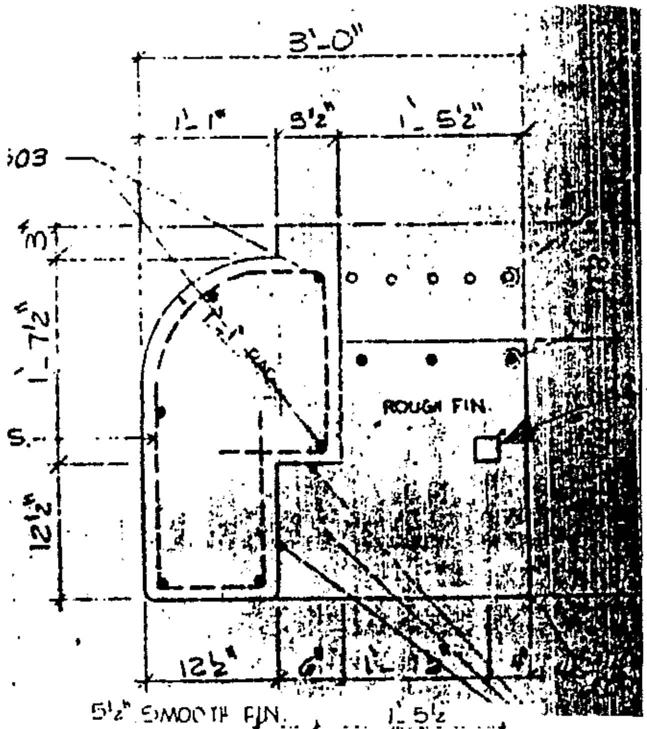
W2 = 2.458X2.0X145 =713#/'

W1+W2 =596#/' + 713#/' =1309#/'

$$\tilde{X} = \frac{596 \times 0.77 + 713 \times 2.77}{1309} = 1.86' = 1'\text{-}10\ 1/2''$$

C.G. @ CENTER LINE OF THE COLUMN

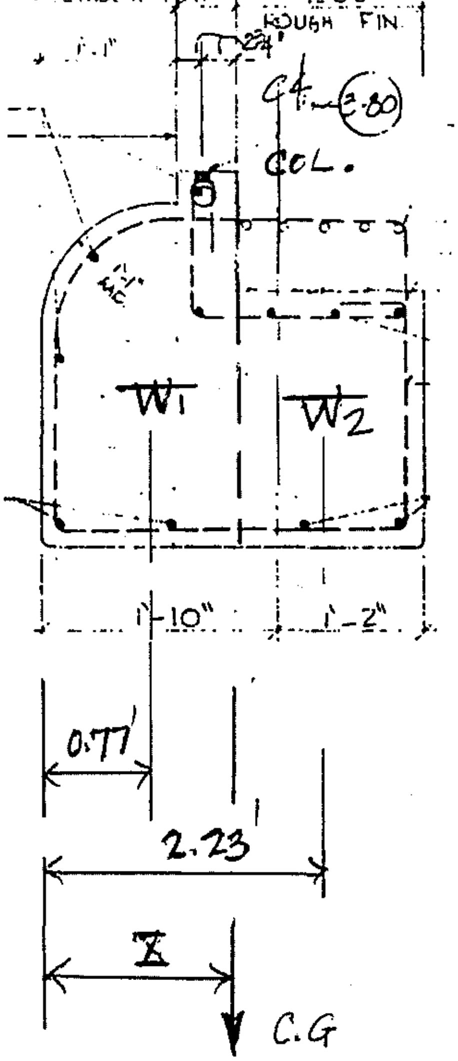
THEREFORE, RB3 REACTION APPLIES @ CENTER LINE OF THE COLUMN.



RB-13 AT ROOF

RB-13 TOTAL LENGTH =30'-0"

STEEL TUBE AT SUPPORT @ EA. END OF COLUMN



W1=2.67X1.54X145= 596X30= 17880#.

