Investigation of November 19, 1990 Excavation Collapse at 14th and H Streets, N.W. Washington, D.C.

U.S. Department of Labor
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U.S. Department of Labor
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Office of Construction and Engineering

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EXECUTIVE SUMMARY

An open excavation, 150 ft. x 208 ft. by 47 ft. deep for a 12-story office building with four levels of parking at 14th and H Streets, N.W., Washington, D.C., collapsed on November 19, 1990 at approximately 8:30 p.m. Construction had stopped for the day at the time of the accident. This accident therefore did not cause any injury or death, though it had significant potential for casualties.

Personnel from the Baltimore Area Office and the National Office of the Occupational Safety and Health Administration (OSHA), arrived at the scene on the evening of the accident. The Office of Construction and Engineering, OSHA, Washington, D.C., was requested to provide assistance in the technical assessment of the collapsed structure and in determining the cause of the accident.

The OSHA investigation began soon after the accident and included observation of the collapsed structure and geotechnical and structural analysis of the excavation support system. Based on the results of the investigation, OSHA concludes that:

1. The internal support system erected at the construction site, was not capable of resisting the soil load which could be reasonably expected. Certain members and their connections were not adequately designed to suit the field conditions for the expected loads.

2. The spacer beams (outlooker beams) between the wale and the soldier beams and their connections and the sloping wale were determined to be the most highly stressed members and connections of the support system and contributed to the collapse.

1. INTRODUCTION

On November 19, 1990, at approximately 8:30 p.m., the structural steel support system of a 47 foot deep open excavation site at 14th and H Streets, N.W., Washington, D.C., collapsed causing a cave-in of several thousand cubic yards of soil. The collapse caused a loss of the soldier beam and lagging wall in the immediate area of the collapse. The collapse caused the internal support system to slide and fall into the open excavation site. The edge of the internal support system slid 35 feet westward from the eastern perimeter. The sidewalks along 14th and H Streets were lost, as well as an alleyway between the United Press International (UPI) building and the excavation site. Due to the time of day, there were no fatalities or injuries in this collapse, although there was a potential for a significant number. Figures 1.1 and 1.2 are photographs taken of the site after the collapse.

Personnel from the Occupational Safety and Health Administration (OSHA), Baltimore Area Office, arrived at the scene within 1 hour after the collapse to collect evidence in the form of photographs and videotapes. The Director of the OSHA Office of Construction and Engineering and the Director of Field Programs, in Washington, D.C., were at the site within 4 hours of the accident to assess the general safety of the site and to ensure abatement of the hazard. The Office of Construction and Engineering was subsequently requested, by the Regional Administrator of OSHA Region III, to provide assistance to the Baltimore Area Office to evaluate the excavation support system in the investigation of the collapse. The staff of the Office of Construction and Engineering visited the site continuously during stabilization of the collapse and subsequent removal of the collapsed steel members, from November 20, 1990 through March 16, 1991. Structural steel members after being identified and flame cut, were moved to a site belonging to SMC Concrete Construction Company, Inc. (SMC), in Lorton, Virginia for storage and subsequent detailed examination.

The OSHA investigation included: (1) review of the original design drawings and calculations; (2) identification of all steel members of the internal support system at the site; (3) observation of the collapsed internal support system from the time of the initial collapse to the final recovery stage; (4) performing a geotechnical and structural analysis of the internal support system to determine the resisting capacity of the internal support system against the intended applied loads. The Office of Construction and Engineering worked together with the personnel from the OSHA Baltimore Area Office throughout the investigation.

The Area Director Gary Griess, the Safety Supervisor John Wiseman, and the Compliance Officer David Miller, made significant contributions to this investigation report.

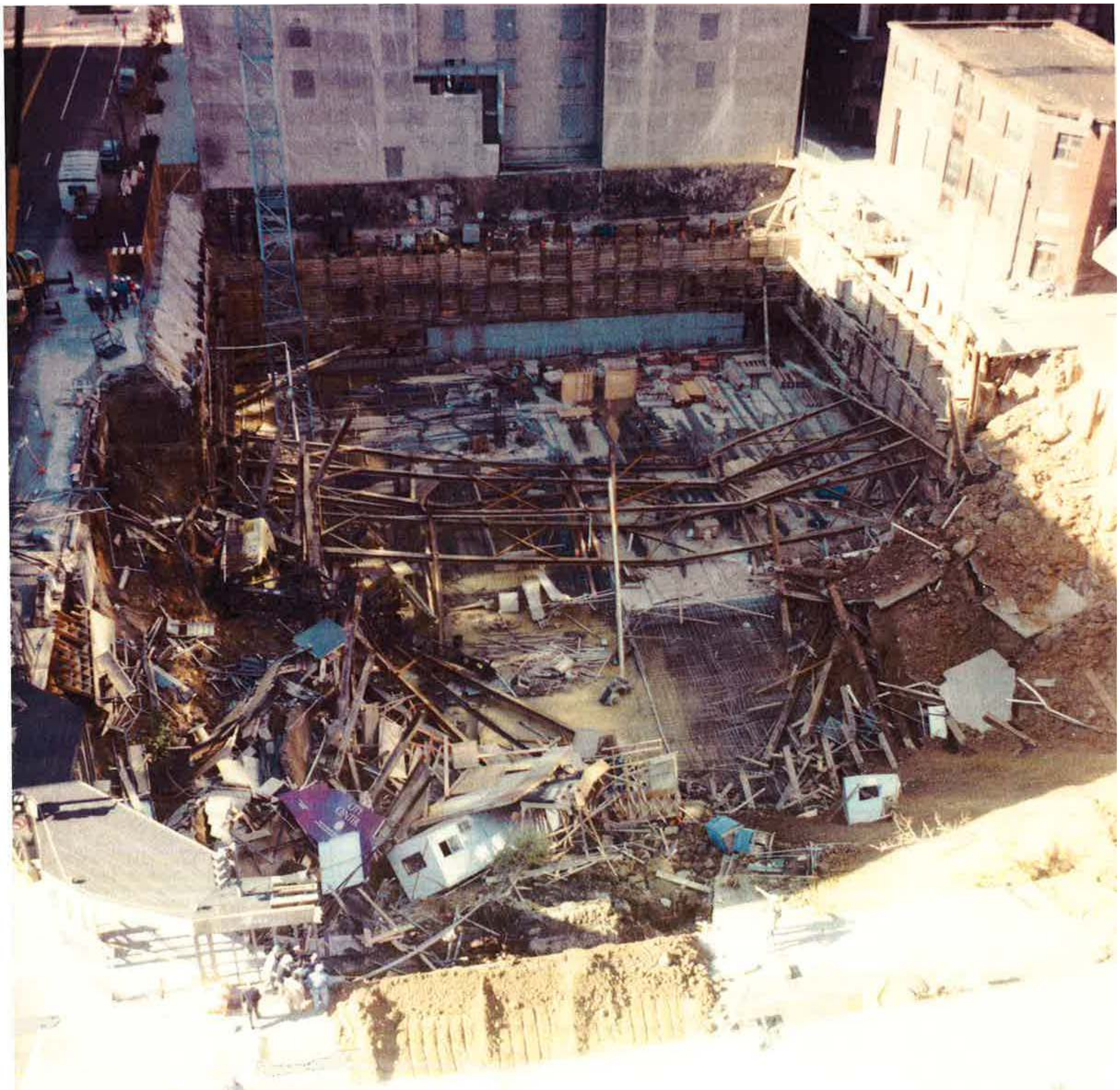


Figure 1.1. Overall View of the Collapsed Site (Looking Toward West)

Figure 1.2. Overall View of the Collapsed Site (Looking Toward East)

2. CONDUCT OF THE INVESTIGATION

At the request of the OSHA Baltimore Area Office, the general contractor immediately provided four drawings for the sheeting and shoring system and eighteen pages of calculations prepared by the shoring subcontractor for the project. These documents were then forwarded to the OSHA Office of Construction and Engineering in Washington, D.C. for review. Based on contract documents obtained by OSHA, the shoring subcontractor for the project was responsible for the design and construction of the excavation support system. The general contractor claimed that the excavation subcontractor for the project was responsible for any soil excavation during the stages after the excavation support system was secured in each stage. Then the concrete subcontractor for the project was to be responsible for preparing the formwork, placing the rebars, pouring the concrete, and removing the internal bracing system when the permanent concrete gained sufficient strength to resist the required lateral earth pressure.

On January 18, 1991, OSHA requested additional materials and information from the general contractor on the site's as-built conditions of the internal support system, soldier beam driving records, size and length of outlookers (a connecting member between soldier beam and wale) and its welding details, excavation schedule, site preloading records, surveying and instrumentation records, and other earth pressure measurement records. On February 13, 1991, OSHA received four pages of surveying information from the shoring subcontractor. Then on April 17, 1991, OSHA received a letter from the shoring subcontractor through the general contractor. This letter basically stated that the majority of the information OSHA requested was lost in the construction trailer with the excavation collapse and was not recovered in the re-excavation process. However, the shoring subcontractor of the project did offer the following clarifications on the information OSHA requested.

- Although the as-built drawings were lost, each individual piece of steel was labeled and numbered by the shoring subcontractor during removal and was inspected by OSHA personnel both at the site and at the SMC storage yard.

- The only additional design computations, beyond the 18 pages of calculations already submitted, were related to the replacement of some failed tiebacks with rakers on the third tier. These computations were lost in the construction trailer.

- The soldier beam driving records, excavation schedule and preloading records were lost in the construction trailer. However, the shoring subcontractor indicated that all of the struts and diagonals were jacked to one-half of the design load, except Nos. 6, 7, 10 and 11.
- No load cells or strain gauges were utilized in this project.

In the letter of April 17, 1991, welding details for outlooker to soldier beam and outlooker to wale were enclosed. It is OSHA's understanding of this letter that the company did not prepare design calculations for sizing the outlookers (spacers). Based on OSHA field observation, the lengths of the outlookers were determined by the installed locations of the soldier beam and wale beam, and the outlooker was welded perpendicular to the wale beam and then welded to the soldier beam.

Since the excavation collapse, the staff of the Office of Construction and Engineering constantly visited the site: (1) to investigate the failure pattern; (2) to witness the refilling and re-excavation processes; (3) to identify the steel members with the shoring subcontractor; (4) to take the necessary measurements; (5) to select the steel members to be stored for detailed examination; (5) to establish the actual internal support system before the collapse.

This office prepared an independent calculation for the lateral earth pressure. The calculation agreed with the original design loading used by the shoring subcontractor for the project. Manual computations were performed to determine the level of stress in installed structural elements and to verify their adequacy to resist the expected load. In addition, the strength of the soldier beam-outlooker-wale beam welded connections were calculated and the lateral earth pressure at the time of the collapse was estimated. Finally, computer analyses of the as-built internal support system were performed for each of the three tiers for several loading conditions to verify the adequacy to resist the expected lateral pressures. Members which were found to be overstressed were identified.

3. DESCRIPTION AND OBSERVATIONS OF COLLAPSE

In this chapter, the general observation of the collapse is described. Based on the order of steel members recovered the basic failure pattern is discussed. The dimensions of the recovered structural members are compared with those shown in the original design drawings prepared by the shoring subcontractor for the project. These dimensions were then used for the structural analysis in Chapter 4.

3.1 BACKGROUND

The construction site is located on the northwest corner of the intersection of 14th and H Streets in the northwest quadrant of Washington, D.C. The project under construction, called the City Center Office Building, is a 12-story reinforced concrete structure with four levels of underground parking. Figure 3.1 shows the orientation of the collapsed site.The size of this building and the excavation is 150 feet in the north-south direction, with 14th Street on the east side and the Southern Building on the west side and the excavation is 208 feet in the east-west direction, with United Press International (UPI) building and the Metro McPherson Square escalator entrance on the north side and H Street on the south side. The depth of excavation is generally 47 to 48 feet, depending on the adjacent ground surface elevation. At the location of elevators, foundations for tower cranes and heel blocks, there were excavations up to 53 feet in depth below the street level. However, the deeper excavations were quite a distance away from the perimeter of the main excavation, thus, had little influence on the collapse.

Due to the size of the excavation, an external support with wales and tiebacks was more economical than an internal support system with wales, struts, diagonals and rakers. However, the adjacent UPI building basement, the existing utility lines and the underground right-of-way permit constraints, prohibited the installation of tiebacks in certain locations. Thus, the west portion of the excavation was supported by a tieback system, while the east portion was supported by a steel bracing system. In addition, due to the proximity (13 feet) of the Southern Building on a shallow foundation in the west side of the excavation, this portion of the excavation was heavily instrumented with inclinometers and additional building settlement monitoring points. Based on the original design drawings, soldier beams 3 through 13 were supported by three tiers of tiebacks, soldier beams 14 though 57 were supported by three tiers of internal bracing, soldier beams 58 through 63 were supported by an internal bracing for the top two tiers and tiebacks for the third tier, solider beams 64 through 76 were supported by two tiers of tiebacks due to the top sloped cut, soldier beams 77 through 101 were supported by three tiers of tiebacks. Figure 3.2 shows the plan location of each soldier beam in the design stage. The establishment of the actual support system based on the recovered steel members is described in the subsequent section.

For the protection of adjacent building foundations and utilities, the support system was designed to maintain a prestress in the system and to minimize the stress change in the adjacent ground mass to limit its deformation until the permanent structural concrete gained sufficient strength to resist this prestress load. Thus, any tier of the internal support system cannot be removed before the adjacent concrete gains sufficient strength. In addition, to build the basement wall to coincide with the property line to obtain the maximum space, it is necessary to place the temporary soldier beam and lagging wall just outside the property line, and use this wall as one-half of the formwork for concrete pouring. To accomplish these two goals, sufficient clearance must be maintained between the soldier beam and wale, to accommodate the thicknesses of the perimeter wall and columns of the permanent basement, and to provide for removing the wale and cutting off the protruding outlookers from inside the basement wall when the concrete wall has gained adequate strength. For this project, the required clearance between the soldier beam and wale is about 3 feet, thus, the minimum length of the outlooker would be at least 3 feet.

3.2 OBSERVATIONS BEFORE REEXCAVATION

3.2.1 General Westward Sliding and Falling Pattern

As described in the earlier section, the excavation support system was not symmetrical in the east-west direction. The wales along the north and south walls did not extended all the way to the west wall of the excavation. For the top tier, the north and south wales were terminated about 30 feet and 35 feet, respectively, away from the west wall. For the middle and bottom tiers, all wales were terminated about 93 feet away from the west wall. The

November 19, 1990 collapse, sent the north top and middle wales about 37 feet and 20 feet westward, respectively, and both fell about 24 feet and 12 feet, respectively, as indicated in Figure 3.3. Similar sliding and falling also happened along the south wall, the south top and middle wales slid about 37 feet and 25 feet westward, respectively, and fell about 22 feet and 10 feet, respectively as indicated in Figure 3.4. Due to this westward sliding, the west end of the top tier north wale hit the lagging of the west wall before it broke away as indicated in Figure 3.5. The top tier south wale was sheared off at its sloped location between soldier beams 63 and 64. This is probably due to a very high westward force from the east wall during the collapse. As a result, the remainder of the top tier south wale, between soldier beams 64 to 71, was still attached on the south wall as indicated in Figure 3.4.

The three lines of struts in both top and middle tiers, although bent and twisted, appeared to travel with the wales due to their unbroken connections. A similar falling pattern was also observed for the pin pile frame, since most of the pin pile connections were still attached to the struts. Pin pile No. 6 was an exception, its welding connections had been broken from the main frame. The above observations were indicated in Figures 3.4 through 3.6.

As indicated in both Figure 3.5 and Figure 3.6, the soldier beams of both the north and south walls were bent westward and inward, the westward movement was probably due to the pulling force from the wales, while the inward movement was due to the earth pressure from the back. Based on the uncovered top of the east wall soldier beams, they were simply bent inward, that is westward, due to the earth pressure from the back. Although a portion of the top wales with a diagonal in the northeast corner was bent upward after the westward travel, this probably was due to secondary effects after the initial collapse.

3.2.2 Extent of Initial Collapse

As indicated in Figures 3.7 and 3.10, the initial collapse started along 14th Street and included the entire 20 foot width of the sidewalk for almost the entire 150 foot length of the excavation. The sliding of the 14th Street side of the internal support system then pulled away the internal support system that was holding up the alley on the north side and H Street on the south side. As a result, the entire 20 feet width of the alleyway for almost the entire 114 foot length of the internal support system caved in as indicated in Figure 3.8. Similarly, the entire 20 foot width of the sidewalk for a length of about 114 feet along H Street also caved in as indicated in Figure 3.9.

Construction equipment and other facilities in place above the sidewalk also fell into the open excavation. These included the construction trailers, two portable toilets, parts of the wooden sidewalk protection, a mobile crane, a city light post, three trees, a PEPCO transformer manhole, and a portion of a telephone duct. The extent of the initial collapse at the ground surface level in the plan is indicated in Figure 3.10.

3.2.3 Second Collapse

After the initial collapse, the general contractor started to refill the excavation along the 14th Street side, to stabilize the nearly vertical failure surface and to prevent further cave-in. The refilling operation was successful in preventing further cave-in along 14th Street. This operation was not sufficient along H Street. On November 20, about 1:40 p.m., 17 hours after the initial collapse, a 10 foot wide and 100 foot long section of H Street collapsed into the excavation site, along with remaining wooden sidewalk protection, as indicated in Figure 3.11.

The refilling operation was continued until all the collapsed surfaces were stabilized by a 1(horizontal) to 1(vertical) soil berm. This new soil slope was then covered by a plastic sheet for moisture protection. No further excavation collapses were observed.

3.2.4 Extent of Non-collapsed Wall

The soldier beam wall supported by the external tieback system did not fail as a result of the collapse, although a few tieback wales were damaged by the sliding and falling internal support system. Seven rakers were installed by the shoring subcontractor at the damaged tieback wale locations after the collapsed steel members of the internal support system in the excavation were secured. They were installed at the bottom tier of soldier beams 9, 10 and 11, and middle and bottom tiers of soldier beams 12 and 13, as indicated in Figure 3.12.