W2=1.46X2.0X145= 423#/'X30'=12700#

W1+W2= 30580#

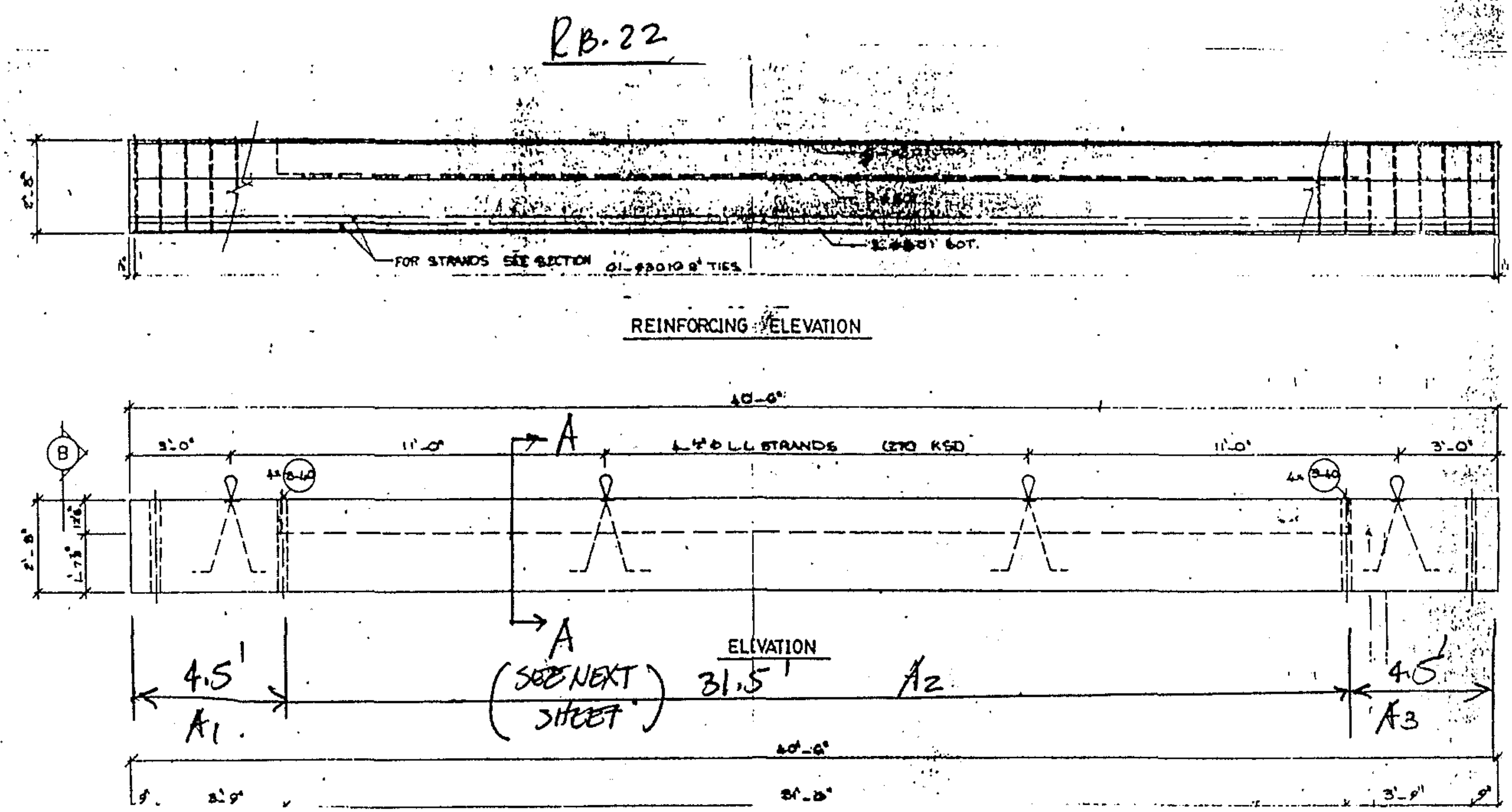
C.G. OF THE BEAM FROM FACE

$$X=\frac{17880X0.77+12700X2.23}{30580} = 1.376' = 16.5"$$

THEREFORE,

C.G. OF THE RB-13 FROM THE CENTER LINE OF THE COL.

≅ 5" = 0.42"



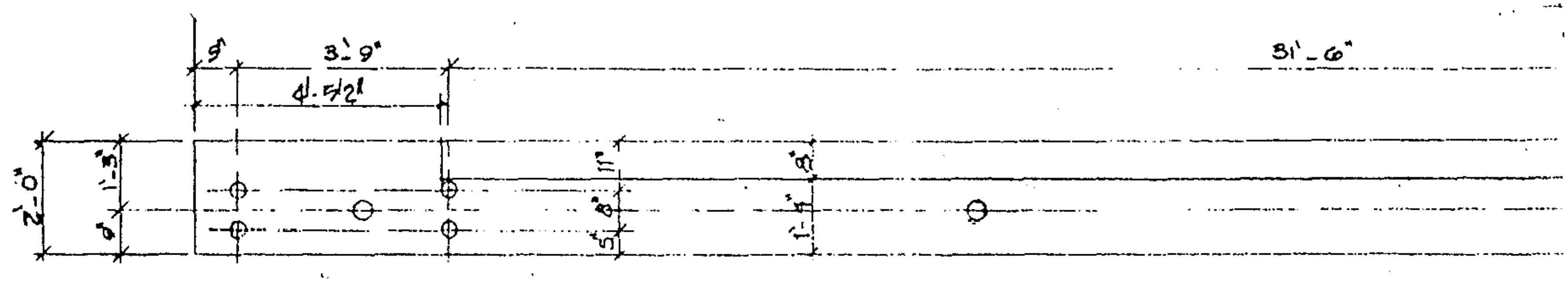
BEAM MARK RB-22

RB22 (1)

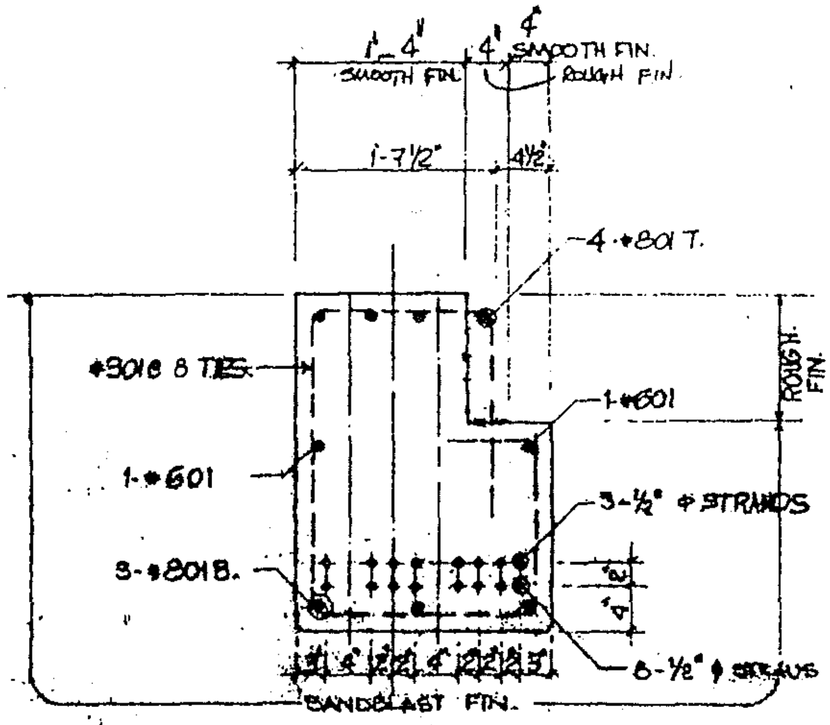
AREA A1 = 24" X 32" = 768 "2 ------773 #/'

A2 = (24 X32) -(12 X 8) = 672 "2 --------- 677 #/'

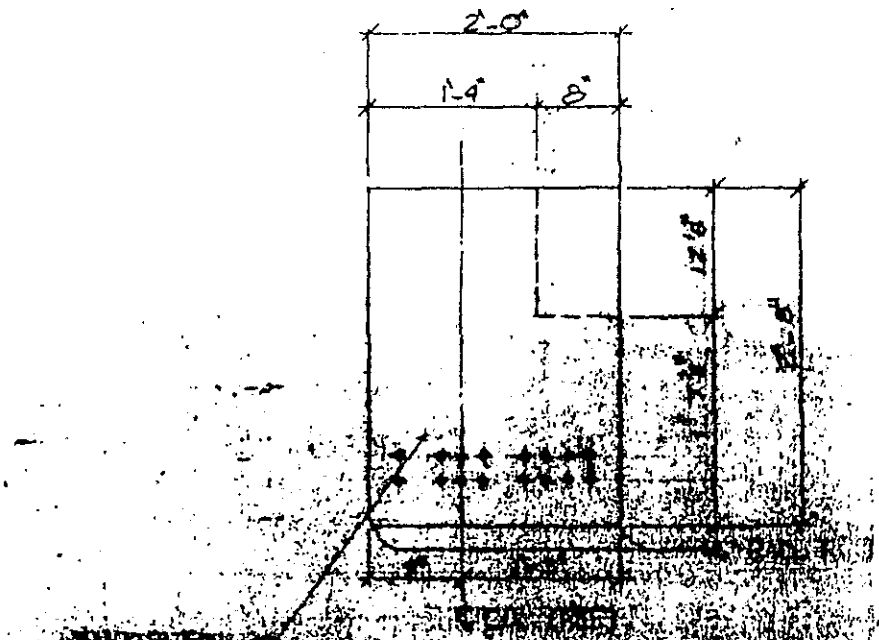
$$\text{REACTION AT END} = \frac{773 \times 9 + 677 \times 31.5}{2} = 14{,}140\ \#$$