Soldier beams 14 and 15, supported by the internal bracing system, tilted into the excavation site 2.5 feet and 4 feet, respectively, at the ground surface elevation, but did not collapse. This condition is probably created by the soil arching between the unfailed tieback wall and the UPI building, and the existing telephone manhole, to reduce the lateral earth pressure on these two soldier beams. These two beams were secured by six rakers, four were installed immediately after the collapse, as indicated in Figure 3.12. The remaining two rakers for the third tier were installed during the reexcavation operation. Both beams were incorporated in the final reshoring system.

Soldier beam 63 was supported by the internal bracing system for the first two tiers, however, the lower portion of the beam was supported by a raker system as shown in Figure 3.13. This beam tilted, but did not collapse. Figure 3.14 shows that this vertical beam was supported at the ground level by a horizontal beam anchored in the H Street pavement.

3.2.5 Uncovered Steel Members

After the completion of the refilling operation, the installation of rakers at the damaged tieback locations, and securing the collapsed steel member; removal of the uncovered steel members was started by the shoring subcontractor on December 6, 1990. The removal operation was accomplished by flame cutting the steel members in pieces, starting from the perimeter of the collapsed system to reduce possible internal stresses. Each individual steel member was identified and numbered by the shoring subcontractor and was inspected by OSHA personnel. The entire wale for all three tiers and any connecting elements to the wale, solider beam-outlooker connections, three pinpile-strut connections, two pinpiles, and approximately five steel members with failure surfaces were saved. All these steel members were stored at the SMC Yard for detailed examination.

The following summarize the observations of the support system:

- Almost all outlookers, on the north and south walls were still attached to the soldier beam and were sheared off at the wale end, as indicated in Figures 3.15 and 3.16. The outlookers also showed a westward deformation as shown in Figures 3.17 and Figure 3.18.

- The south wall sloping wale, between soldier beams 63 and 64, was sheared off as indicated in Figures 3.19 and 3.20. While the remaining portion was still attached between soldier beams 64 to 71 as indicated in Figure 3.4.

- Most of the eight uncovered wale-strut connections were still intact as indicated in Figure 3.21, for the top wale between soldier beams 14 to 16. However, the top wale-strut connection at soldier beam 18 was cracked as indicated in Figure 3.22 and the middle tier wale-strut connection at soldier beam 16 fell apart when it was moved up to the ground surface.

- The top tier wale-diagonal connection at soldier beam 20 was separated, as indicated in Figure 3.23. This may be due to additional load on the diagonal during the initial collapse.

- The bottom wale-raker connection was separated as indicated in Figure 3.24. This may be due to the fact that the lower end of the raker is fixed in the heel block and the raker is not free to move with the sliding internal support system.

- A typical wale connection with a strut is presented in Figure 3.25. An outlooker had also been connected to the right hand side of the wale but was sheared off in the collapse. This connection was located on the south wall, middle tier, at soldier beam 63. As indicated in this figure, the right flange was bent inward on the compression side of the outlooker attachment location.

- The majority of splice joints along the six struts were in relatively good condition with only a few broken. For example, a splice joint in the middle tier strut near pinpile 7 was broken as indicated in Figure 3.26.

- Most of the pinpile frames and connections to the struts were badly bent and twisted as a result of sliding and falling, except pinpile 6 was separated from the pinpile frame as indicated in Figure 3.4. Almost all diagonal lacings were still connected to the struts.

- Based on the above observation, it appears that the wale-strut frame after it sheared off from the outside outlooks, slid and fell as a unit for both top and middle tiers.

3.3 <u>OBSERVATIONS AFTER REEXCAVATION</u>

Reshoring of the refilled open excavation started on December 6, 1990, by driving inclined (10 degree $\pm$) soldier beams and installing tiebacks as the re-excavation progressed. This work was done by the shoring subcontractor. The tops of the new soldier beams were about 20 feet outside the original excavation limits. Except for a portion of the north wall, the existing soldier piles for the UPI building construction were incorporated in the reshoring.

After starting the reexcavation, lagging, tieback installation and removal of the toppled mobile crane on top of the original soldier beams, the removal of the remaining steel members began on February 4, 1991. It should be noted that the refilling and reexcavation operations would cause additional movement, deformation and separation of steel members from the initial collapsed state. However, the general falling pattern and the order of stacking for the steel system was not changed as a result of these operations. In addition, if a steel connection was recovered still intact, it was intact after the initial collapse. The major findings through the entire reexcavation stage are listed in the following sections.

3.3.1 General Collapse Pattern

At the time of the initial collapse, there was a backhoe, a grade-all and a loader parked on top of the final subgrade near the mid portion of the collapsed south wall. After the initial collapse, the final falling pattern of the south wall was influenced by the existence of this equipment. As indicated in Figures 3.27 and 3.28, the soldier beams Nos. 59 through 52 and the associated wales did not fall all the way to the subgrade level due to the backhoe nearby. The relative location of the backhoe and the top tier wale is indicated in Figure 3.29. The soldier beams shown in Figures 3.27 and 3.28 were twisted and bent toward the west into the excavation.

Along the collapsed north wall, since there was no obstruction along the collapse path, the soldier beams fell directly to the subgrade on top of the associated wales and diagonals. Figure 3.30 shows the low portion of soldier beams Nos. 19 through 24. Again, those soldier beams were twisted and bent toward the west into the excavation.

For the east wall, except for two soldier beams at each end, all remaining 16 soldier beams fell as a unit to the subgrade, as indicated in Figure 3.31. The original lagging between soldier beams probably influenced the formation of this failure pattern. Also indicated in this figure, the soldier beams experienced very little twisting when they fell.

3.3.2 Embedment Conditions

All 13 pinpiles were soundly embedded and bent near the surface of the final subgrade. Figures 3.32 and 3.33 show the embedded conditions on pinpiles 2 and 10, respectively.

All 50 soldier beams were embedded, with no pullouts or kickouts, as indicated in Figure 3.34, for soldier beams 22 through 29. All soldier beams started bending and twisting at the level of final subgrade.

All five rakers were embedded in three heel blocks. All five rakers were attached to the heel blocks during the reexcavation process. No movement of heel blocks was observed.

3.3.3 Outlooker and Its Welding Conditions

All outlookers were sheared off from the north and south wales between soldier beams 14 to 28 and 49 to 63. About two-thirds of the total 89 outlookers were still attached to the soldier beams. It is believed that the remaining one-third were either broken off during the refilling and reexcavation process, or torn off during the initial collapse. The above described condition can be seen in Figures 3.27 through 3.30. However, different conditions were observed on the three east wall wales. In this case 50 percent of the outlookers (30 pieces) were separated from both soldier beam and wale, the remaining 25 percent (15 pieces) were attached to the soldier beams and the same percentage were attached to the wale.

This is probably because the east wall outlookers were under different stress conditions than the north wall and south wall outlookers.

Since the outlookers were not dimensioned in the original sheeting and shoring drawings, the measurement of the size and length of each outlooker and its welding conditions to the wale and soldier beam was one of the major activities during the field investigation. The outlooker size and length is summarized in Section 3.5. However, the welding condition on both ends of the outlookers is discussed herein.

OSHA received the welding detail for the outlookers to soldier beams and outlooker to wales on April 17, 1991. It indicated the weld lengths were 12 inches on top of the top flange, two welds each 5.8 inches long on top of the bottom flange, and 12 inches on one side of the web. The weld size is 5/16 inch. However, in the field there was basically no welding on the web, except where HP14x73 buildup outlookers on the middle tier were connected to a W18x106. The length of the flange weld was the same as the width of the flange, for example, a W12x53 outlooker would have a 10 inch long weld, on top of a 10-inch wide flange. Figure 3.35 shows the outlooker wale welding condition on the bottom north wale at soldier beam 14 location. Figure 3.36 presents the outlooker soldier beam welding condition on soldier beam 63 at the top tier.

It was observed that the majority of the failures of the outlooker-wale and outlooker-soldier beam connections occurred through the fillet welds rather than in the base metal of the structural members.

3.3.4 Construction Deficiencies

In the course of the reexcavation and recovery of steel members, the following construction deficiencies were observed.

- The soldier beams were significantly twisted during driving, this condition was more pronounced in the northeast corner, where a gravel layer was encountered. Figure 3.37 shows that soldier beam 30 was twisted about 90 degrees at the third tier, so that the outlooker was welded to the web of the beam. Subsequently, a support beam was added to house the lagging. Soldier beams along the south wall were also tilted away from the excavation limit as indicated by the actual length of the associated outlookers.

- The lower flanges of the wales were cut, probably for seating the wall on misaligned brackets, before welding of the outlookers. Figure 3.38 shows the middle north wale at soldier beam 18 was undercut in both flanges, the size of the cut was 15 inches long by 4 inches wide. This cut will reduce the bending strength by about 25 percent.

- Quite a few prestress diagonal-wale connections were separated during the recovery stage. Figure 3.39 shows the top tier diagonal 4 at soldier beam 20 location. It appears that those welded connections including shim plates did not have sufficient strength to resist the shear force. The diagonal prestress connection should be located within the diagonal as proposed in the original design drawing.

- In the design drawings, all 50 soldier beams related to the collapse were listed as size HP14x73. However, the installed soldier beams 34, 36, 37, 38, 39, 40, 43 and 46 were size HP12x74. Although the HP12x74 weighs more, the bending resistance is 12 percent less and passive soil resistance is 14 percent less than the case with HP14x73 members.

3.3.5 Completion of Member Recovery Work

The recovery work was completed on Saturday, March 16, 1991, at about 3:00 p.m. At the time of completion, there were only concrete rebars and wet soil left on top of the final subgrade. No more original steel members related to the collapse were left in the excavation.

3.4 DESCRIPTION OF FAILURE PATTERN

The general description of the sliding and falling pattern of the west uncovered portion of the internal support system was included in Section 3.2.1. The general discussion of the falling pattern of the covered portion of the system was contained in Section 3.3.1. In this section, the discussion is concentrated on the east wall and the stacking order of the soldier beams in the northeast and southeast corners.

There were 20 soldier beams installed along the east wall of the excavation before the collapse, numbered 29 through 48. As described earlier, soldier beams 31 through 46 fell as a unit to the subgrade. The recovered position of these soldier beams were nearly parallel. This may be due to the original lagging between the beams which served as a constraint to prevent the soldier beams from free falling. The lateral displacement of the top of these soldier beams was estimated about 4 feet towards the north. This is probably due to a slightly higher actual lateral earth pressure from the south wall. The middle and bottom tier outlookers at soldier beams 41, 42, 43, 44 and 46 were still attached to both soldier beam and wale as indicated in Figure 3.40, for the middle outlooker at soldier beam 44. This also indicated that the middle and bottom tier wales on the east wall did not have much north-south movement, except for falling westward.

At the beginning of the steel removal process, the shoring subcontractor established a numbering system based on the order in which the steel pieces were flame cut. Therefore, the order of stacking can be derived from the numbering system. There were two exceptions to this rule, that is, for soldier beams 27 and 28 in the northeast corner of the north wall. At the time of cutting, these two beams could not be positively identified. The sections of these two beams were then numbered when their bottom locations were exposed.

For the southeast corner, based on the order of recovery, soldier beam 49 should be on the very top, the next layer would be soldier beams 50 through 53 from the south wall, the bottom layer would be soldier beams 48 to 45 from the east wall. The rest of the soldier beams were not related. Figure 3.41 shows the bottom location of soldier beams 44 through 53. Note in this picture soldier beam 48 was pulled up due to excavation. Figure 3.42 presents a simplified plan location of these 10 solider beams.

For the northeast corner, based on the general observation and the order of recovery, soldier beam 28 was on the very top and soldier beam 27 was next. Soldier beams 24 through 26 from the north wall were the third layer. The fourth layer was soldier beam 29. The bottom layer was soldier beams 30 through 33. Figure 3.43 shows the top location of soldier beams 29 through 31. Figure 3.44 presents a simplified plan location of these 10 soldier beams.

Based on the above information, it is evident that the east wall failed first.

3.5 ESTABLISHMENT OF AS-BUILT INTERNAL SUPPORT SYSTEM

One of the main purposes of the field observation was to confirm the type of support system and member sizes installed in the field and to compare these with the original design drawings, since as-built drawings were not available. For example, the size and length of outlookers was not specified in the original design drawings, thus, every outlooker, when recovered, was measured. Based on the OSHA field observation and measurement, the major modifications to the original design drawings are listed as follows. Most of the modifications were verified by the shoring subcontractor in letters dated April 17, 1991 and May 1, 1991.

- Based on the examination of the condition of the wale-strut and wale-diagonal brace connections, it was found that all struts and diagonal braces were preloaded, except for diagonal braces Nos. 6, 7, 10 and 11. This is based on the fact that preloaded connections should have a jacking plate on the strut or diagonal brace and shimming plates in the connections.

- On the south wall third tier, six tiebacks, as indicated in the original design drawings were replaced by three rakers. However, since the wale was terminated at solider beam 62, soldier beam 63 and the third raker on this beam was not considered as part of the collapsed internal support system.

- Based on the field measurements, the size of wale between soldier beams Nos. 37 and 40, for all three tiers, had been modified from the designed W24x146 or W24x162 to W27x146. The size of the first tier wale between soldier beams Nos. 40 and 59 was increased from the design W14x159 to W14x193. The size of the second tier wale between soldier beams Nos. 14 and 21 was increased from the designed W18x106 to W14x159. The size of the pinpiles Nos. 9 through 13 was increased from the designed HP8x36 to HP10x42. The size of soldier beams Nos. 34, 36, 37, 38, 39, 40, 43 and 46 was modified from the designed HP14x73 to HP12x74.

All the above information was incorporated in the as-built internal support system used for the analysis in Chapter 4. The installed internal support system for the top, middle and bottom tiers is shown in Figures 3.45, 3.46 and 3.47, respectively.

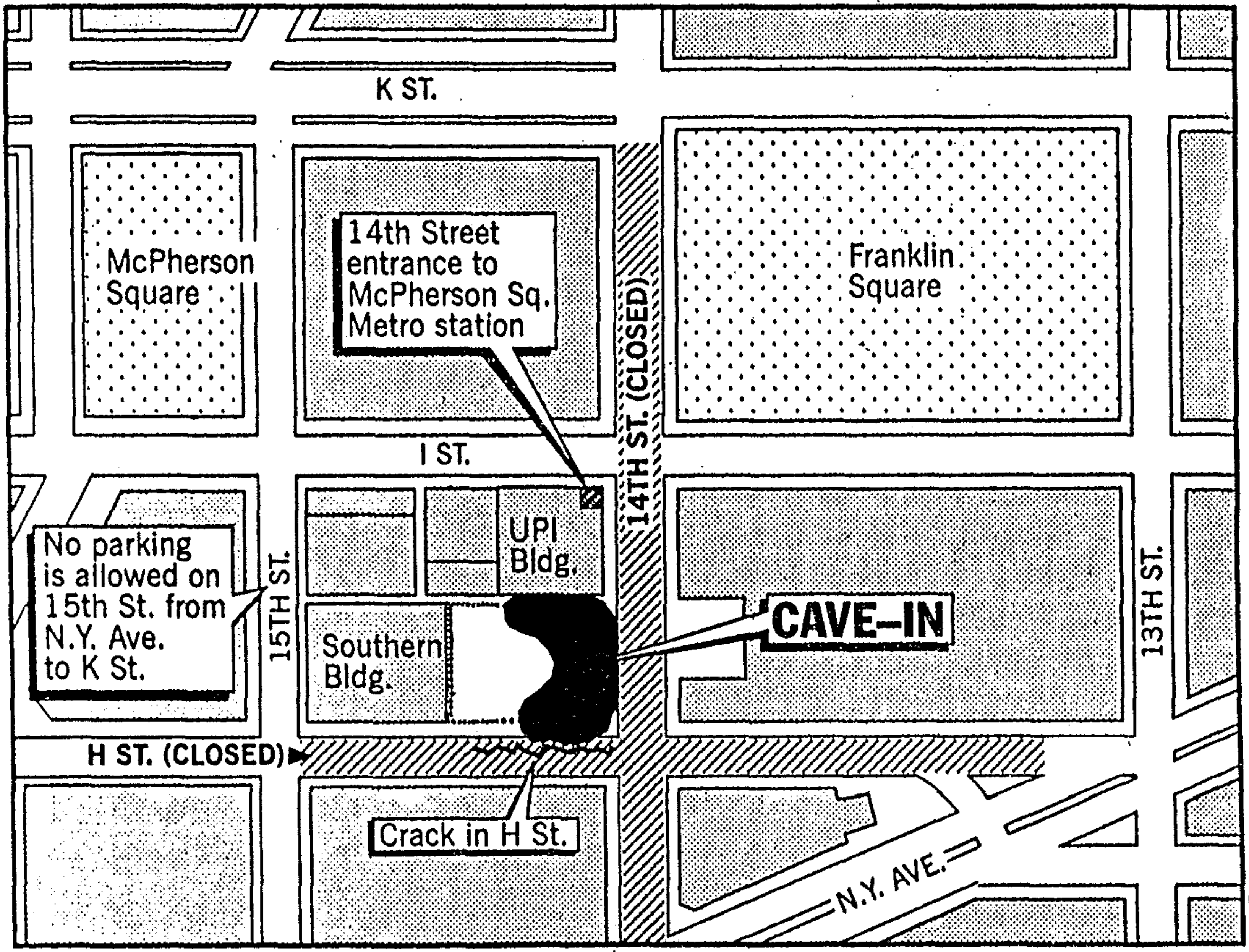


Figure 3.1 Orientation of the Collapse Site (Modified from Washington Post, 11/21/90).

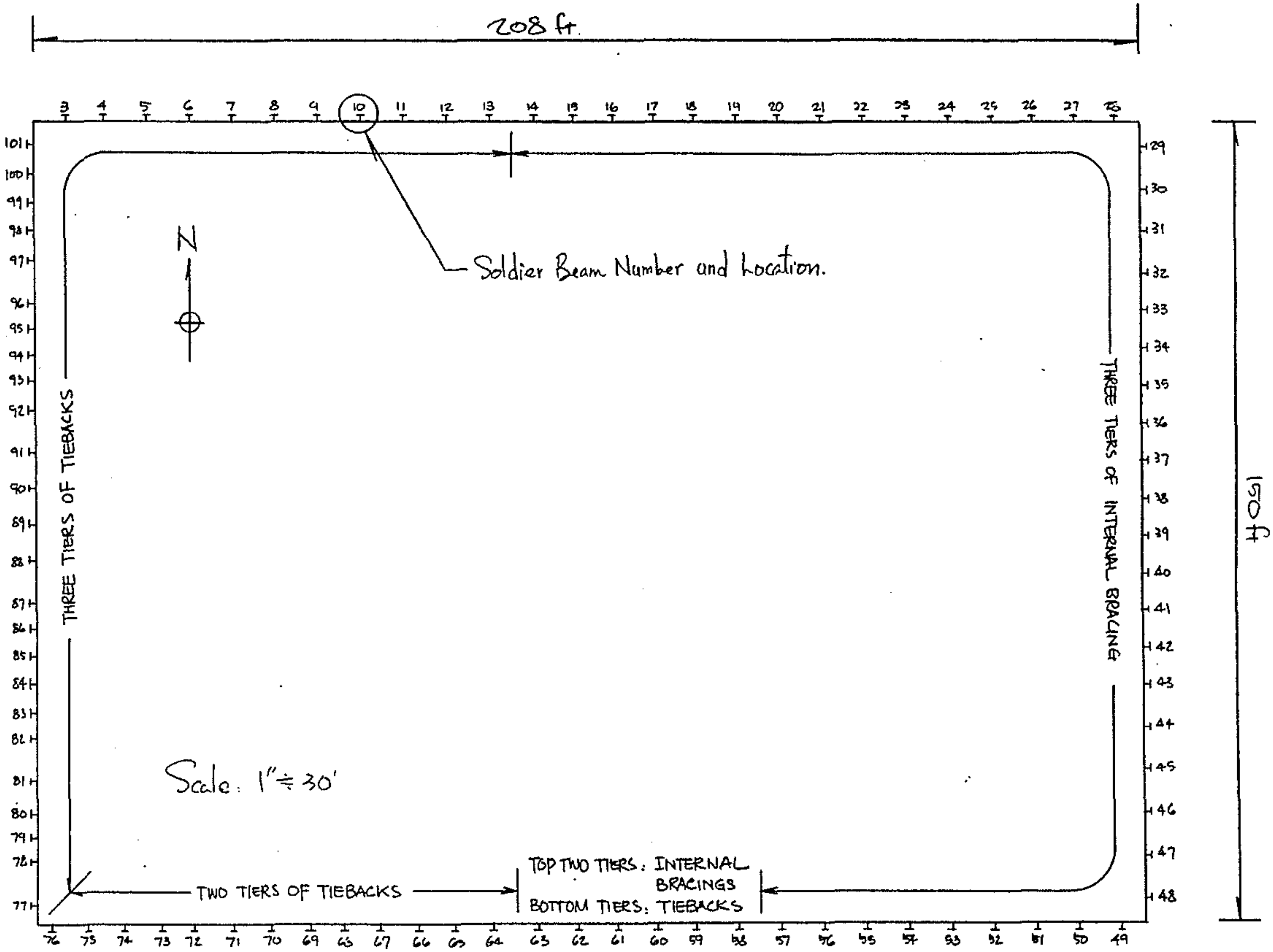


Figure 3.2 Plan Location of Soldier Beams in the Original Design Plan

Figure 3.3 Sliding and Falling of North Wall Wales (Looking Toward North).

Top tier south wale, between soldier beams 64 to 71, was still attached.

Pinpile 6 separated from the main frame.

Figure 3.4 Sliding and Falling of South Wall Wale (Looking Toward South).

The broken piece of top tier north wale.

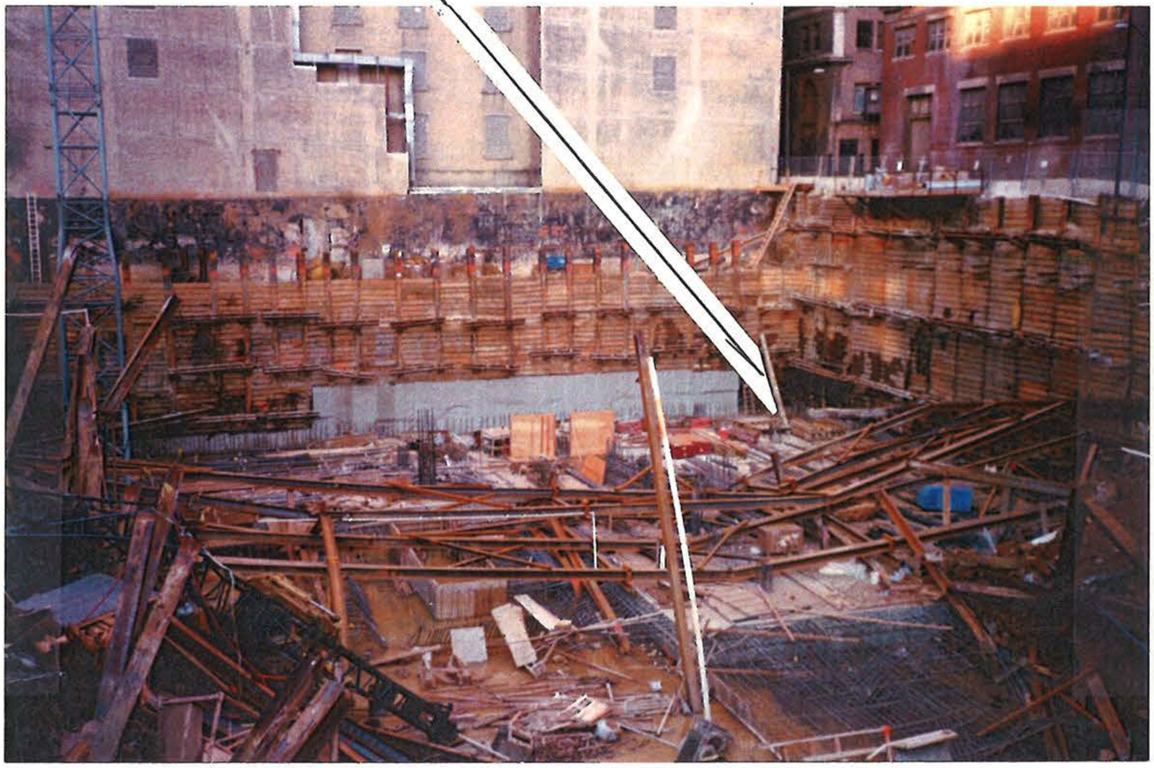


Figure 3.5 Three lines of Struts Traveled with Wales (Looking Toward West).

Figure 3.6 Three lines of Struts Traveled with Wales (Looking Toward Northwest).

Figure 3.7 Failure Soil Surface along the East Wall after the Initial Collpase (Looking Toward East). Note: Most of the failure surface is covered by fill material for the slope stabilization.

Figure 3.8 Failure Soil Surface along the North Wall after the Initial Collapse (Looking Toward Northwest).

Figure 3.9 Failure Soil Surface along the South Wall after the Initial Collapse (Looking Toward Southwest).

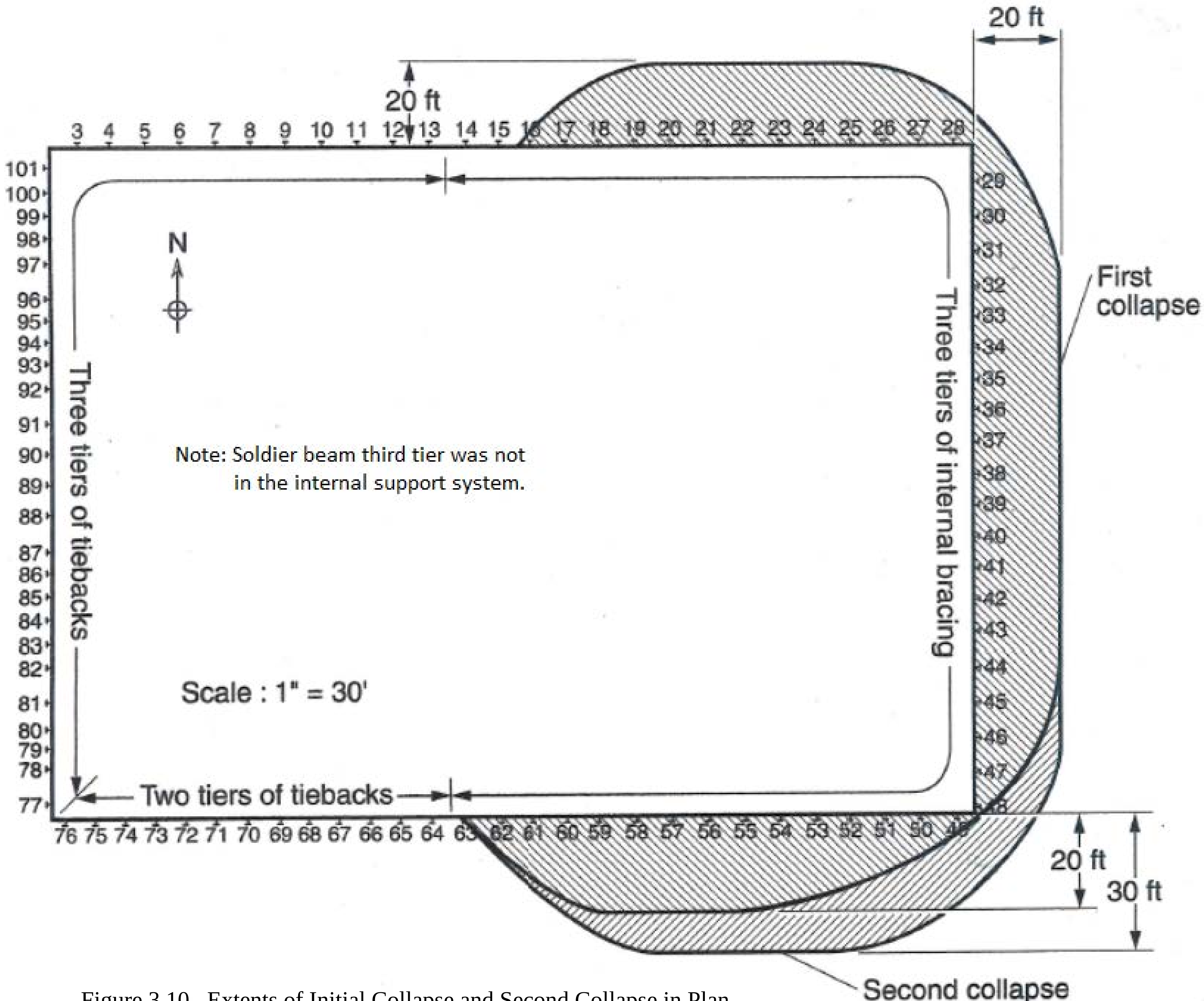


Figure 3.10 Extents of Initial Collapse and Second Collapse in Plan

Figure 3.11 Failure Soil Surface along the South Wall after the Second Collapse (Looking Toward South).

Figure 3.12 Seven Rakers Installed to Secure the Damaged North Tieback Wall (Looking Toward North).

Figure 3.13 A Raker System Supporting the Bottom Portion of Soldier Beam 63 (Looking Toward West).

Figure 3.14 An Anchored Horizontal Beam Securing the Top Portion of Soldier Beam 63 (Looking Toward East).

Figure 3.15 Outlookers Still Attached to the Soldier Beam after the Initial Collapse (Looking Upward North).

Figure 3.16 Outlookers Still Attached to the Soldier Beam after the Initial Collapse (Looking Upward North).

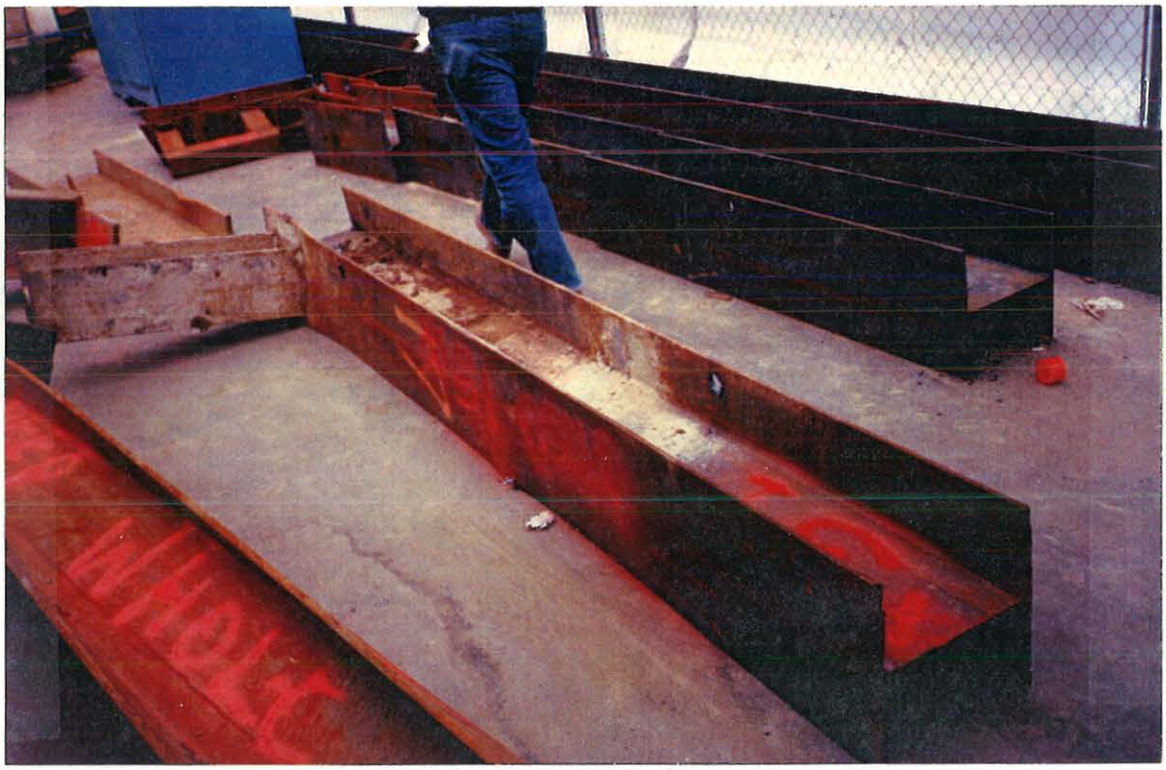


Figure 3.17 Outlooker of Soldier Beam 17 Top Tier Showed a Westward Deformation (Looking Toward the Bottom of the Soldier Beam).

Figure 3.18 Outlooker of Soldier Beam 60 Top Tier Showed a Westward Deformation (Looking Toward the Top of the Soldier Beam).

Figure 3.19 Sheared Off Section of the Sloping Wale (looking Toward East End of the Wale).

Figure 3.20 Sheared Off Section of the Sloping Wale (Looking Toward West End of the Wale).

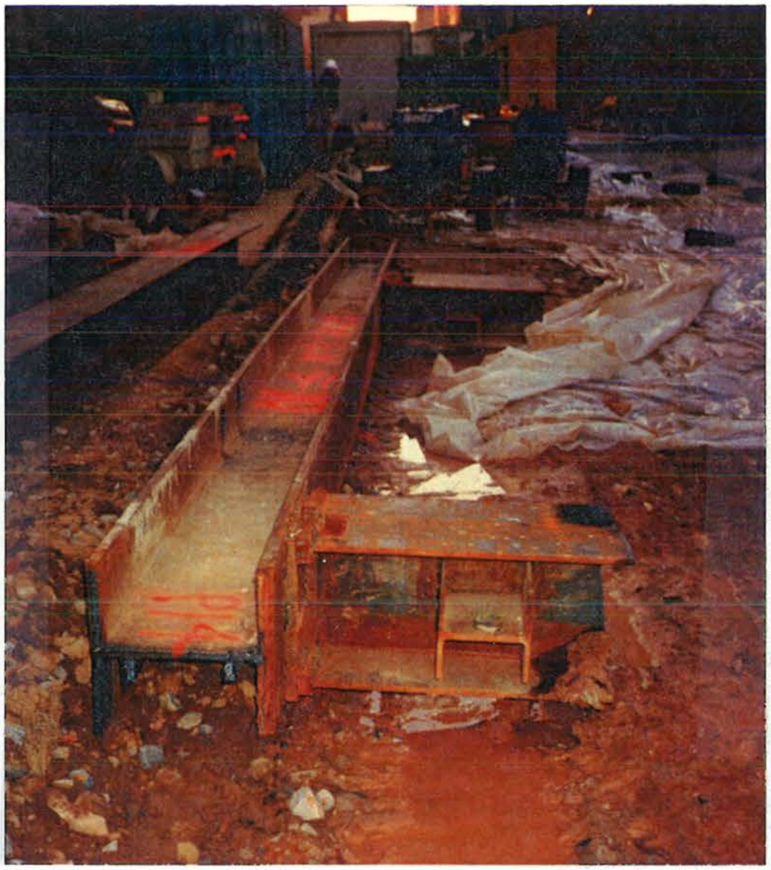


Figure 3.21 Intact Top Wale-Strut Connections with Preloading Plates and Shimming Plates, Wale Between Soldier Beams 14 to 16.

Figure 3.22 Cracked Top Wale-Strut Connection at Soldier Beam 18.

Figure 3.23 Separated Top Tier Wale-Diagonal Connection at Soldier Beam 20.