PLAN



(A) SECTION

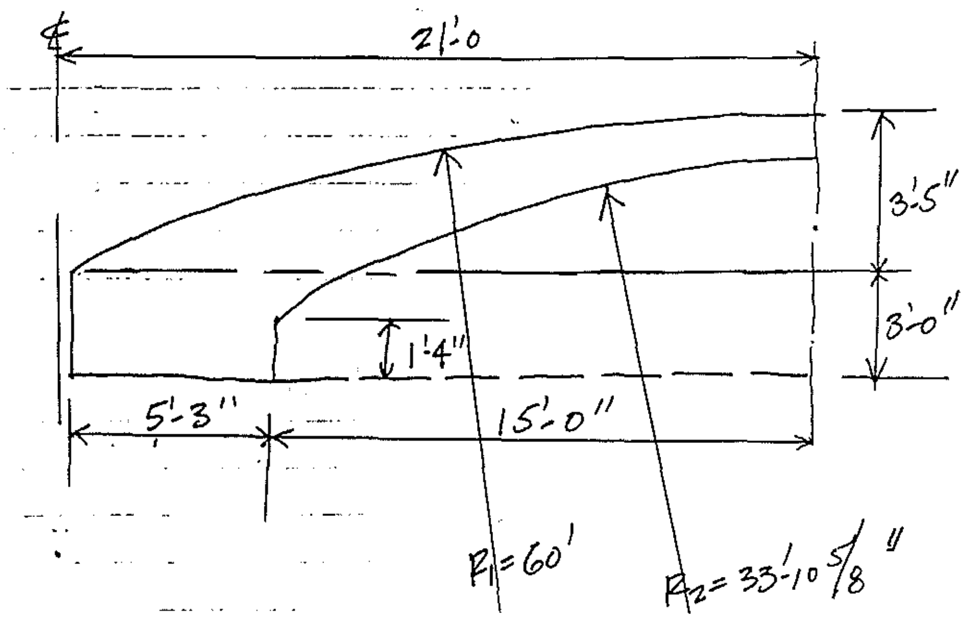


(B) END VIEW

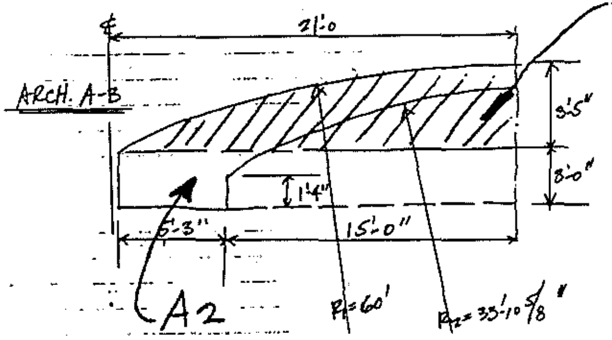
BEAM RB-22

RB 22 (2)

ARCH A-3



WIDTH OF THE ARCH.=18"



AREA A1

$C = 2 \times 20.25 = 40.5'$

$b = r - \frac{1}{2}\sqrt{4r^2 - c^2}$

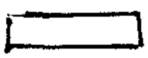
$= 60 - \frac{1}{2}\sqrt{[4(60)^2 - (40.5)^2]}$

$= 3.52'$

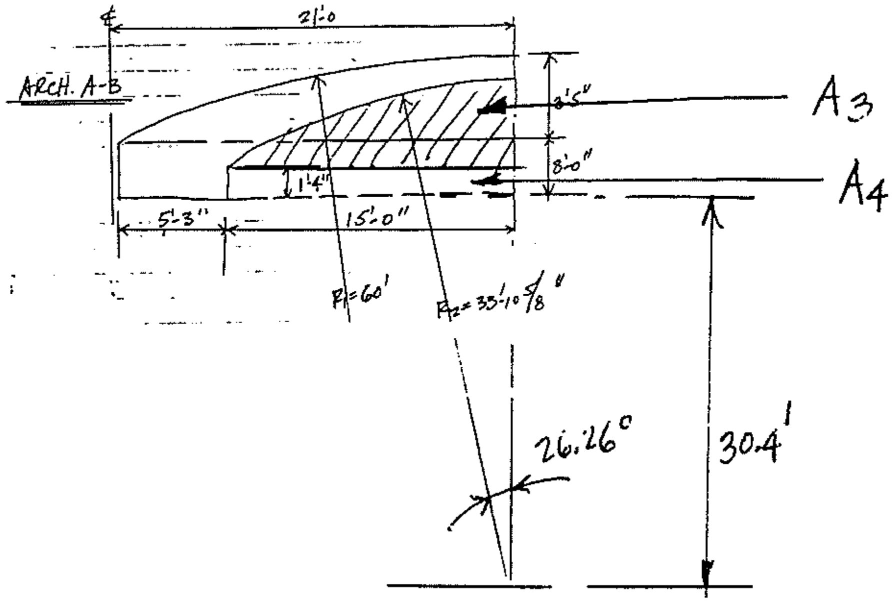
ANGLE A= $\left(\sin^{-1}\frac{20.25}{60}\right) \times 2 = 39.45^{\circ}$

$\widehat{\text{ARC}}\ A = \frac{\pi r^2\ (39.45)}{60} = 41.31'$


AREA A1 = $\frac{1}{2}\left[\frac{41.31 \times 60 - 40.5 \times (60-3.52)}{2}\right] = 47.8$ SQ. FT.


AREA A2 = 3.0 X20.25 = 60.75 SQ. FT.

TOTAL AREA OF A1 + A2 =47.8+60.75 =108.55 SQ.FT.



$$\widehat{ARC} = \frac{\pi \times 33.9 \times 26.26 \times 2}{180} = 31'$$

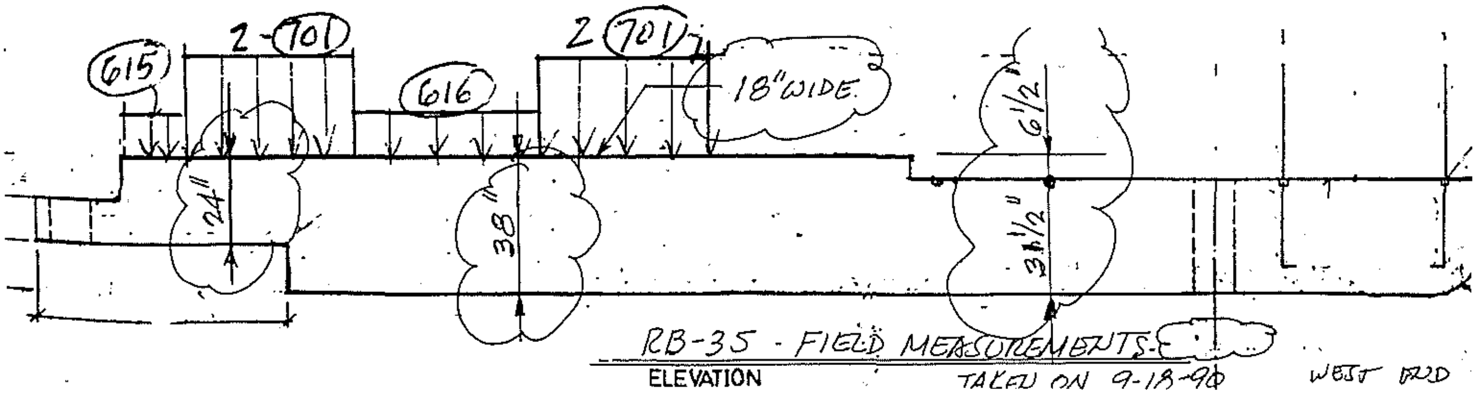
AREA A3 $= \frac{1}{2}\left[\frac{31 \times 33.9 - 30 \times (33.9 - 3.5)}{2}\right] = 34.7$ SQ.FT.

AREA A4 = 1.33X15 =19.95 SQ. FT.

TOTAL ARES OF A3+A4 = 34.7 +19.95 = 54.65 SQ. FT.

TOTAL AREA OF 1/2 ARCH = (A1+A2) -(A3+A4)
=53.9 SQ. FT.

TOTAL WEIGHT OF THE 1/2 ARCH A3
= (53.9X1.5) X145
= 11723#



DEAD WEIGHT OF THE SLAB ON RB-35 @ COLLAPSE

1 PLANK SLAB = 58#/sq. ft. X [(30'-0")-(0'-8")]X ½ =851 #/'

2 PLANK SLABS= 58 X29.33 = 1701 #/'

REACTION FROM BEAMS AND ARCH A3

RB-22 =14150#

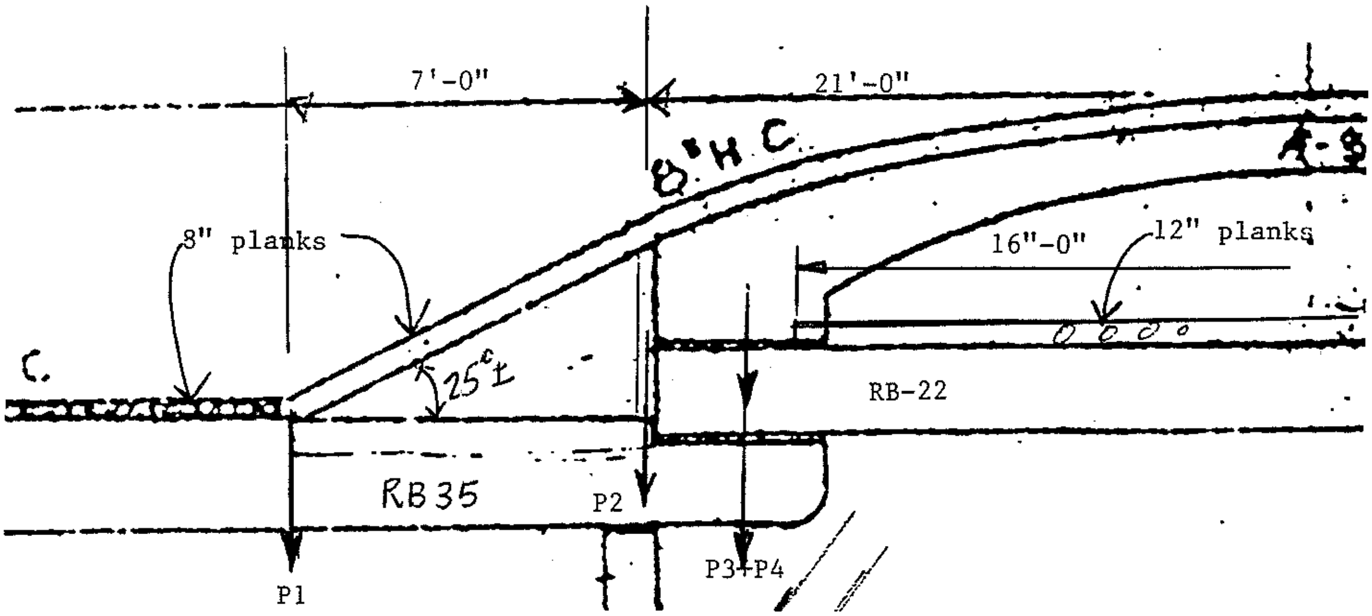
ARCH A3 = 11723 #--------NOT EXIST AT THE TIME OF THE COLLAPSE

REACTION FROM RB-13

RB-13 = $\frac{30580}{2}$ = 15290 #

APPLIES AT 0.42' FROM CENTER LINE OF THE COLUMN.

TOTAL LOADS TO RB -35 (DEAD LOAD) WHEN ALL ROOF SLABS ARE IN PLACE:



LOAD (REACTION) FROM SLOPING PORTION OF THE ROOF SLABS

$$P1 = P2 = 58X\ 29.33\ X \frac{7'}{\cos 25} = 6.57 \text{ kips}$$

REACTION FROM ROOF SLAB @ ARCH AND @ RB-22

$$P3 = 58X\ 21\ X\ \frac{29.33}{2} = 17862\ \#$$

$$P4 = 87\ X16\ X\ \frac{29.33}{2} = 20414\ \#$$

TOTAL REACTION TO THE CANTILEVER END OF THE BEAM

P= ARCH A3 + RB 22 +P3 + P4

= 64.13 kips

COLUMN PROPERTIES

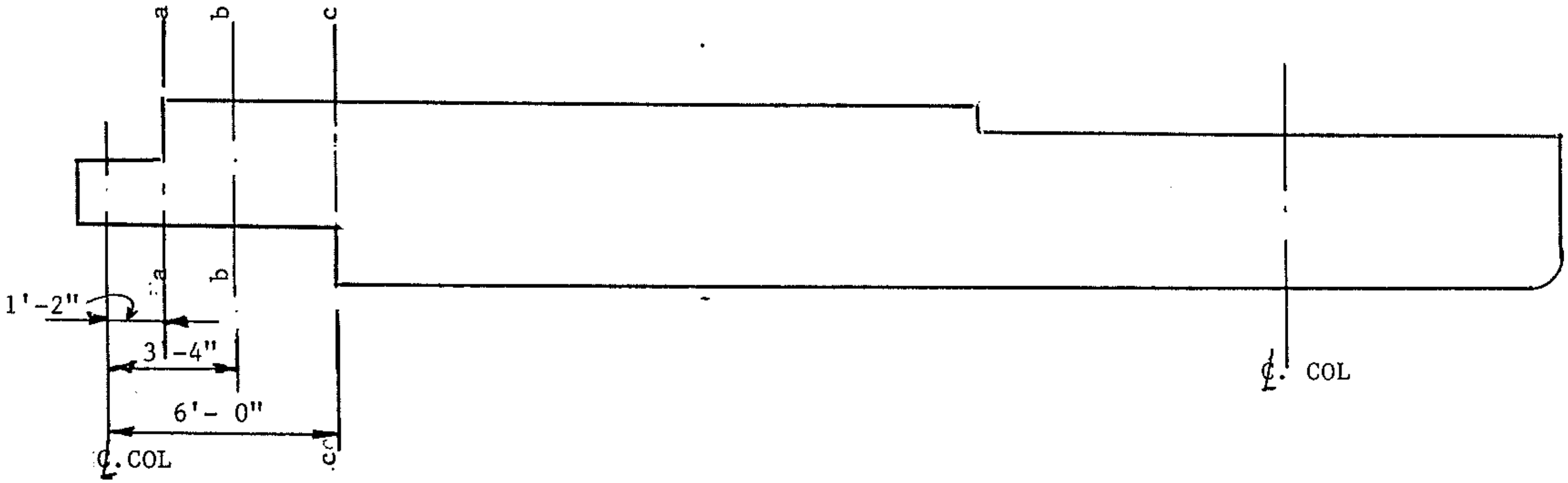
COLUMN C3.5 = 16" ϕ EXTERIOR COLUMN

COLUMN C 3.4A =16" ϕ INTERIOR COLUMN

$$I= \frac{\pi \times 18^2}{64} = 5153 \text{ IN.}^4$$

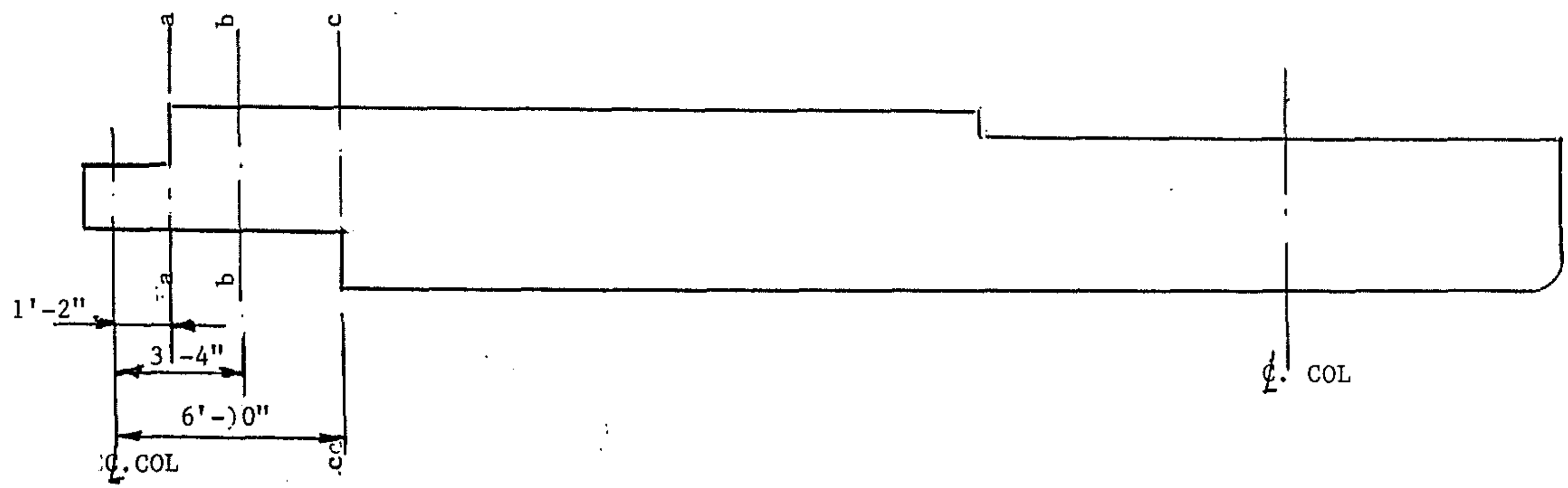
$$A= \pi (9)^2 = 254 \text{ IN SQ.}$$

$$Ec = 57000\sqrt{f'c} = 4030 \text{ KSI} \text{ — ACI 8.5}$$



CRITIAL SECTION / BENDING MOMENT (KIP-FT) / LOADING CONDITION	SECTION a-a		SECTION b-b		SECTION c-c	
	M_D	1.4 M_D	M_D	1.4 M_D	M_D	1.4 M_D
LOAD CASE 1: SELFWEIGHT OF BEAM RB-35, REACTION FROM BEAM RB-22, AND REACTION FROM SPANDEL RB-13	0.33	0.5	11.67	16.34	22.70	31.8
LOAD CASE 1a: SAME AS ABOVE, BUT WITHOUT REACTION FROM SPANDEL RB-13	6.44	9.07	17.30	24.2	27.71	38.8

DEAD WEIGHT OF BEAM MEMBER ONLY, PRIOR TO ANY PLANK INSTALLATION.

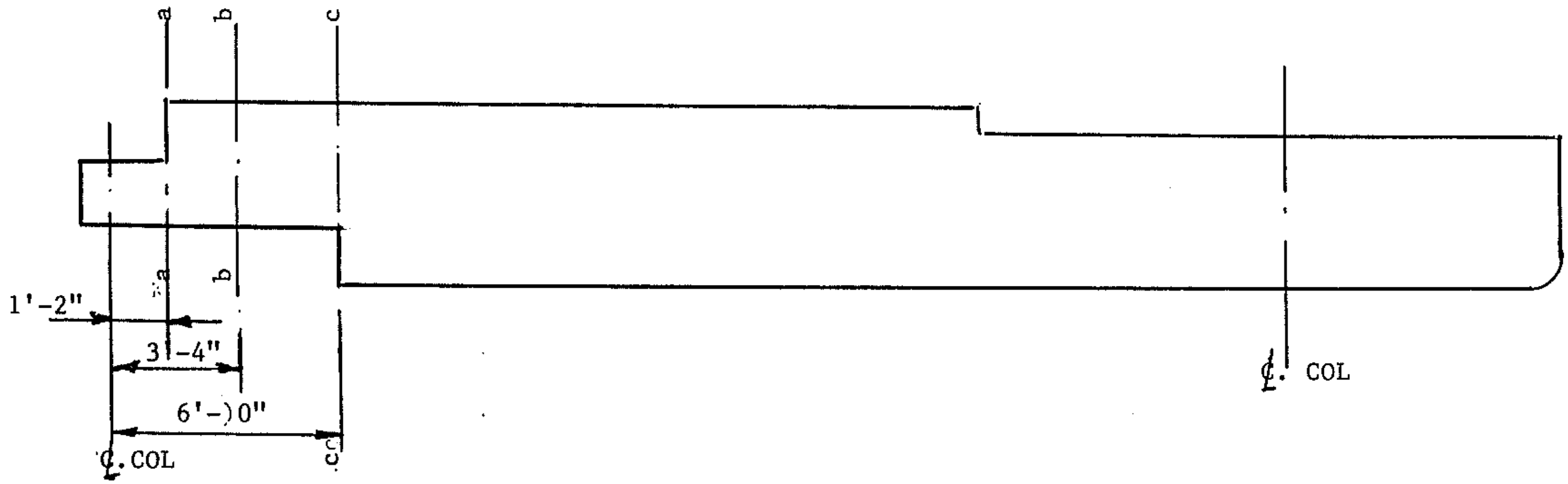


CRITICAL SECTION / BENDING MOMENT (KIP-FT) / LOADING CONDITION		SECTION a-a		SECTION b-b		SECTION c-c	
		M_D	1.4 M_D	M_D	1.4 M_D	M_D	1.4 M_D
LOAD CASE 2: SELFWEIGHT OF BEAM RB-35, REACTION FROM RB-22, REACTION FROM RB-13, AND WEIGHT OF THE 6 PLANKS		14.27 (14.28)	19.98	51.25 (51.29)	71.75	87.37 (87.43)	122.32 **
LOAD CASE 2a: SAME AS ABOVE:	FULL FIXITY AT TOP OF THE COLUMN HINGED AT BOTTOM	(-2.73)		(34.23)		(70.30)	
	BOTH ENDS OF COL ASSUMED FIXED	(-5.53)		(31.62)		(67.43)	