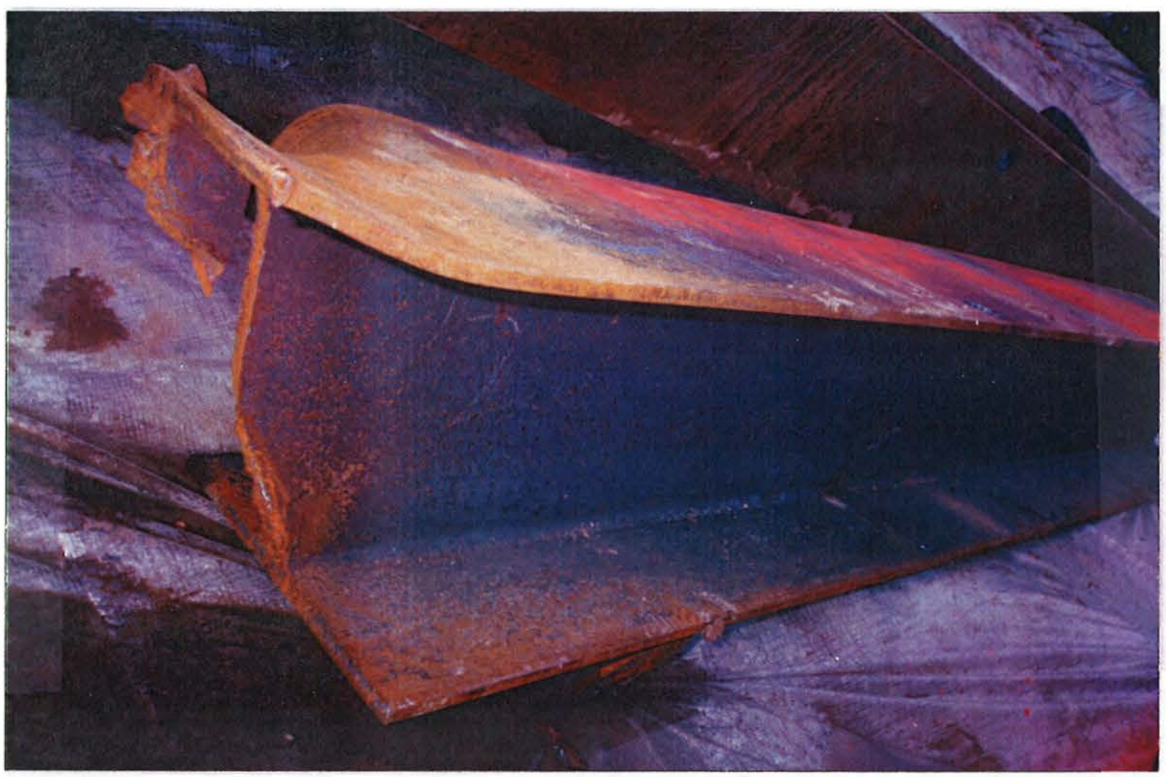


Figure 3.24 Separated Bottom Wale-Raker Connection at Soldier Beam 14.

Figure 3.25 Middle Wale-Strut Connection at Soldier Beam 63.

Figure 3.26 Splicing Joint in the Middle Strut near Pinpile 7.

Figure 3.27 Recovered Position of South Wall Soldier Beams 59 through 55 (Looking Toward North).

Figure 3.28 Recovered Position of South Wall Soldier Beams 56 through 52 (Looking Toward North).

Figure 3.29 Relative Position of the Backhoe and the South Wall Top Wale (Looking Downward). Note: No outlookers were attached to the wale.

Figure 3.30 Recovered Position of North Wall Soldier Beams 19 through 24 (Looking Toward East).

Figure 3.31 Recovered Position of East Wall Soldier Beams 45 through 39 (Looking Toward North).

Figure 3.32 Embedment Condition of Pinpile 2 (Looking Toward South).

Figure 3.33 Embedment Condition of Pinpile 10 (Looking Toward Southwest).

Figure 3.34 Embedment Condition of Soldier Beams 22 through 29 (Looking Toward East).

Figure 3.35 Outlooker Wale Welding Condition on the Bottom North Wale at Soldier Beam 14.

Figure 3.36 Outlooker Soldier Beam Welding Condition on Soldier Beam 63 at the Top Tier.

Figure 3.37 Bottom Outlooder Welded to the Web of Twisted Soldier Beam 30.

Figure 3.38 Middle North Wale at Soldier Beam 18 Undercut in Both Flanges.

Figure 3.39 Separated Prestressed Top Diagonal-Wale Connection at Soldier Beam 20.

Figure 3.40 Middle Outlooker at Soldier Beam 44 Still Attached to Both Soldier Beam and Wale (Looking Toward North).

Figure 3.41 Bottom Location of Soldier Beams 46 through 51 (Looking Downward).

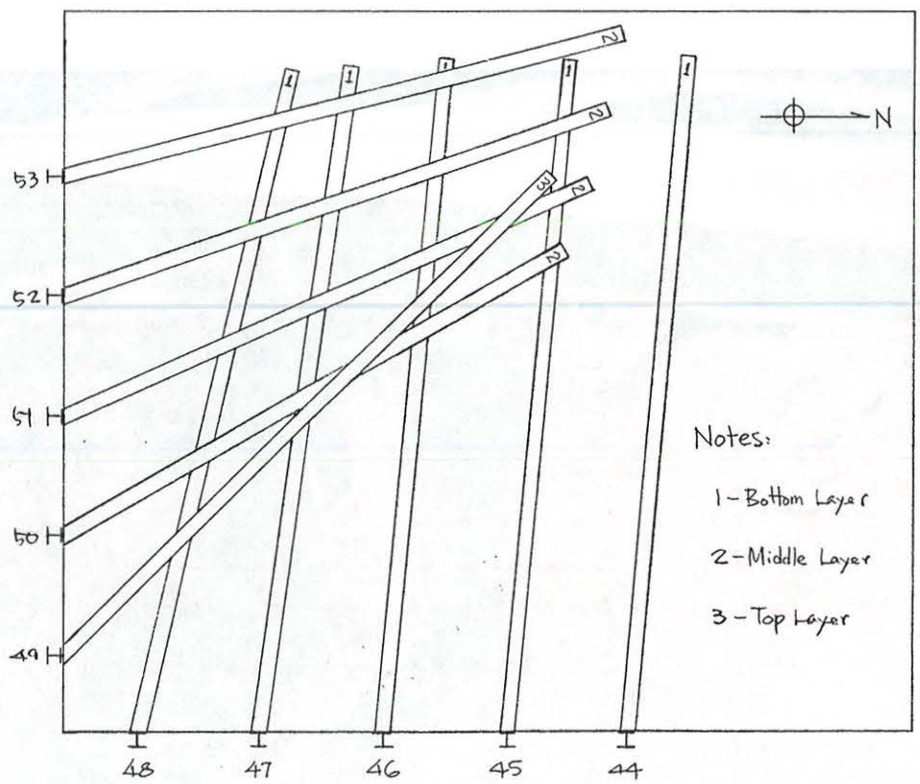


Figure 3.42 Approximate Falling Pattern of Soldier Beams in the Southeast Corner of the Excavation in Plan.

Figure 3.43 Top Location of Soldier Beams 29 through 31 (Looking Toward Southwest).

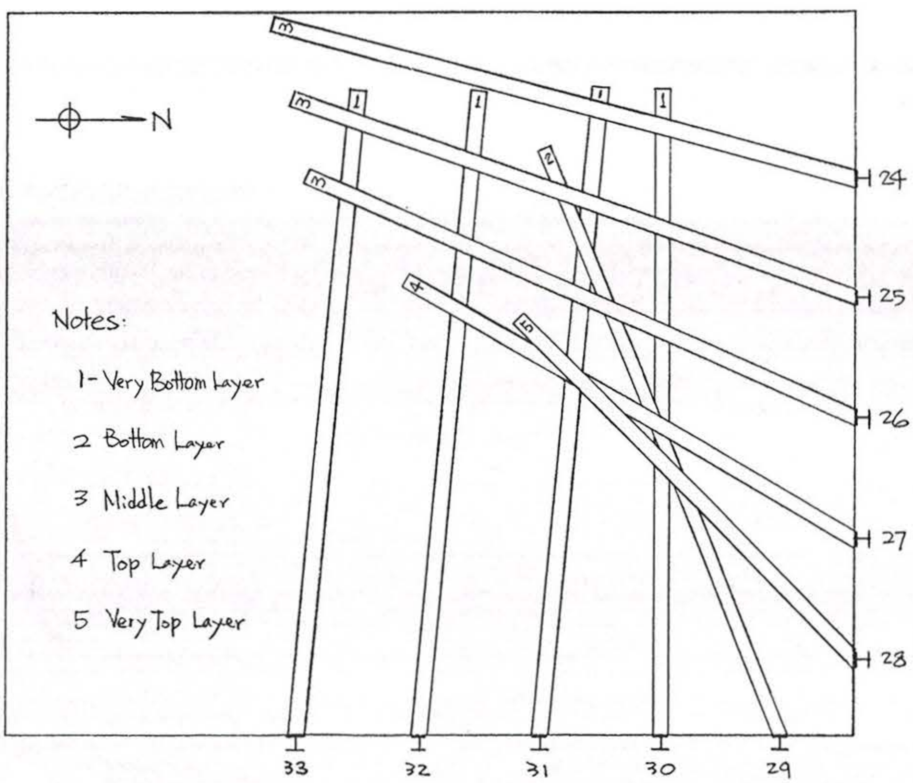
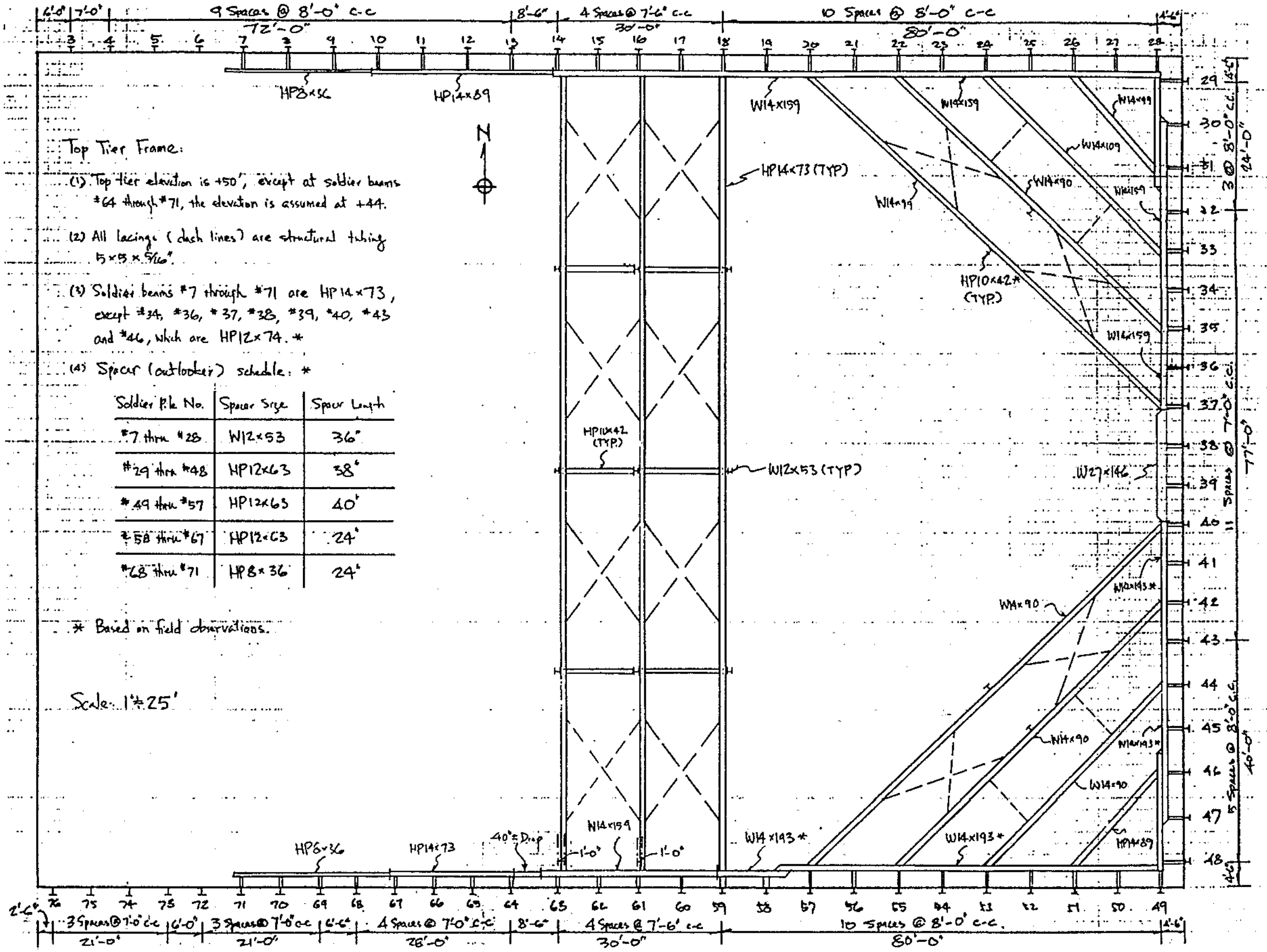


Figure 3.44 Approximate Falling Pattern of Soldier Beams in the Northeast Corner of the Excavation in Plan.



Soldier Pile No.	Spacer Size	Spacer Length
#7 thru #28	W12×53	36'
#29 thru #48	HP12×63	38'
#49 thru #57	HP12×63	40'
#58 thru #67	HP12×63	24'
#68 thru #71	HP8×36	24'

Figure 3.45 As-built Condition of the Top Tier Internal Support System.

Middle Tier Frame.

(1) Middle tier elevation is + 38'.

(2) All lacings (dash lines) are structural tubing 5×5×5/16"

(3) Soldier piles #14 through #63 are HP14×73, except #34, #36, #37, #38, #39, #40, #43 and #46, which are HP12×74.*

(4) Spacer (out bolur) schedule: *

Soldier Pile No.	Spacer Size	Spacer Length
#14 thru #21	HP12×63	36"
#22 thru #28	HP14×73	36'
#29 thru #48	HP14×73	38'
#49 thru #63	HP14×73	40'

* Based on field observations.

Scale: 1" = 25'

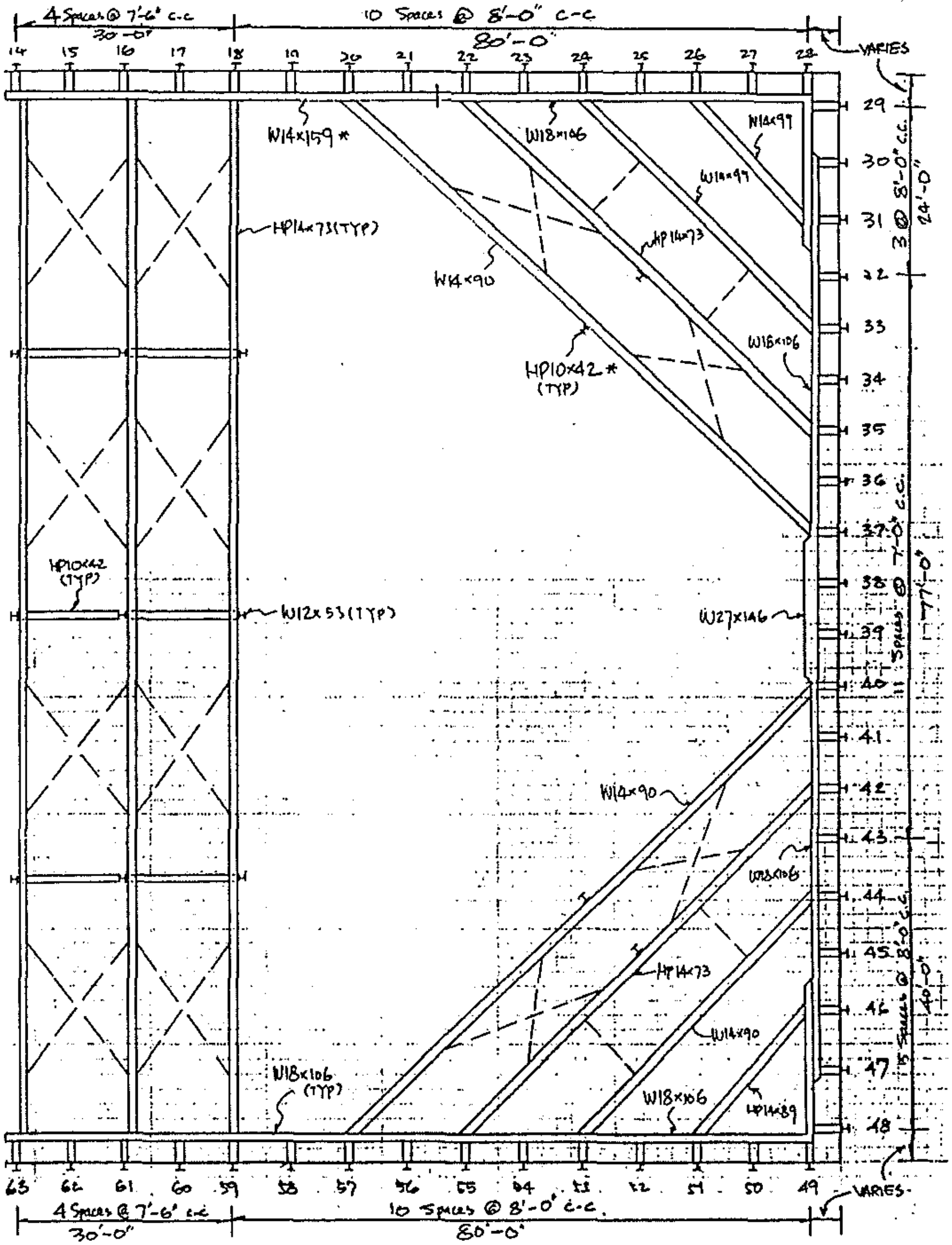


Figure 3.46 As-built Condition of the Middle Tier Internal Support System.

Bottom Tier Frame:

(1) Bottom Tier elevation is +26.5'.

(2) All lacings (dash lines) are structural tubing 5×5×5/16".

(3) Soldier beams #14 through #62 are HP14×73, except #34, #36, #37, #38, #39, #40, #43 and #46, which are HP12×74. *

(4) Spacer (outlooker) schedule: *

Soldier Pile No.	Spacer Size	Spacer Length
#14 thru #28	HP12×63	38"
#29 thru #48	HP12×53	36"
#49 thru #62	HP12×63	44"

* Based on field observations.

Scale: 1" ≐ 25'

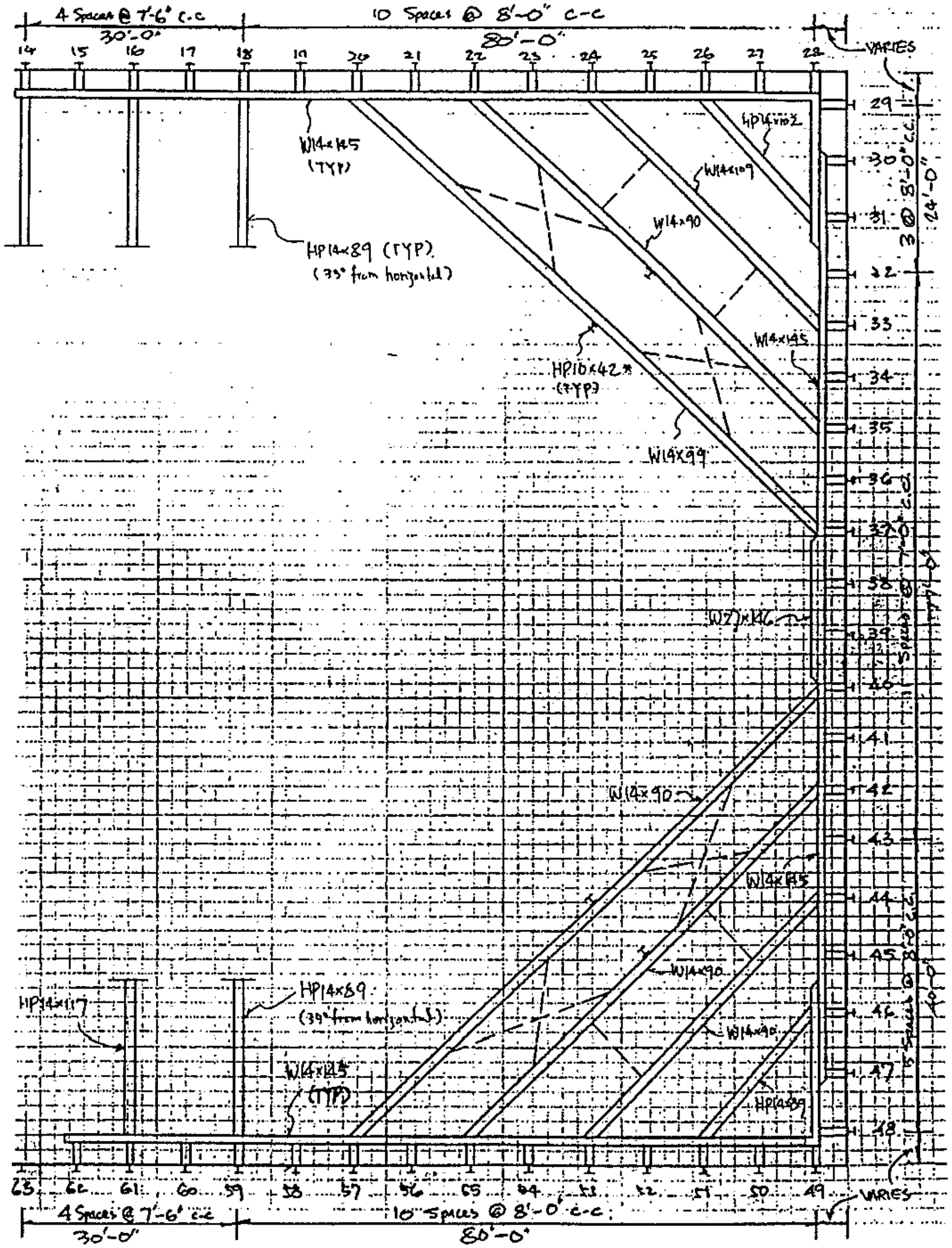


Figure 3.47 As-built Condition of the Bottom Tier Internal Support System.

4. GEOTECHNICAL AND STRUCTURAL ANALYSIS OF THE INTERNAL SUPPORT SYSTEM

4.1 GEOTECHNICAL ANALYSIS

In this section, based on the review of the project geotechnical report and its boring logs, the control geotechnical parameters are selected and the lateral earth pressure for the structural analysis is computed. Based on Reference 2, the earth resistance tangential to the excavation wall is calculated, the soil spring constants are computed, and the failure loads, based on the observed failure wedge size, were estimated.

4.1.1 Horizontal Earth Pressure

Based on the geotechnical report of the project and its boring logs, the excavation and the surrounding area consists of an average of 7 feet of fill material on top of 40 feet of interbedded strata of B1, B2 and a little of B3. B1 is a lean clay layer, B2 is a sand layer, and B3 is a gravel layer. The groundwater table was at El. 20 (39 feet below street level) in 1987, the year the exploration drilling was completed. The overall average blow count for all six borings within the excavation depth is 12 blows per foot.

Based on the above information, the following geotechnical parameters are assumed for the computation of lateral earth pressure:

ϕ = 30° (angle of internal friction)
c = 0 (cohesion)
γ_{moist} = 120 pcf = 0.12 kcf (density of moist soil)

The groundwater pressure is not included in the computation for the soldier beam and lagging wall because spaces between the lagging preclude a buildup of hydrostatic pressure. A lateral pressure based on an assumed traffic and construction equipment surcharge of 600 psf is included. For a multi-level internal and external support system, a trapezoidal shaped pressure diagram is used. Based on Reference 2, a multiplying factor of 1.1 is applied to the total active pressure where some horizontal movements is tolerable in noncritical areas and a factor of 1.3 is applied to minimize the horizontal movements in critical areas.

The calculated pressure is 0.030H for a noncritical area and 0.035H for the critical area. The sketches and detailed computations are included in Appendix A1. These pressures agree with the design load used by the shoring sub contractor for the project.

4.1.2 Earth Resistance Tangential to the Excavation Wall

The passive soil pressure perpendicular (normal) to the excavation wall usually is considered effective, in particular below the final subgrade level. However, the passive soil pressure tangential to the excavation, as assumed by the shoring subcontractor may not be a realistic resistance. This is due to the fact that in the direction of the passive failure wedge of the first soldier beam, there will be ten more solider beams ahead using the same soil weight as part of their failure wedge. In reality, a shear failure plane will be developed along the face of the inside flange of the soldier beams in the soil mass before the soil resistance can reach the assumed passive soil pressure level. Therefore, the frictional resistance along the shear failure plane is the basis for estimation of the earth resistance tangential to the excavation wall.

Based on the sketches and computations in Appendix A2, the total frictional resistance is about 147 kips per soldier beam. The shoring subcontractor overestimated this resistance by a factor of 2.43 in his calculations. Note that the frictional resisting force discussed herein is for failure analysis only and should not be considered in the design of the excavation support system.

4.1.3 Geotechnical Parameters for Computer Analysis

For the computer analysis of the internal support system, in addition to the frame dimensions, member sizes and horizontal earth pressures, soil spring constants are also required to model the soil-structure interaction. These constants are derived as follows:

With the excavation depth from the ground surface to 47 feet (El.59 to El.12) the soil strata are classified as fill material and terrace deposit, based on the geotechnical report on the project. Based on Reference 2 the modulus of subgrade reaction is 75 kcf for fill material and 150 kcf for terrace deposit.

Since there is only 7 feet of fill, a modulus of 150 kcf is assumed for the entire depth of excavation.

From the computations included in Appendix A3, a longitudinal spring constant of 110 kips per inch was obtained. The value of this constant was also checked with the soil deformation after preloading of the struts.

The tangential spring constant is calculated, based on the ratio of the passive soil pressure perpendicular to the soldier beam and the frictional soil resistance tangential to the flange of the soldier beam. A value of 16 kips per inch was assigned for the tangential spring constant.

The variation of the loading distribution in the structural frame, i.e., the shear forces on the outlookers is not that sensitive to the variation of the spring constants. For example, in increasing the tangential spring constant by 100 percent, the corresponding variation of the shear force on the outlookers is about 9 percent.

4.1.4 Estimation of Failure Loads

Two failure conditions were evaluated. The first condition was based on the extent of the initial collapse at the ground surface and a minimum size of a failure wedge from the internal friction angle of the caving soil to estimate the minimum failure load. The second failure condition was based on the extent of the initial collapse at the ground surface and the interpretation of pictures taken immediately after the initial failure, to estimate the most probable failure load.

4.1.4.1 Minimum Failure Load

Based on all accounts, the initial collapse brought the entire 20-foot wide sidewalk along the 14th Street into the excavation. Only one section of the granite curb was left in its original position, this granite piece was supported by a PEPCO switch box under the 14th Street pavement, as indicated in Figure 1.2. There was no other collapse along 14th Street. Therefore, it can be reasonably assumed that the width of the failure wedge was 20 feet at the ground surface level.

Based on the discussion in Section 4.1.1, the internal friction angle (ϕ) of the soil within the excavation depth is 30 degrees, thus, a line can be drawn from the bottom edge of the curb line at an angle of 60 degrees ($45° + \phi/2$) from horizontal downward to intersect the original excavation wall to establish the minimum failure wedge. Note that the depth of a vertical tensile crack is not considered in this estimation. A higher ϕ angle will result in a larger failure wedge.

The sketches and detailed computations of the minimum failure load are included in Appendix A4. The loading in each tier in terms of percentage of expected load is listed in Table 4.1. Note that two types of loading distribution, trapezoidal and triangular, are listed in this table for the minimum failure load. However, the above two cases should cover all possibilities. In addition, both distributions are included in the computer analysis in the subsequent section.

4.1.4.2 Most Probable Failure Load

The failure wedge in this case is established from the minimum failure wedge and interpretation of pictures of the east wall, and north and south walls. From Figures 3.7, 3.8 and 3.9, there were near vertical failure surfaces under the pavement of east and south walls and along the UPI building external wall in the north side. The average depth of this vertical failure surface is about 11 feet. This surface along 14th Street can reasonably be assumed as a vertical tensile crack between the curb line and the pavement for the analysis. However, this crack may not have been easily detected on top of the pavement before the collapse.

The failure wedge for the most probable failure load is constructed by drawing two lines, a downward vertical line from the curb line and an outward and upward line from the bottom of the excavation limit, with an angle of 60 degrees from horizontal. The length of the vertical line from the top of the pavement to the intersection point is 11 feet.

The detailed computations of the most probable failure load are included in Appendix A4. The loading in each tier in terms of percentage of expected load for both failure loads is listed in Table 4.1.

Table 4.1

Distribution of the Earth Pressure Immediately Preceding the Collapse in Each Tier

Percentage of Expected Load

Loading Conditions	Top Tier	Middle Tier	Bottom Tier	Overall
Minimum Failure Load Trapezoidal Distribution	69	59	29	52
Minimum Failure Load Triangular Distribution	31	60	62	51
Most Probable Failure Load Trapezoidal Distribution	75	75	75	75

4.2 GENERAL DESCRIPTION OF INTERNAL SUPPORT SYSTEM

The structural support for the open excavation consisted of two systems, i.e., an internal bracing system and a tieback system. The internal bracing system consisted of soldier beams, wales, outlookers connecting the soldier beams and wales, struts, braces and pinpiles. The tieback systems consisted of soldier beams, tieback wales and the tiebacks. The open excavation, 150 ft. x 208 ft., was supported at three levels called the top tier, the middle tier and the bottom tier. The respective approximate elevations of the tiers were 50 ft., 38 ft. and 26.50 ft. Figures 3.45, 3.46 and 3.47 indicate the as-built dimensions and member sizes of the internal support system and the identification numbers of soldier beams based on field observations. The lengths of the outlookers, which are of significant interest to this report, are also based on field measurements. Table 4.2 summarizes the locations of tiebacks and bracing systems at different points, based on field observations.

Table 4.2

Soldier Beam	Top Tier	Middle Tier	Bottom Tier
3 to 13	Tieback	Tieback	Tieback
14 to 57	Bracing	Bracing	Bracing
58 to 63	Bracing	Bracing	Bracing
64 to 76	Tieback	Tieback	Tieback
77 to 101	Tieback	Tieback	Tieback

NOTE: Soldier beam No. 63 is not a part of the bracing at bottom tier.

The west wall of the excavation was entirely supported by the tiebacks and did not experience any damage during the collapse. This wall was, therefore, of little interest to this report and was excluded from the structural analysis. The north and south walls, 208 feet long, of the open excavation were supported by tiebacks and bracing systems. Different lengths of walls were supported by different systems at different tiers. On the north side, approximately 89 feet at all three tiers were supported by the tieback system. The rest of the wall was supported by the internal bracing system. On the south wall, approximately 89 feet at the top and middle tiers were supported by the tieback system, approximately 97 feet at the bottom tier was supported by the tieback system. The balance of the wall was supported by the internal bracing system. Field observations indicated that the lengths of walls supported by the tieback system did not show any significant signs of distress and continued to support the soil retained by the laggings. Therefore, the tieback systems on the north and south side of the wall were also excluded from the structural analysis. The east wall was entirely supported by the bracing system.

The internal bracing consisted of soldier beam, wales, outlookers, cross-lot struts, diagonal braces and rakers at different locations and tiers. Three cross-lot struts consisting of W14x73 beams were located at the top and middle tiers each connecting soldier beam numbers 14 and 63, 16 and 61 and 18 and 59. The struts were vertically supported by pin-piles (W12x53) at approximately the quarter points. There were diagonal lacings in the horizontal plane between the struts consisting of structural tubing 5x5x3/16" in each span. See Figures 3.45, 3.46 and 3.47. Eight diagonal braces were provided at each of the top, middle and bottom tiers connecting soldier beams 20 and 37, 22 and 35, 24 and 33, 26 and 31, 40 and 57, 42 and 55, 44 and 53 and 46 and 51. Different sizes of structural members were used at different locations and have been noted on Figures 3.45, 3.46 and 3.47. The four longer diagonal braces spanning between soldier beams 20 and 37, 22 and 35, 40 and 57 and 42 and 55 were vertically supported by pinpiles (HP10x42) at the center of their spans. There were lacing in the horizontal plane between the diagonal braces. The member sizes of the cross-lot braces were observed to be the same as indicated on the plan. The sizes for the pinpiles for the diagonal bracing were different from those shown on the plan.

Wale sizes at three different tiers are indicated in the above figures, based on field observations. As discussed in Section 3.5, sizes of certain wales were changed in the field from the shoring subcontractor's plan.

The soldier beams sizes were documented as they were recovered from the debris and are shown in the above-mentioned figures. The size for soldier beam Nos. 34, 36, 37, 38, 39, 40, 43 and 46 were different from those shown in the subcontractor's plan.

The outlookers are short steel members connecting the flanges of the wales and the soldier beams. Their size and lengths were documented as the structural elements were recovered and removed from the debris. The size and length of each outlooker beam has been indicated in the above-mentioned figures. The lengths used for the analysis are the actual measurements of the outlooker beams. The size varied from location to location as did the lengths. On the shoring subcontractor's plans, the size of the outlookers were only indicated in a detail entitled, "Diagonal Brace Detail," as HP12x53, without any dimension of its length. In Section 2.2 and Section 3.3 on the drawing entitled, "Sections and Borings," dimensions of outlookers are given as 4 feet inclusive of the wale and 2 feet 6 inches clear, respectively.

The field welded connections of the outlookers to the soldier beam and wales were of significant importance to the integrity of the structure as they provided the load path to transfer the forces due to the active earth pressure on the east wall to the resisting passive soil on the north and south walls. Inadequacy of these connections would compromise the capability of the structural support system.

The length of the outlookers were cut to fit at the site to suit the actual distance between the wale and the solider beams. This was necessary because the soldier beams, while being driven, had a tendency to go off the vertical alignment due to sub-strata conditions and, thus, the position of the driven soldier beams could not be assured to be in a straight line.

The typical welding detail of the outlooker to the soldier beam and wale is shown in Figure 4.1, as per the information obtained from the field observation. Overhead welding was avoided and the top surface of the top and bottom flanges were connected with a 5/16" fillet weld. The web of the outlooker was not welded except for the outlookers comprising of built-up sections. As per information obtained from the field, welding electrodes having an ultimate tensile strength of 70 ksi were used. In the calculations in this report, therefore, the capacity of the welds was evaluated based on 70 ksi strength.

The outlookers at the three tiers on the north and south wall and their end connections were subjected to forces in three directions. An axial force due to the active soil pressure resisted by the soldier beam to which it is connected, a shearing force due to the active soil pressure of the east wall, and a moment due to the shearing force at the rigid connection of the outlooker to the wales and the soldier beam. The force having the significant impact on the capacity of the welds was the bending moment about the minor axis of the outlooker. The axial and shearing force had lesser effect. The weld stresses due to the bending moment were much higher than stresses from the shear and axial force. In order to facilitate the evaluation of the strength of welded connections of outlookers which were of varying sizes and lengths, a number of graphs were drawn to determine the welded capacity based on the ultimate tensile strength of the weld as 70 ksi. See Appendix A. Two cases were considered, e.g., Case I and Case II, as defined below.

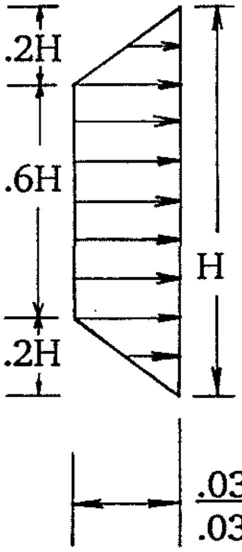
Case I: Both end connections of the outlookers were regarded as rigid and capable of resisting moments. Maximum capacity was based on the weld reaching a combined stress of 70 ksi in the compressive zone of the flange.

Case II: While the connection of the outlooker to the wale was considered rigid, the connection to the soldier beam was regarded as pinned because of the marginal torsional rigidity of the soldier beam. Maximum capacity was based on the weld reaching a combined stress of 70 ksi in the compression zone of the flange.

The structural analysis of the internal bracing system was undertaken to compute the stresses in the various elements of the system and to examine if there was any distress in any member or its connection due to the expected load. Manual design calculations were done to compute the expected soil pressure up to the final level of excavation and to calculate forces in different members of the system based on generally accepted engineering principles. Manual computations are generally employed in the industry in the design of such structures.

A computer analysis was also conducted to model each tier of the support system as a three dimensional structure to compute the forces in various members based on their relative stiffness due to the expected soil pressure. The computer results were used to supplement and compare with the manual calculations. Analysis was also done to determine stress levels in critical members due to varying lateral earth pressures.

The expected earth load was calculated on the basis of the trapezoidal pressure diagram, as shown below. No other surcharge load or hydrostatic pressure was assumed.



H = depth of excavation

<u>.03H for soldier beam Nos. 14 through 23, 34 through 63</u>
.035H for soldier beam Nos. 24 through 33

The above trapezoid represents the load which the structure was intended to support without failure. All the calculations were done on the basis of the above force diagram. The structural members were also examined for lateral earth pressure lower than shown above to determine their sensitivity to reduced loads. The above trapezoidal load agrees with the design loads assumed by the shoring subcontractor in its computation. This earth pressure diagram is commonly used in the design of excavation support systems in the Washington, D.C. area.