LOADING CONDITION AT THE TIME OF THE ACCIDENT (LOAD CASE 2)

** : BENDING MOMENT EXCEEDS ALLOWABLE AS PER ACI -318-83

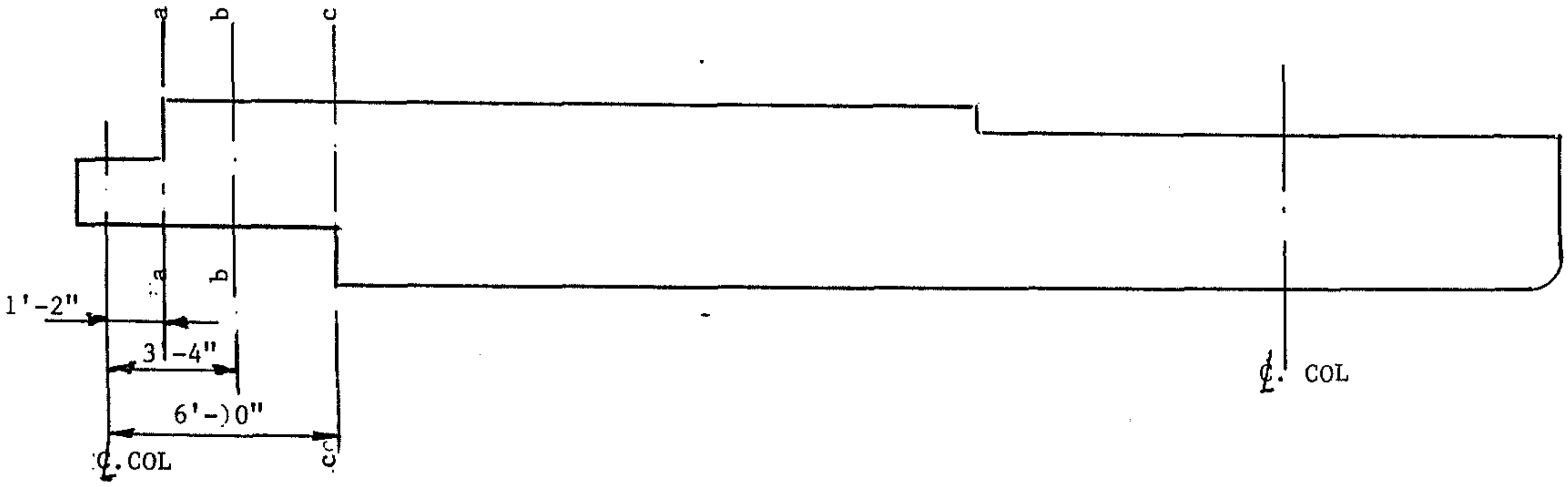
(***.**): BENDING MOMENT OF THE MEMBER WHEN Icr IS USED



CRITICAL SECTION / BENDING MOMENT (KIP-FT) / LOADING CONDITION	SECTION a-a		SECTION b-b		SECTION c-c	
	M_D	1.4 M_D	M_D	1.4 M_D	M_D	1.4 M_D
LOAD CASE 3: SELFWEIGHT OF BEAM RB-35, REACTION FROM RB-22 & RB-13. DEAD WEIGHT OF THE 6 PLANKS AND THE REACTION OF ARCH A-3	12.55 (12.56)	17.57	46.32 (46.33)	64.85	78.48 (78.50)	109.87

LOADING AT THE TIME OF THE ACCIDENT AND WITH THE REACTION FROM ARCH A-3

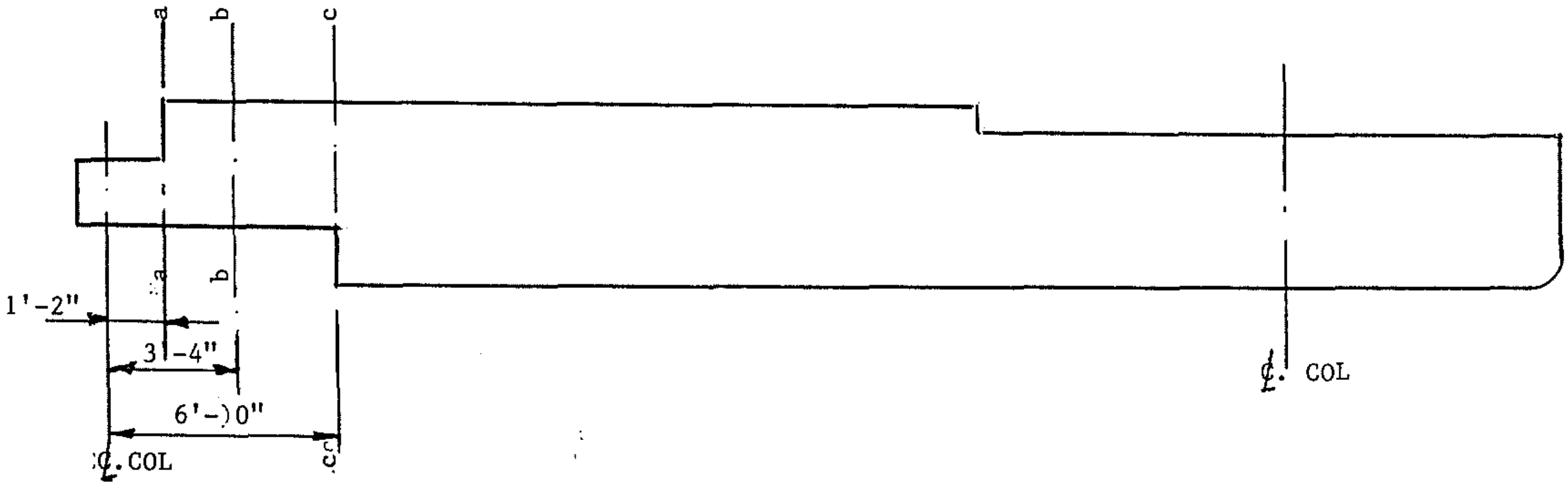
(***.*) : Icr. USED



CRITICAL SECTION / BENDING MOMENT (KIP-FT) / LOADING CONDITION	SECTION a-a		SECTION b-b		SECTION c-c	
	M_D	$1.4\ M_D$	M_D	$1.4\ M_D$	M_D	$1.4\ M_D$
LOAD CASE 4: SELFWEIGHT OF BEAM RB-35, REACTIONS OF RB-22, RB-13 AND ARCH A-3, ALL LEVELED PLANKS INSTALLED (10 total)	19.26 (19.28)	26.96	65.14 (65.19)	91.20	109.14 (109.23)	152.8 **

ALL 10 PLANKS IN EXTERIOR BAY LEVELED ROOF PORTION ARE INSTALLED.