4.3 MANUAL ANALYSIS OF INTERNAL SUPPORT SYSTEM

Manual computations, for the internal bracing system, were done to determine the level of stresses in various structural elements and to verify their adequacy to resist the expected load. The expected load was represented by the trapezoidal force diagram, shown above, was based upon which the lateral load at each tier of support was computed, See Figure 4.2.

In all the computations, actual sizes of the members based on field measurements were used. The yield strength of the structural steel members was assumed to be 36 ksi and the ultimate tensile strength of welds was 70 ksi. The members examined were the wale on the north, south and east wall, diagonal braces, cross-lot struts, the sloping wale on the south wall, the outlookers and their connections. If the combined stresses of a member did not exceed the yield strength of 36 ksi, it was deemed to have the capacity to resist the imposed load. For this investigation, the stresses were not compared to the level of allowable stresses which is generally the basis for design of structures. The allowable stresses are always lower than the yield stress.

The outlookers are key elements in the transfer of the imposed loading due to the earth pressure from the east wall of the excavation to the north and south wall soldier beams. The outlookers were subject to the axial load due to the active pressure from the soldier beam to which it was attached and a shearing force from the wale. In addition, it was subject to a flexural moment about its minor axis due to its rigid connections.

The connections of the outlooker beam were also subject to similar forces. The outlookers were evaluated for two cases, e.g, Case I and Case II, and so were the connections. In Case I, it was assumed that the outlookers are rigidly connected at both ends. In Case II, the outlooker end at the wale beam was considered fixed but the other end of the outlooker was regarded as pinned. These two cases represented the extreme boundary conditions, the actual case would be in between the two extremes. In evaluating Case I, the point of contraflexure was regarded at the center of the outlooker. Therefore, the flexural demand on the outlooker in Case I would be one-half of Case II.

The following level of stresses were determined in the outlooker beams and their connections for Case I and II.

Table 4.3

Stresses in Outlooker (Fy = 36 ksi) Under Expected Earth Pressure in ksi

WALL		NORTH			SOUTH			
Tier	Soldier Beam #	7-13	14-21	22-28	49-57	58-63	64-67	68-71
Top	Case I Case II	56 112	65 121	65 121	49 90	32 57	25 50	64 128
Middle	Case I Case II	N/A N/A	65 123	47 88	48 90	48 90	N/A N/A	N/A N/A
Bottom	Case I Case II	N/A N/A	77 146	77 146	86 164	86 164	N/A N/A	N/A N/A

N/A = Not applicable.

____ No. 63 to be excluded from the bottom tier.

Under the expected load, all the outlookers at all tiers were stressed beyond their yield stress, if the Case II assumption was considered. Some were overstressed as high as four times the yield stress. If the Case I assumption was made, the result was similar except that at the top tier, the outlookers on soldier beams 58 through 67 were stressed below the yield strength. The remaining outlookers under the Case I assumption were stressed well beyond their yield stress.

The welded connections of the outlookers were then analyzed to examine their level of stresses. From field records, it was concluded that E70 electrodes were used and, therefore, a value of 70 ksi as the ultimate tensile strength (Fu) was used as described earlier. It may be noted here that the design is generally done with an allowable value of only 30 percent of the ultimate tensile strength. Under the expected load, all the welded connections were in distress, if the Case II assumption was considered. The level of stress in the welds was many times higher than the ultimate tensile strength and much higher when compared to the allowable design value of 0.3 Fu. If the Case I assumption was made, the result was similar except that at the top tier, the welded connections of the outlookers located at soldier beams 64 through 67 were stressed slightly below the ultimate strength. Table 4.4 summarizes the stresses in the welds under both conditions, e.g., Case I and Case II.

Table 4.4

Stresses in Outlookers Welds (Fu = 70 ksi) Under Expected Earth Pressure in ksi

		NORTH			SOUTH			
Tier	Soldier Beam #	7-13	14-21	22-28	49-57	58-63	64-67	68-71
Top	Case I Case II	146 293	181 326	181 326	123 220	85 143	59 117	130 258
Middle	Case I Case II	N/A N/A	161 297	115 209	117 213	117 213	N/A N/A	N/A N/A
Bottom	Case I Case II	N/A N/A	190 351	190 351	212 395	212 395	N/A N/A	N/A N/A

N/A = Not applicable.

_____ No. 63 to be excluded from the bottom tier.

The third area of distress, as determined by the computations, was the sloping wale between soldier beam Nos. 63 and 64 at the top tier on the south wall. The wale, W14x159, at the higher elevation (+50.00') had undergone a 40 degree slope to the lower elevation (+44') and to a reduced size of HP14x73. The sloping member was subjected to an axial and shearing force and a flexural moment about its minor axis. The combined stress was of the order of 158 ksi, much greater than the yield stress. Besides, the outlookers located at the soldier beams Nos. 63 and 64, at the south top tier, experienced flexural moments about the major axis in addition to the moment about the minor axis which resulted in additional distress in the member.

The fourth area of distress, based on the computations, was the wales at the top, middle and bottom tiers on the east wall between soldier beam Nos. 37 and 40 was subject to high stresses. The wales were subjected to an axial load and a flexural moment about their major axis. The resulting combined stress were greater than the yield strength.

All other members were determined to remain below the yield stress and no significant design deficiencies were noticed.

For manual computations, see Appendix A.

4.4 COMPUTER ANALYSIS OF INTERNAL SUPPORT SYSTEM

A three-dimensional computer program was used for the analysis [5]. The objective of the structural analysis was to determine the internal member forces of the excavation support system under the full earth pressure, which the internal bracing system was expected to encounter during the construction period. In addition to the full earth pressure, two other loading conditions were also considered, i.e., the estimated earth pressure preceding the collapse and preloading of the members.

The three-dimensional model included beam elements consisting of outlookers, wales, struts, diagonal braces, lacing and vertical pinpiles at each tier. The linear elastic program is capable of including second order P-Delta effects in the analysis.

The internal member forces for all the above loading conditions were compared with the yield strength of the steel (36 ksi). Those members having a total combined stress of greater than 36 ksi were identified.

To account for the passive resistance of the soil surrounding the soldier beams, two external boundary conditions were considered, e.g., Case I and Case II. First, in Case I, the soldier beam support joints were defined in the model as fixed support joints with a set of translational springs attached to them. The spring constants were 110 k/in. in the direction parallel to the web of the soldier beams and 16 k/in. in the direction perpendicular to the web of the soldier beams derived from appropriate subgrade reactions. Under this assumption, the soldier beams were allowed horizontal translation in two directions without any vertical movement. Second, in Case II, the soldier beam support joints were defined as fixed supports with the same translational springs as defined earlier, but in addition, the soldier beam support joints were also allowed to have full rotational freedom about the longitudinal axis of the soldier beams. The support joints were permitted to have lateral translational movements as well as rotational movement, but vertical movement of the soldier beam was not allowed. The assumptions made in Case I and Case II above were deemed to be extreme conditions and the actual condition was regarded to be in between the two. The influence of several intermediate values of translational spring constants for Case II on the behavior of the entire system were considered before the final value was chosen.

The three-dimensional model of the top tier of the internal support system is shown in Figure 4.3. There are 280 structural members including outlookers, wales, struts, braces, lacings and vertical pinpiles. The intersecting points of soldier beams and outlookers were modeled as support joint elements, the far end of the pinpiles at the middle tier were also defined as pinned supports. There are a total of 78 support joints at the top tier.

The computer model of the middle tier is shown in Figure 4.4. There are 261 beam members that include outlookers, wales, struts, braces, lacings and pinpiles with 76 support joints. The support joints are at the intersection of soldier beams and outlookers and at the far ends of the pinpiles at the top and bottom tiers. The same external boundary conditions of the soldier beam supports were made at this tier, e.g., Case I and Case II. The values for the spring constants were the same.

The computer model of the bottom tier is shown in Figure 4.5. There are 171 beam members and 67 support joints at this tier. Again, the spring constants employed in the analysis were the same for Case I and Case II.

In all three tiers of the computer models, the welded connections of outlookers to wale flanges were assumed to be fixed. For the outlooker connection to the soldier beam, see the discussion above for the assumptions of Case I and Case II. The joints at the struts/braces and wales were assumed to be fixed and pinned in separate computer runs. The connection between the lacings and struts/braces were regarded as pinned. The intersecting joint of the pinpiles and struts/braces were assumed to be pinned. As stated earlier, the far ends of the pinpile were regarded as pinned with lateral translation.

The forces applied at each support joint of the soldier beam and outlooker intersection point are shown in the figures in Appendix B. These loads were derived based on the trapezoidal earth pressure diagrams as shown in Figure 4.6. A four member element computer model was used for the soldier beam analysis. The soldier beams were assumed to be laterally supported at each tier of the internal bracing system and pinned at the bottom subgrade level. Results of the forces acting at each tier of the internal bracing system from the computer analysis are comparable to the results of the manual computation as discussed in the previous section of this report.

Examination of the member stresses at the preload stage was also done. The preload forces were applied at the intersecting joints of struts/braces and wales of each tier. The preload was assumed to be 50 percent of the total design load of strut and brace members as specified in the excavation support drawing prepared by the shoring subcontractor. The preload forces are shown in Figures 4.7, 4.8 and 4.9.

The wales were regarded as continuous horizontal members on the north, east and west walls. The size of the members were verified by the field observation. See Figures 3.45, 3.46 and 3.47 for different sizes of the wales.

The 6 foot drop in the elevation of the wale on the south wall was accordingly modeled. All splices in the wales were considered as fully welded connections to maintain continuity in the member.

Forces and combined stresses were computed in various members of the frame at each tier by application of the expected earth pressure derived from the trapezoidal force diagram. Each tier, as explained earlier, was run as an independent three dimensional space frame. Forces and combined stresses were also computed by applying "the minimum failure load." The minimum failure load is the approximate soil pressure applied to the bracing system immediately prior to the collapse, see discussion in Section 4.1.4. For this minimum load, the trapezoidal and triangular distribution of the earth pressure were considered. See Figure 4.26. The forces and combined stresses were computed for each distribution.

With the application of the expected load and the earth load at the time of the collapse, two cases were considered. The first case, was to assume the connection of the outlooker to the soldier beam flange as rigid and in the second case, the connection was assumed pinned. These two cases were undertaken to examine the level of stresses in the members particularly in the outlooker and its connection at two extreme boundary condition. The capacity of the welded connection of the outlooker beam was computed manually, as a result of axial, shearing force and moment acting at the joint, discussed earlier.

As a result of the structural analysis, the following conclusions are made under the application of the expected loading condition:

- All outlookers on the north and south wall at the top, middle and bottom tiers are overstressed beyond their yield strength in Case I and Case II. The axial force, shearing force and bending moments of the outlooker beams are listed in Appendix B. Those members having a combined stress greater than 36 ksi are identified in Figures 4.10, 4.11 and 4.12 for Case I and Figures 4.13, 4.14 and 4.15 for Case II for the top, middle and bottom tiers, respectively.

- The moment connections of the outlooker beams with wales and soldier beams in Case I and with wales in Case II were overstressed at all three tiers on the north and south wall beyond the ultimate tensile strength of the welds.

- The sloping wale between solider beams 63 and 64 at the first tier on the south wall was overstressed beyond the yield strength of the member.

- The wale between soldier beam Nos. 37 through 40 on the east wall was overstressed at the top, middle and bottom tiers.

- Other members which were also overstressed are identified in Figures 4.10, 4.11, 4.12, 4.13, 4.14 and 4.15.

The frames at the top, middle and bottom tiers were again analyzed under the application of trapezoidal earth pressure assumed to exist immediately prior to the collapse using both sets of assumptions as explained in Case I and Case II earlier. The following conclusions were reached:

- For Case I, all outlookers at the top tier on the north wall and seventeen outlookers on the south wall were stressed beyond the yield strength. See Figure 4.16. At the middle tier, twelve outlookers on the north wall and three outlookers on the south wall were stressed beyond the yield strength. See Figure 4.17. At the bottom tier, none of the outlookers were stressed beyond the yield point. See Figure 4.18.

- For Case II, all outlookers at the top, middle and bottom tiers on the north and south walls were stressed in excess of 36 ksi. See Figures 4.19, 4.20 and 4.21.

- The sloping wale between soldier beams 63 and 64 at the top tier was overstressed.

- The wale on the east wall between soldier beams 37 through 40 was not stressed beyond the yield strength.

The frames at the top and bottom tiers were further analyzed under the application of a triangular earth pressure assumed to exist immediately prior to the collapse, for Case I and Case II. The middle tier was not analyzed because the applied loads in the trapezoidal and triangular distribution were about the same. The following conclusions were reached.

- For Case I, all outlookers at the top tier on the north and south walls were stressed below their yield point. However, at the bottom tiers, the outlookers were stressed beyond the yield point on the north and south walls. See Figures 4.22 and 4.23.

- For Case II, the outlookers on the north wall were stressed in excess of 36 ksi but on the south wall were stressed below the yield point. At the bottom tier, the outlookers on the north and south walls were stressed beyond their yield point. See Figures 4.24 and 4.25.

- The sloping wales at the top tier between soldier beams 63 and 64 were not overstressed.

- The wales on the east wall at the top and middle tier between soldier beams 37 through 40 were not overstressed either in Case I or Case II.

The structural system was also analyzed to examine the level of stress at the preloading stage. Figures 4.7, 4.8 and 4.9 indicate the plan of the top, middle and bottom tiers, respectively, showing the magnitude of force applied due to preloading. Again, the forces were computed for both cases, Case I and Case II. The members having combined stresses exceeding 36 ksi have been identified in Figures 4.27, 4.28 and 4.29, for Case I at top, middle and bottom tiers, respectively. For Case II, see Figure 4.30, 4.31 and 4.32 for top, middle and bottom tiers. Except for a few, all members were stressed below the yield point.

The welded connections of the outlookers were manually analyzed to examine the level of stresses under all the above conditions explained earlier, i.e., at the application of fully expected load, Cases I and II, the soil load at the time of the collapse with trapezoidal and triangular distribution, Case I and Case II and under preload condition, Cases I and II.

Table 4.5 lists the critical members exceeding the yield strength under different loading conditions. The welded connection exceeding the ultimate tensile strength are also identified. Under the expected earth pressure, the outlookers, their welded connections, the top tier sloping wale and the wale on the east wall were all overstressed beyond their yield capacity. Under the application of the soil pressure assumed to exist immediately prior to the collapse with the trapezoidal or triangular distribution, a large number of outlookers and their connections are distressed.

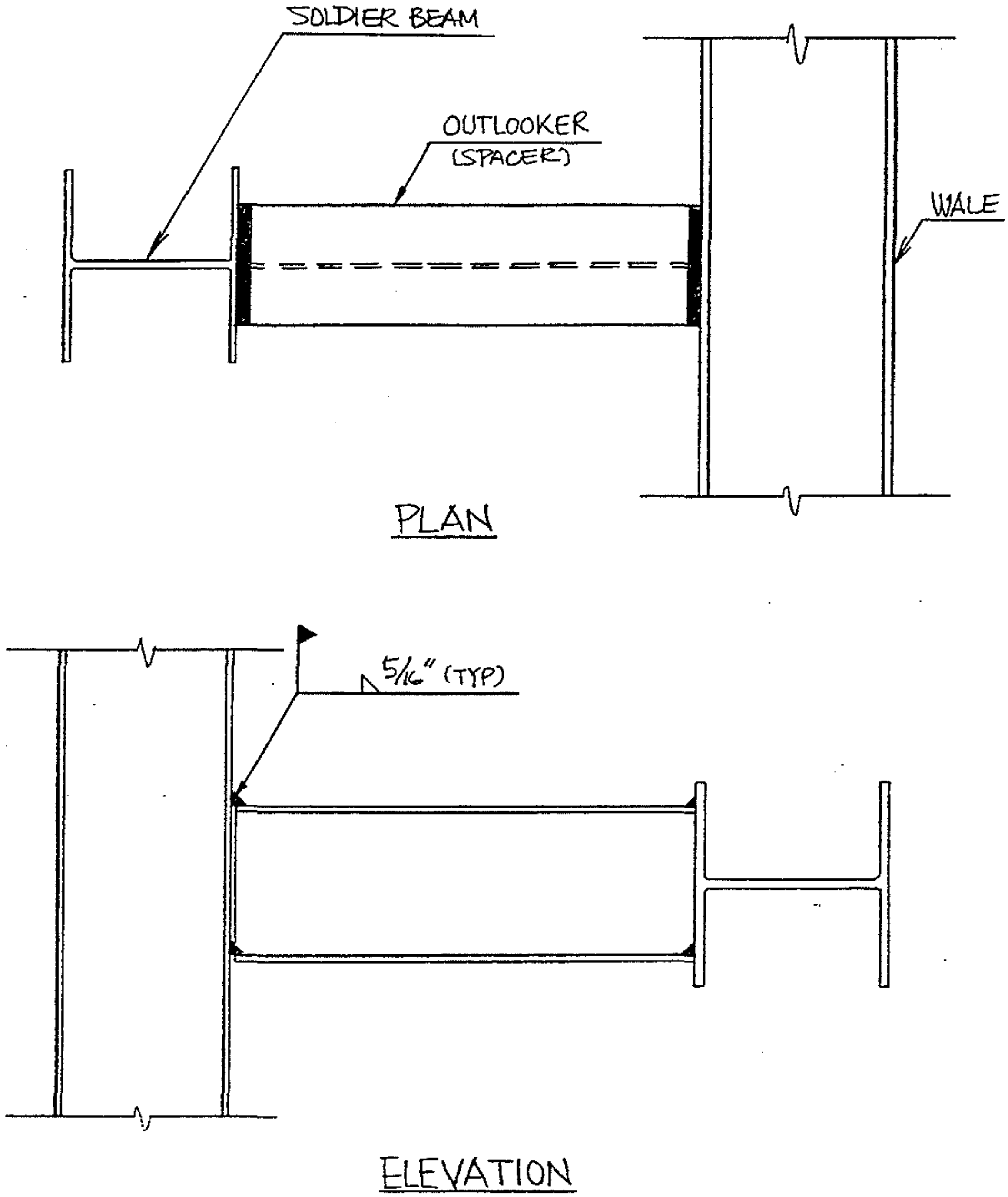
Table 4.5

Tiers	Earth Pressure	Outlooker Case I	Outlooker Case II	Outlooker Connection Case I	Outlooker Connection Case II	Sloping Wale	Wales Between Soldier Beams 37 thru 40
TOP	100% Expected Load	E	E	E	E	E	E
	Soil Pressure Preceding Collapse With Trapezoidal Distribution	E	E	E	E	E	B
	Soil Pressure Preceding Collapse With Triangular Distribution	B	E	B	E	B	B
	Preload	B	B	B	B	E	B
MIDDLE	100% Expected Load	E	E	E	E	N/A	E
	Soil Pressure Preceding Collapse With Trapezoidal Distribution	E	E	E	E	N/A	B
	Soil Pressure Preceding Collapse With Triangular Distribution	E	E	E	E	N/A	B
	Preload	B	B	B	B	N/A	B
BOTTOM	100% Expected Load	E	E	E	E	N/A	E
	Soil Pressure Preceding Collapse With Trapezoidal Distribution	B	E	B	E	N/A	B
	Soil Pressure Preceding Collapse With Triangular Distribution	E	E	E	E	N/A	B
	Preload	B	B	B	B	N/A	B

SYMBOL

E - Stress level exceeding 36 ksi for the structural members or 70 ksi for welds.

B - Stress level below 36 ksi for structural members or 70 ksi for the weld.



OUTLOOKER TO SOLDIER BEAM AND WALE WELDED CONNECTION DETAILS, AS-BUILT CONDITION

FIGURE 4.1

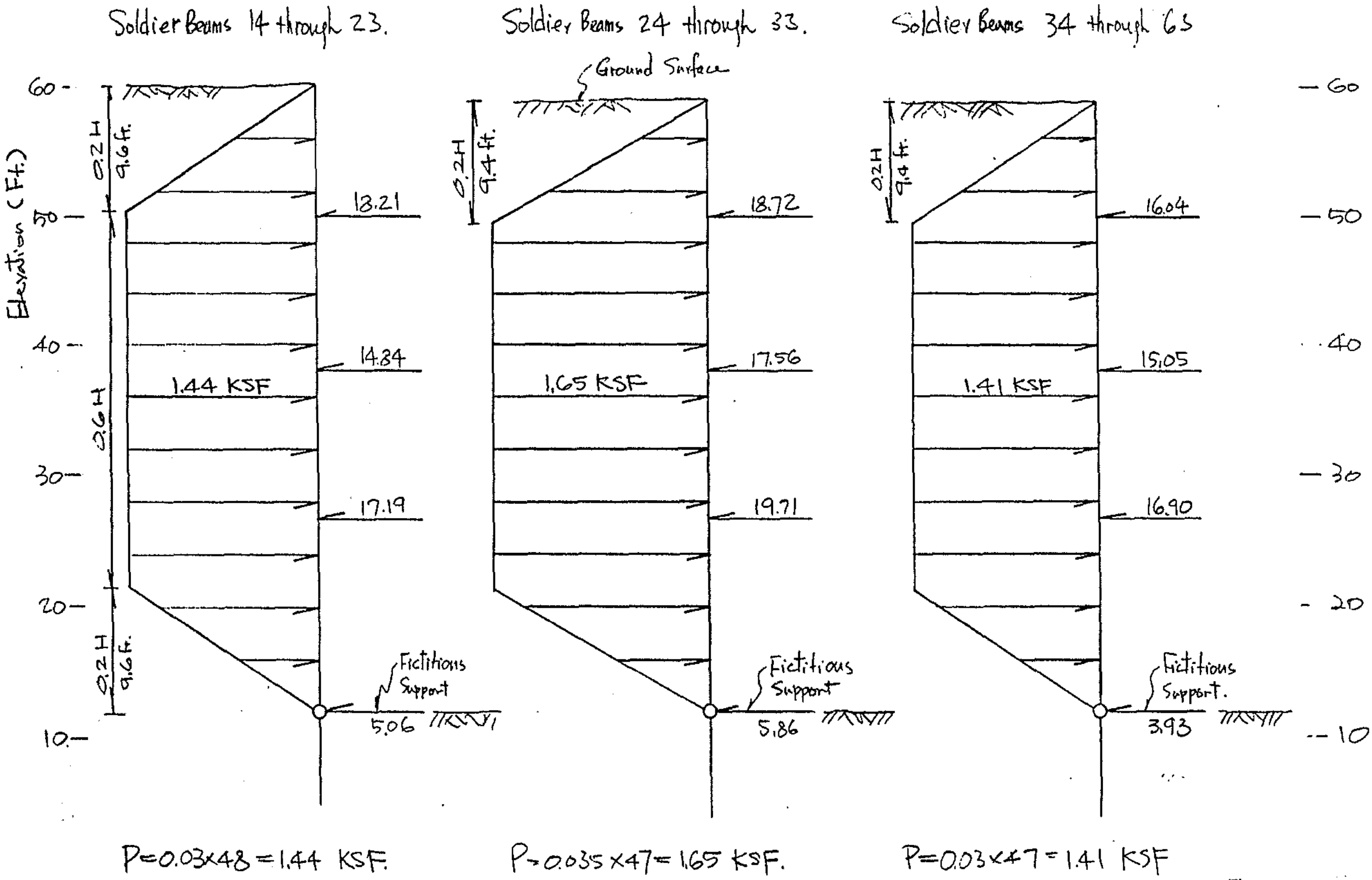
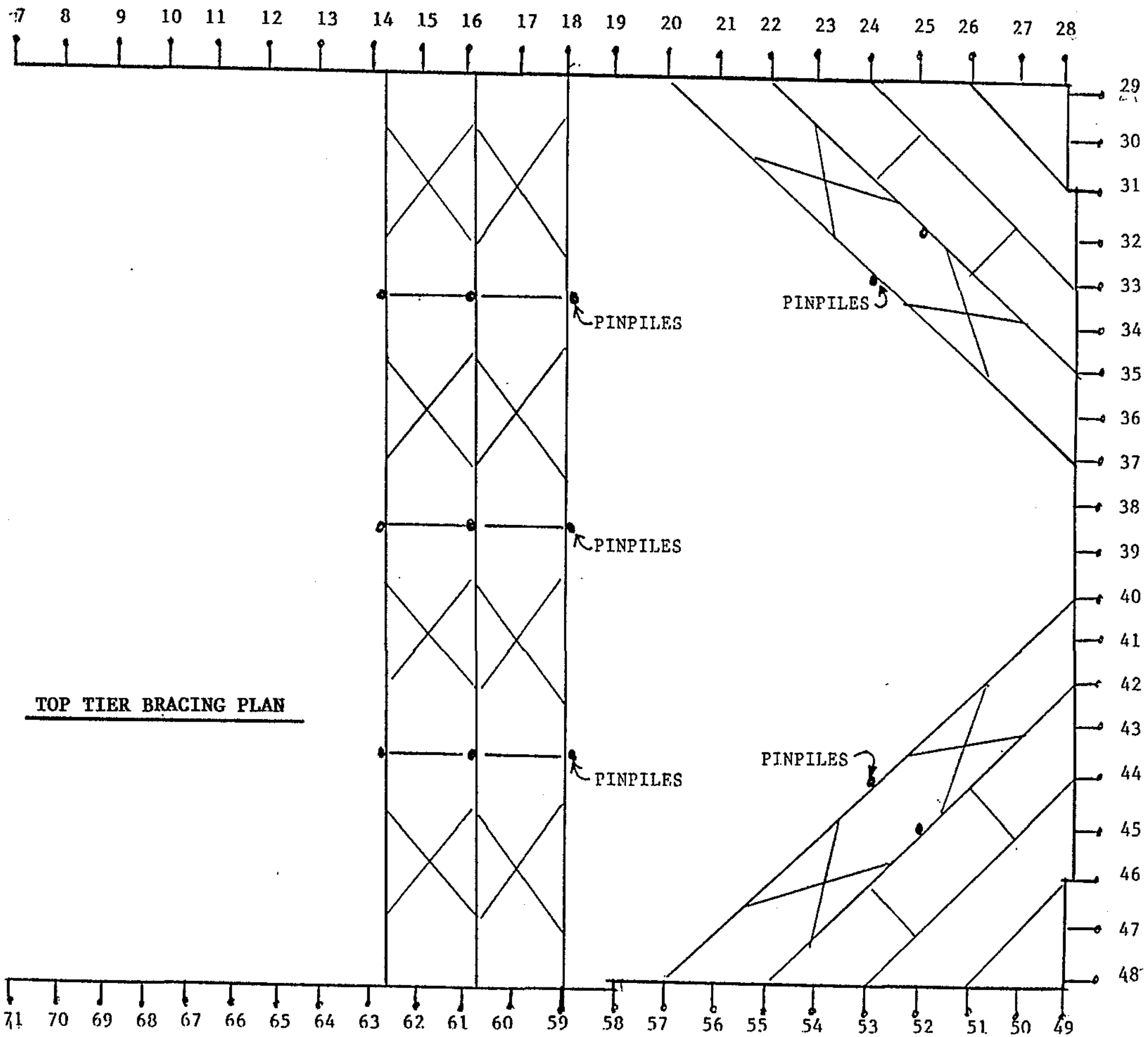


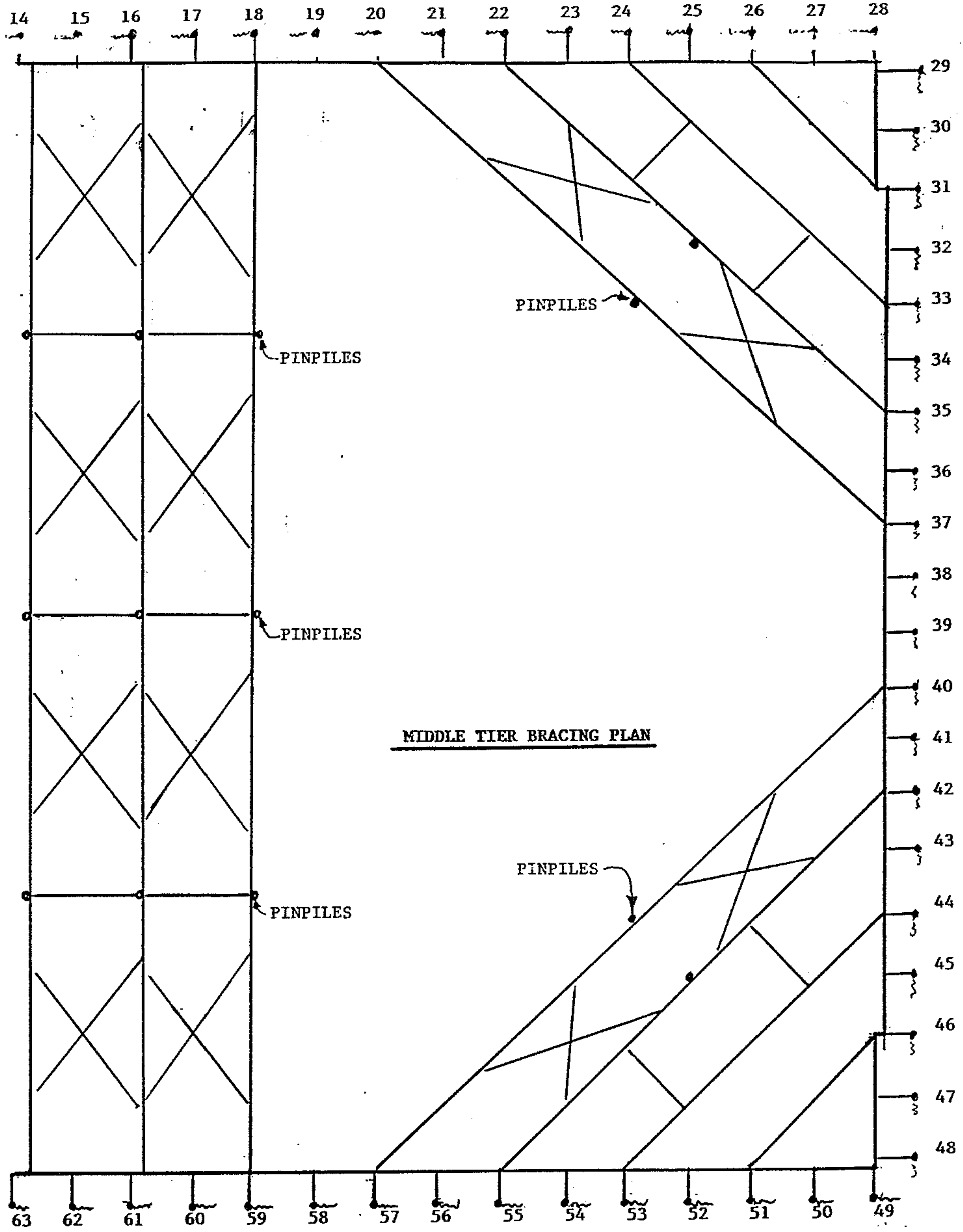
FIGURE 4.2

EXPECTED LATERAL LOADS AT THREE TIERS (MANUAL CALCULATION)