1. (***.**) : Icr USED
2. ** : BENDING MOMENT EXCEEDS ALLOWABLE



CRITICAL SECTION / BENDING MOMENT (KIP-FT) / LOADING CONDITION	SECTION a-a		SECTION b-b		SECTION c-c	
	M_D	$1.4\ M_D$	M_D	$1.4\ M_D$	M_D	$1.4\ M_D$
LOAD CASE 5: ALL BEAMS, ALL PLANKS AND ARCH. (COMPLETED STRUCTURE) NO TOPPING , NO LIVE LOAD	15.59 (15.57)	21.83	54.62 (54.56)	76.44	90.17 (90.07)	126.24**

COMPLETED STRUCTURE PRIOR TO THE CASTING OF THE POUR STRIP (ie, NO TOPPING, NO LIVE LOAD)

1. (***.**) : Icr USED
2. ** : BENDING MOMENT EXCEEDS ALLOWABLE

DESIGN MOMENT STRENGTH OF BEAM RB-35

(A) : SECTION a-a

18"X12" b=18"

d=12"-1 1/2"-3/8"-7/16" =9.7"

As= 2 #7 = 2X0.6 = 1.2 sq. in

ACI 10.3 (A) (1):

$$a = \frac{As.\ fy}{0.85\ f'c\ b} = \frac{1.2\ X\ 60}{0.85\ X\ 5\ X\ 18} = 0.941$$

$$\begin{aligned}\phi\ Mn &= \phi\ [\ As\ fy\ (\ d-\ a/2)] \\ &= \phi\ [\ 1.2\ X\ 60\ (\ 9.7 - 0.941/2)\ ] \\ &= \phi\ [664.5\ k''\ ] \\ &= \phi\ [\ 55.38\ k']\end{aligned}$$

ACI 9.3.2.1

$\phi = 0.9$

$\therefore \phi$ Mn= 0.9 X 55.38

= 49.84 k'

IF ϕ=1.0 ϕ Mn= 55.38 k'

(B): SECTION b-b

18"X24" b=18" h= 24"

d= 24" - 1 1/2" - 3/8" - 7/16" = 21.7"

As = 2-#7 = 2 X 0.6 = 1.2 sq. in.

$$\rho = \frac{1.2}{18 \times 21.7} = 0.00307 \triangleq \rho(\text{min. req'd})$$

ρ(min. req'd)= 0.00333

$$a = \frac{1.2 \times 60}{0.85 \times 5 \times 18} = 0.941$$

ACI 10.3 (A) (1)

ϕ Mn = ϕ [1.2 X 60 X (21.7 - 0.941/2)]

= ϕ [1528 k"]

= ϕ [127.4 k']

ϕ = 0.9 ϕ=1.0

$\therefore$ ϕ Mn = 114.6 k' ϕ Mn = 127.4 k'

(C): SECTION c-c

18"X24" d= 24" -1 1/2" - 3/8" - 7/8" - 7/16" = 20.8"

0 Mn = ϕ [1.2 X 60 X (20.8 -0.941/2)]

= ϕ [1463 k"]

= ϕ [122 k"]

ϕ = 0.9 ϕ=1.0

$\therefore$ ϕ Mn = 110.0 k' ϕ Mn =122 k'

ACTUAL BENDING MOMENT VERSUS ALLOWABLE DESIGN STRENGTH (ACI 318-83)

LOCATION/ LOAD CASE	ACTUAL BENDING MOMENT (KIP-FT)	DESIGN MOMENT STRENGTH (KIP-FT)	EXCEED BY **
SECTION a-a			
LOAD CASE 1	0.5	49.8	
LOAD CASE 2 (1)	20.0	"	
LOAD CASE 3	17.6	"	
LOAD CASE 4	27.0	"	
LOAD CASE 5	21.8	49.8	
SECTION b-b			
LOAD CASE 1	16.3	114.6	
LOAD CASE 2(1)	71.8	"	
LOAD CASE 3	64.9	"	
LOAD CASE 4	91.2	"	
LOAD CASE 5	76.4	114.6	
SECTION c-c			
LOAD CASE 1	31.8	110.0	
LOAD CASE 2(1)	122.3	"	1.11
LOAD CASE 3	109.9	"	1.0
LOAD CASE 4	152.8	"	1.39
LOAD CASE 5	126.2	110.0	1.15

(1)- LOADING CONDITION AT THE TIME OF THE COLLAPSE

** - ACTUAL BENDING MOMENT EXCEEDS ALLOWABLE DESIGN MOMENT STRENGTH

DETERMINE REQUIRED LAP LENGTH @ SECTION b-b

LOADING CASE 4 , Mu =91.2 k'

$$\text{As (req'd)} = \frac{91.2}{4.25 \text{ X } 21.7} = 0.99 \text{ sq. in}$$

$$a = \frac{0.99 \text{ X } 60}{0.85 \text{ X } 5 \text{ X } 18} = 0.7765$$

ϕ Mn = O [0.99 X 60 x (21.7 -07765/2)]

= O [1266 k']

= O [105.5 k']

ϕ = 0.9 ϕ Mn = 94.9 k' $\cong$ 91.2 k '   OK

THEREFORE, As (req'd by analysis) = 0.99 sq. in

ACI 12.15

As (provided) = 1.2 sq. in   (2 - #701)

$$\frac{\text{As (provided)}}{\text{As (req'd)}} \leq 2$$

∴ CLASS-C SPLICE REQUIRED,

L (SPLICE) = 1.7 x ld

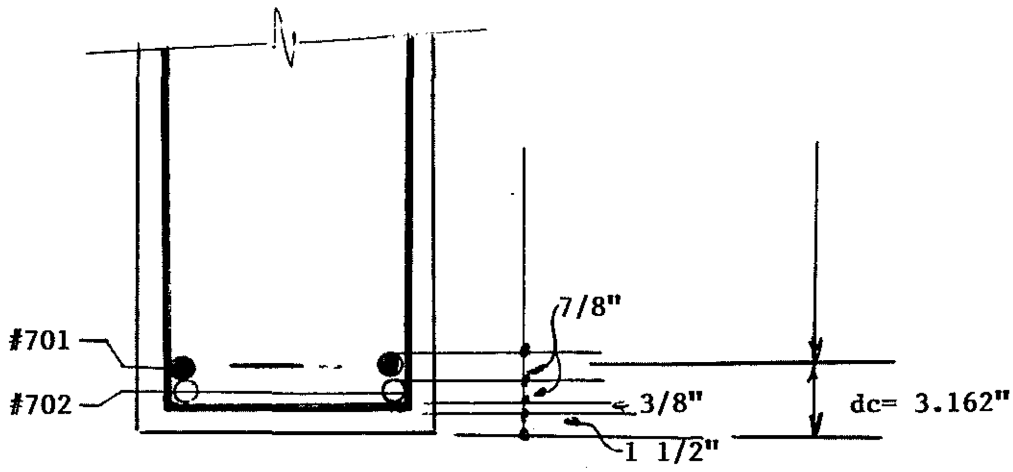
= 1.7 x (0.04 x Ab X fy / f'c $^{\frac{1}{2}}$)

= 1.2 x 21

$\cong$ 36"

ACI 10.6 CRACK CONTROL

SECTION c-c ; CONDITION OF REINF. @ COLLAPSE



$d_c = 1\ 1/2 + 3/8 + 7/8 + 7/16 = 3.162''$

1/2 #701

conc. cover #702 hairpin

stirrup

$$Ac = \frac{3.162 \times 18 \times 2}{2} = 56.92 \text{ sq in}$$

$$Z = 36000 \times \sqrt[3]{56.92 \times 3.162} = 203.2\ K/'' > 175\ K/'' \qquad \text{NG}$$

IF $d_c = 1\ 1/2 + 3/8 + 7/16 = 2.3125$

$Ac = 41.625$

$$Z = 36000 \times \sqrt[3]{41.625 \times 2.3125} = 165\ K/'' \qquad \text{OK}$$

THE AVILABLE FLEXURAL MOMENT STRENGTH OF RB-35
WITH 2-#7 REBARS AND A DEVELOPMENT LENGTH OF 0'-8"

$$T = As \times fy = \frac{2 \times 0.6 \times 60000psi}{21''} \times 8'' = 27{,}428\ \#$$

$$a = \frac{As \times fy}{0.85 \times f'c \times b} = \frac{27428}{0.85 \times 5000 \times 18} = 0.358$$

$$\phi\ Mn = \phi \times As \times Fy \times (d - a/2)$$

$$= \phi \times 27428 \times (20.8 - 0.358/2)$$

$$= \phi \times 565.6\ k''$$

$$= \phi \times 47.1\ K'$$

$\phi = 1.0$ ϕ Mn = 47.1 kip-ft

$\phi = 0.9$ ϕ Mu = 42.4 kip - ft

THEREFORE, WITHOUT THE MOMENT REDUCTION FACTOR

THE FLEXUAL MOMENT STRENGTH = 47.1 kip-ft

WITH THE MOMENT REDUCTION FACTOR OF 0.9

THE FLEXURAL MOMENT STRENGTH = 42.4 kip-ft

APPENDIX B

LABORATORY REPORT

8/28/90
10 50

Professional Service Industries, Inc.
Pittsburgh Testing Laboratory Division

FAX TELEPHONE NUMBER
412/922-4014

RECEIVED
AUG 28 1990
MELLON STUART
DICK ENTERPRISES
BP-07

COVER LETTER

PLEASE DELIVER THE FOLLOWING PAGES TO:

NAME: MR. JIM LONG

FIRM: MELLON-STUART-DICK ENTERPRISES

FAX NO.: 472-0393

FROM:

NAME: C. A. SHERMAN

FIRM/DEPT. CONSTR. SERVICES

TOTAL NUMBER OF PAGES 2 INCLUDING THIS PAGE

IF YOU DO NOT RECEIVE ALL THE PAGES PLEASE CALL US BACK AS SOON AS POSSIBLE AT 412/922-4000

OPERATOR: ____________________

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NOTES: ____________________

[illegible] • [illegible], PA [illegible] • Phone: 412/922-4000

Professional Service Industries, Inc.
Pittsburgh Testing Laboratory Division

812-00311-1
August 27, 1990

REPORT OF : TESTS OF CONCRETE CORES

REPORT FOR : MELLON STUART COMPANY
DICK ENTERPRISES
P. O. BOX 12332
PITTSBURGH, PA 15231

ATTENTION: MR. JIM LONG

Test Samples : 8 - 4" diameter concrete cores drilled from RB-35 beam at Airport

Core Locations : Located by Client

Tests Requested : 1) Compressive Strength
2) Splitting Tensile Strength

RESULTS - COMPRESSIVE STRENGTH

Core Number	Diameter (In.)	Capped Height (In.)	Area ($In.^2$)	Total Load (Lbs.)	L/D	P.S.I.
1A	3.98	8.02	12.44	73,500	—	5910
1B	3.98	7.99	12.44	71,000	—	5710
2	3.98	7.98	12.44	67,000	—	5390
3	3.98	8.01	12.44	68,500	—	5510
4	3.98	7.98	12.44	69,000	—	5550
5	3.98	8.00	12.44	64,000	—	5140
6	3.98	7.99	12.44	64,500	—	5180
7	3.98	7.98	12.44	68,000	—	5470
8	3.98	7.99	12.44	62,000	—	4980

RESULTS-SPLITTING TENSILE STRENGTH

Core Number	Diameter (In.)	Length (In.)	Total Load	P.S.I.
4	3.98	7.95	23,500	470
6	3.98	7.96	26,250	525
7	3.98	8.00	27,000	540

PROFESSIONAL SERVICE INDUSTRIES, INCORPORATED
PITTSBURGH TESTING LABORATORY DIVISION

CAS/mb
3-Mellon Stuart Company
Dick Enterprises

APPENDIX C

PRECAST CONCRETE ERECTION SEQUENCE GUIDELINES

HANNA, GHOBRIAL & ASSOCIATES LTD.
CONSULTING ENGINEERS

939 Goyeau St. Windsor, Ontario Canada N9A 1H7
(519) 253-1188 Fax: (519) 253-1242

April 9, 1990.

Anjo Construction Company,
4540 New Texas Road,
Plum Borough,
Pittsburgh, Pa. 15239

Attention: Mr. Norman Butler

RE: AIRSIDE BUILDING
GREATER INTERNATIONAL AIRPORT
PITTSBURGH, PA.

Dear Sir:

Erection recommendation for N.E. Arm

1) First floor columns have to be laterally braced until the roof's cross girders and arches are in place and rigid joints grouted.

2) Sequence of erection:

a) first floor columns
b) concourse beams
c) grouting of 11" x 11" holes or grout tubes between columns and beams
d) Installation of hollowcore and channel slabs
e) Installation of 2nd floor columns
f) Installation of field bars in the pour strip of concourse beams
g) Placing of pour strips concrete
h) Placing of roof beams
i) Grouting of 11" x 11" holes and grout tubes and rigid poured in place connections
j) Placing of roof hollowcore slabs
k) Placing of beams field top bars
l) Placing of pour strips concrete

Yours truly,
HANNA, GHOBRIAL & ASSOCIATES LTD.

MG/sw

Dr. M. Ghobrial, P.E.