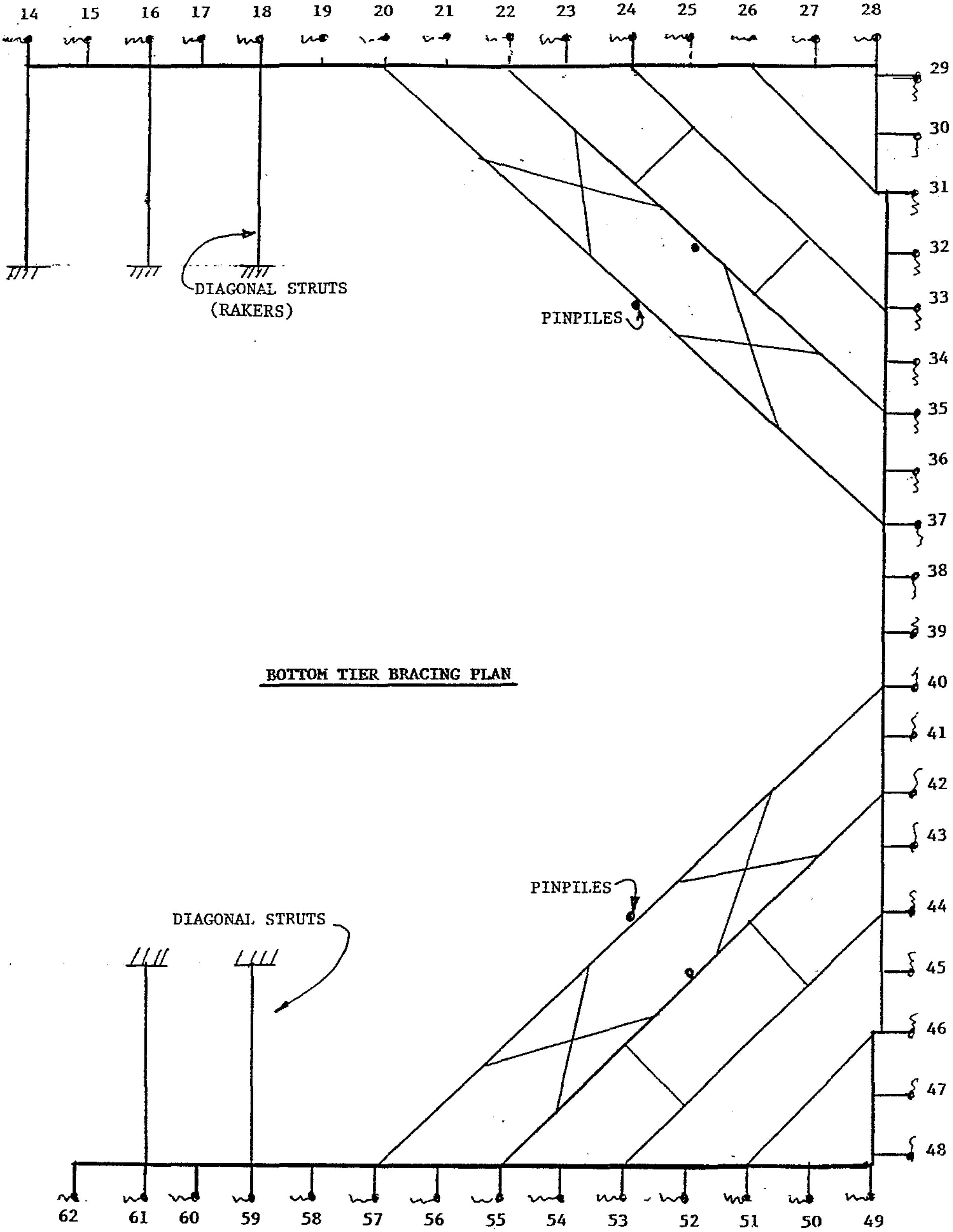
STRUCTURAL ANALYSIS MODEL OF THE INTERNAL SUPPORT SYSTEM

FIGURE 4.3



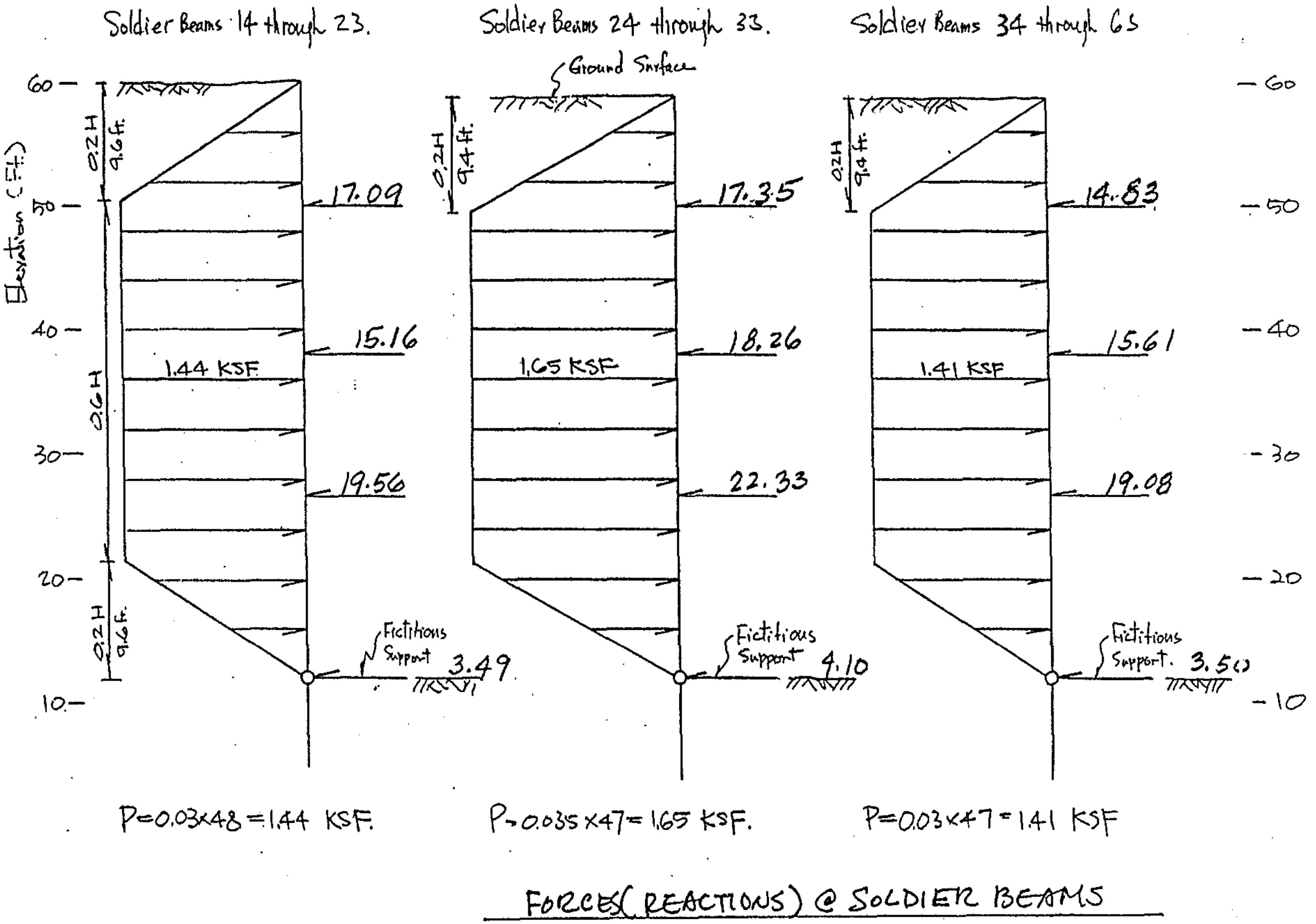
STRUCTURAL ANALYSIS MODEL OF THE INTERNAL SUPPORT SYSTEM

FIGURE 4.4



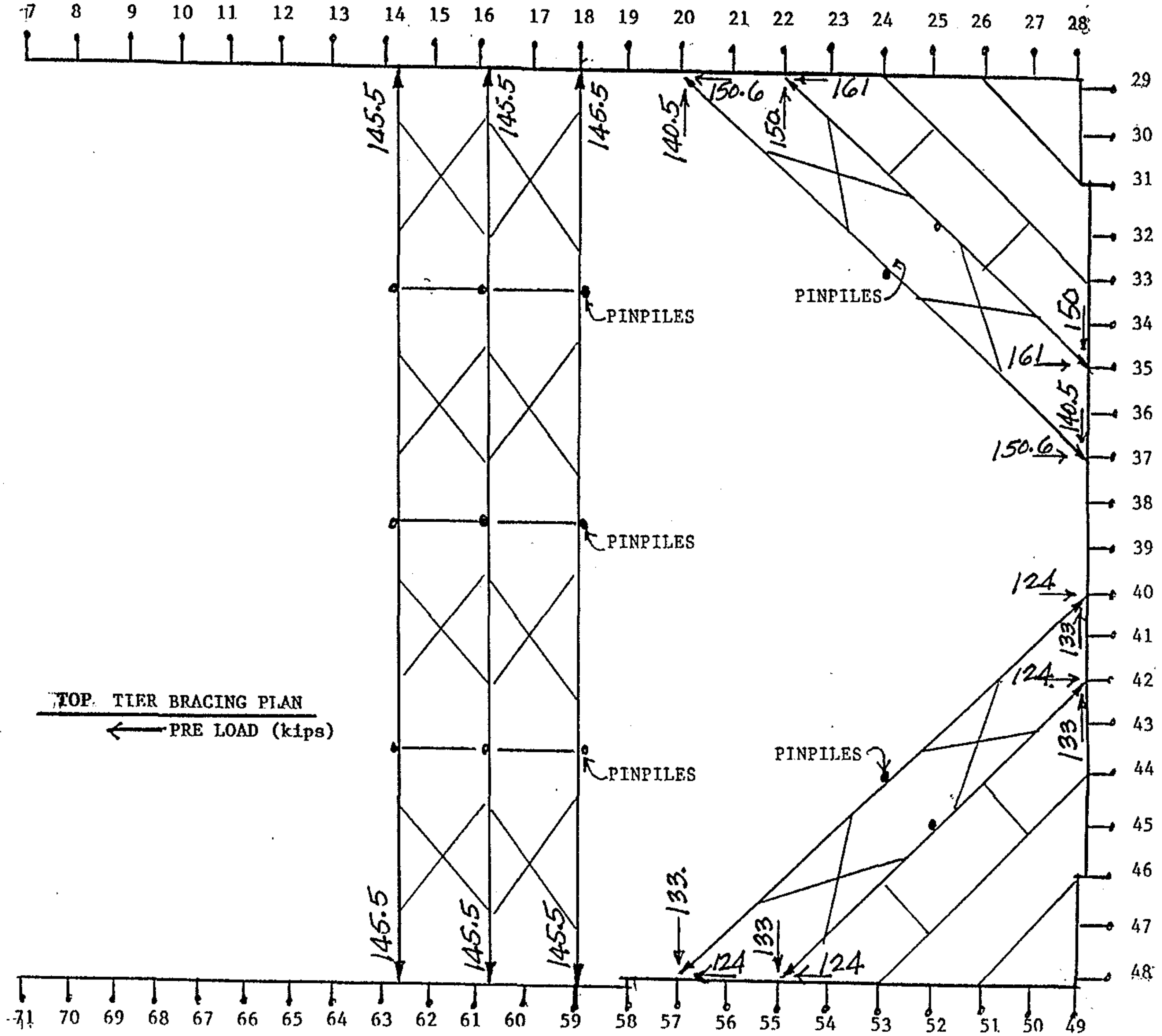
STRUCTURAL ANALYSIS MODEL OF THE INTERNAL SUPPORT SYSTEM

FIGURE 4.5

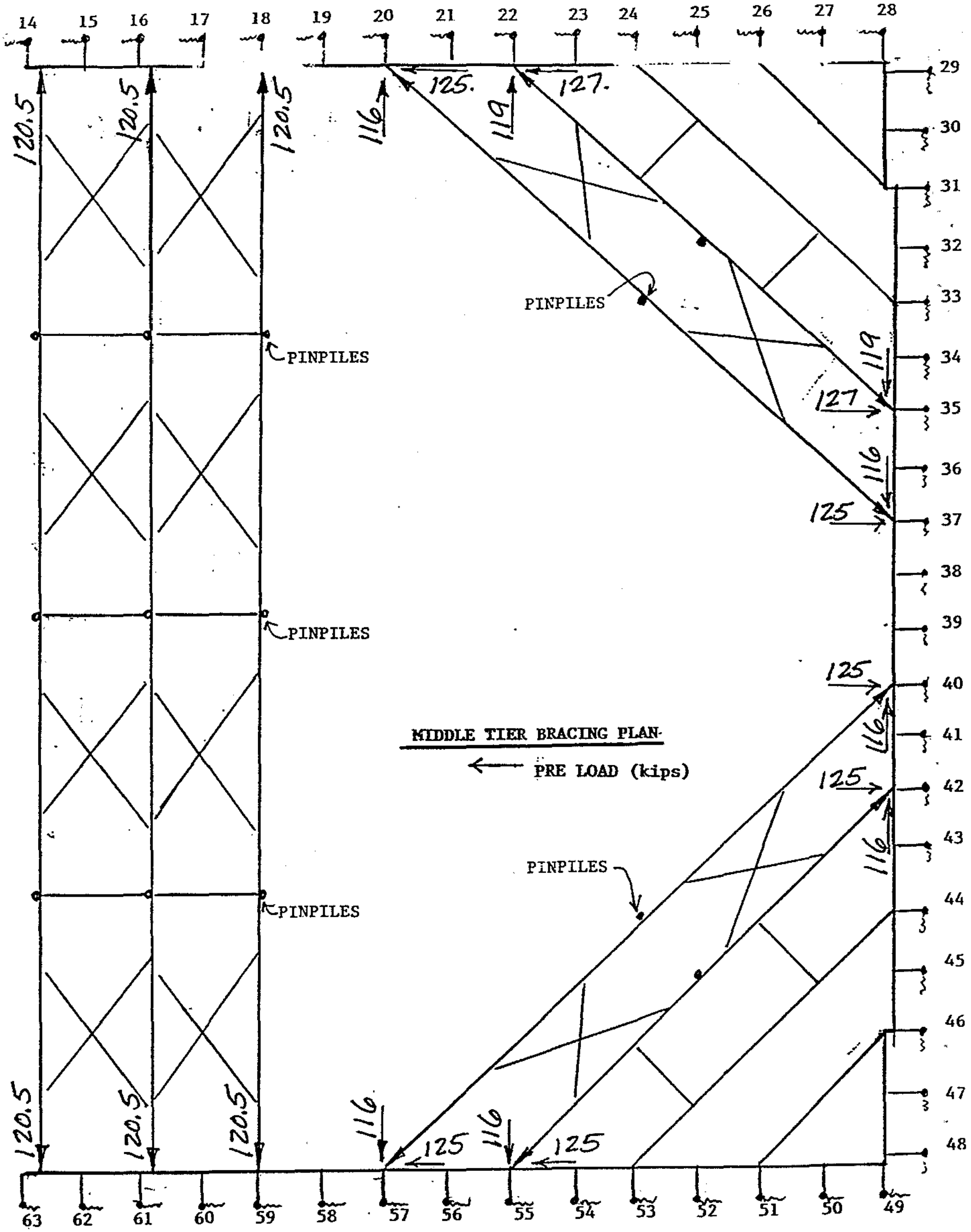


LATERAL LOADS AT THREE TIERS USED IN THE COMPUTER ANALYSIS

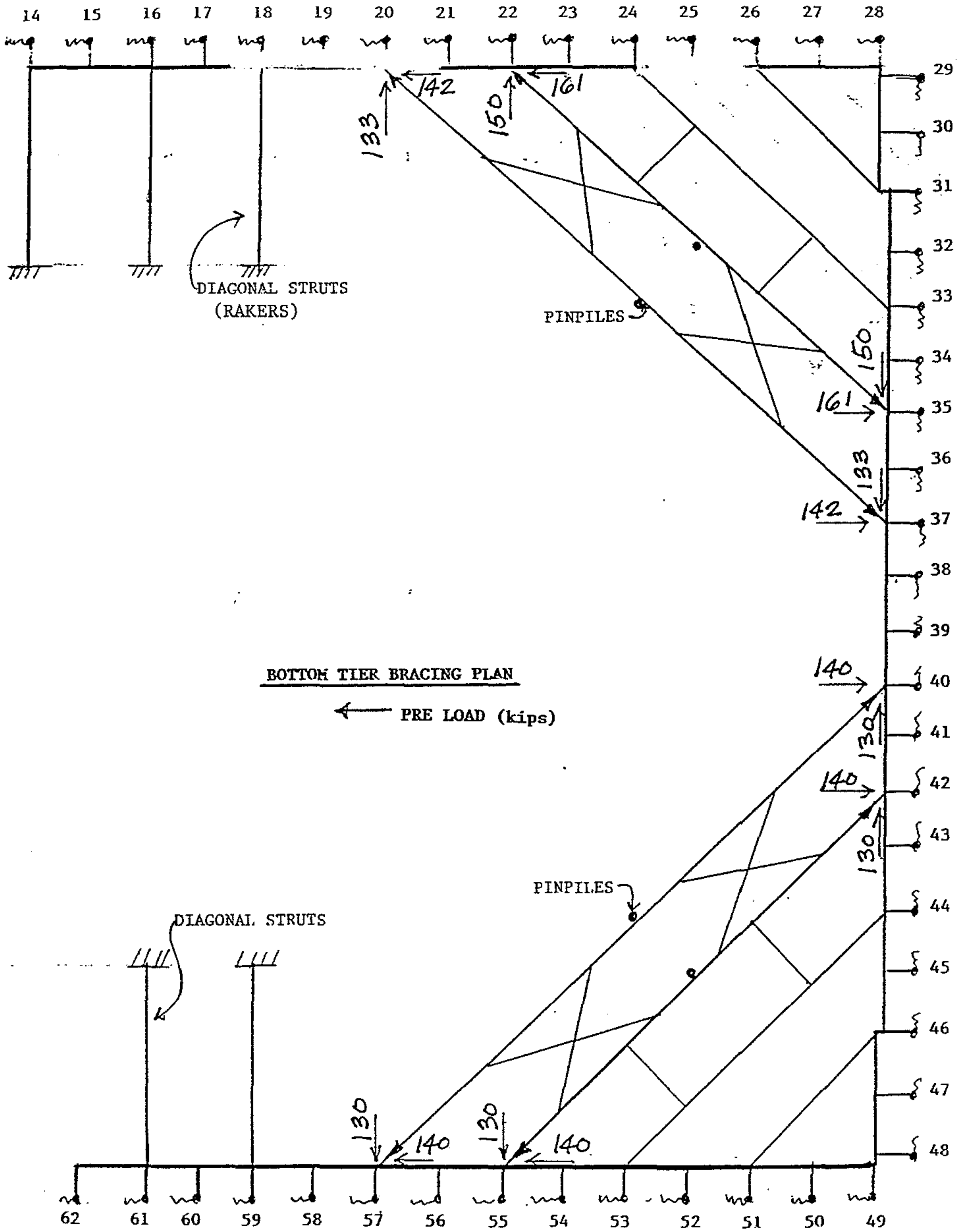
FIGURE 4.6



PRE LOAD OF THE TOP TIER USED IN THE COMPUTER ANALYSIS

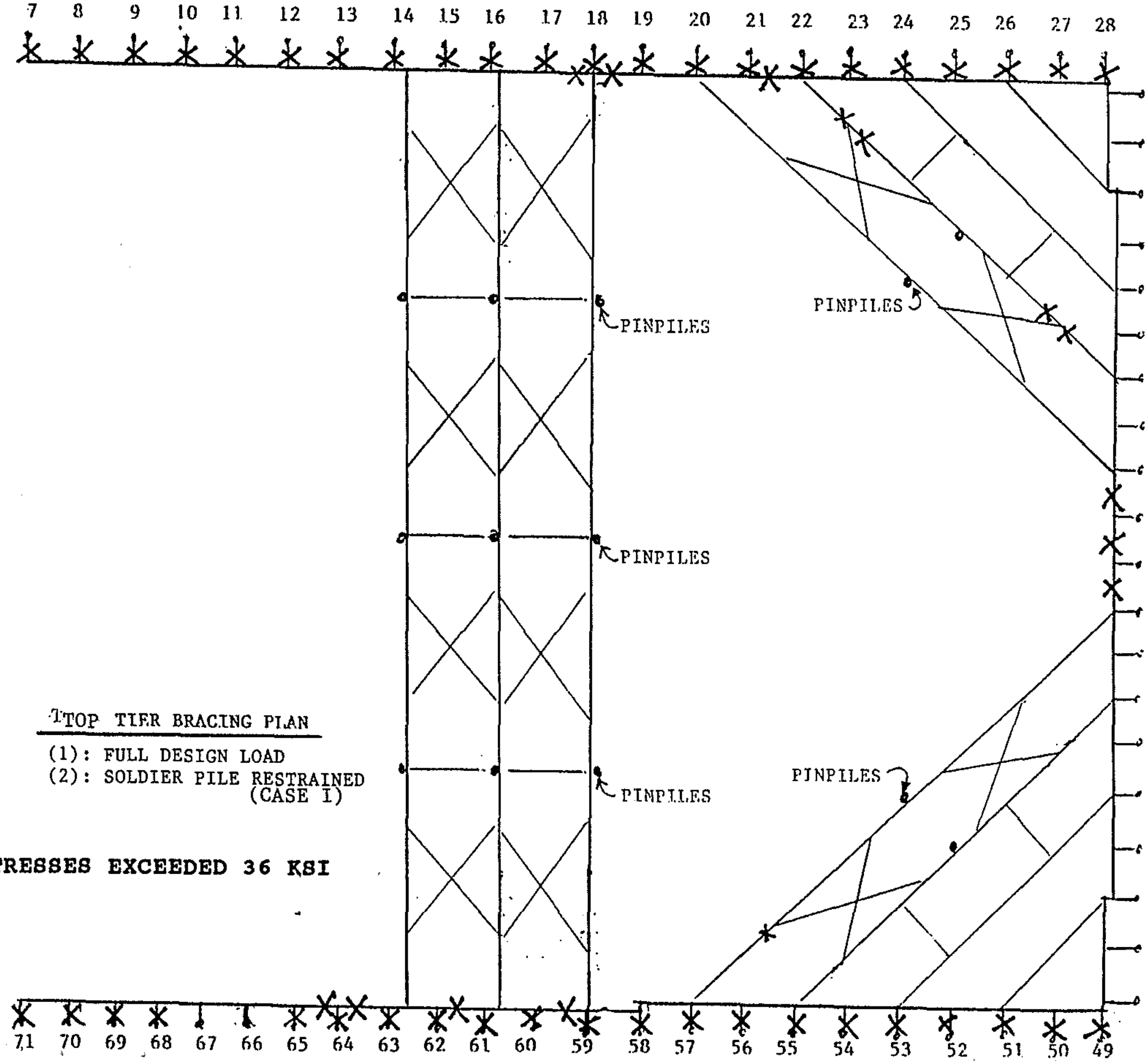


PRE LOAD OF THE MIDDLE TIER USED IN THE COMPUTER ANALYSIS
FIGURE 4.8



PRE LOAD OF THE BOTTOM TIER USED IN THE COMPUTER ANALYSIS

FIGURE 4.9

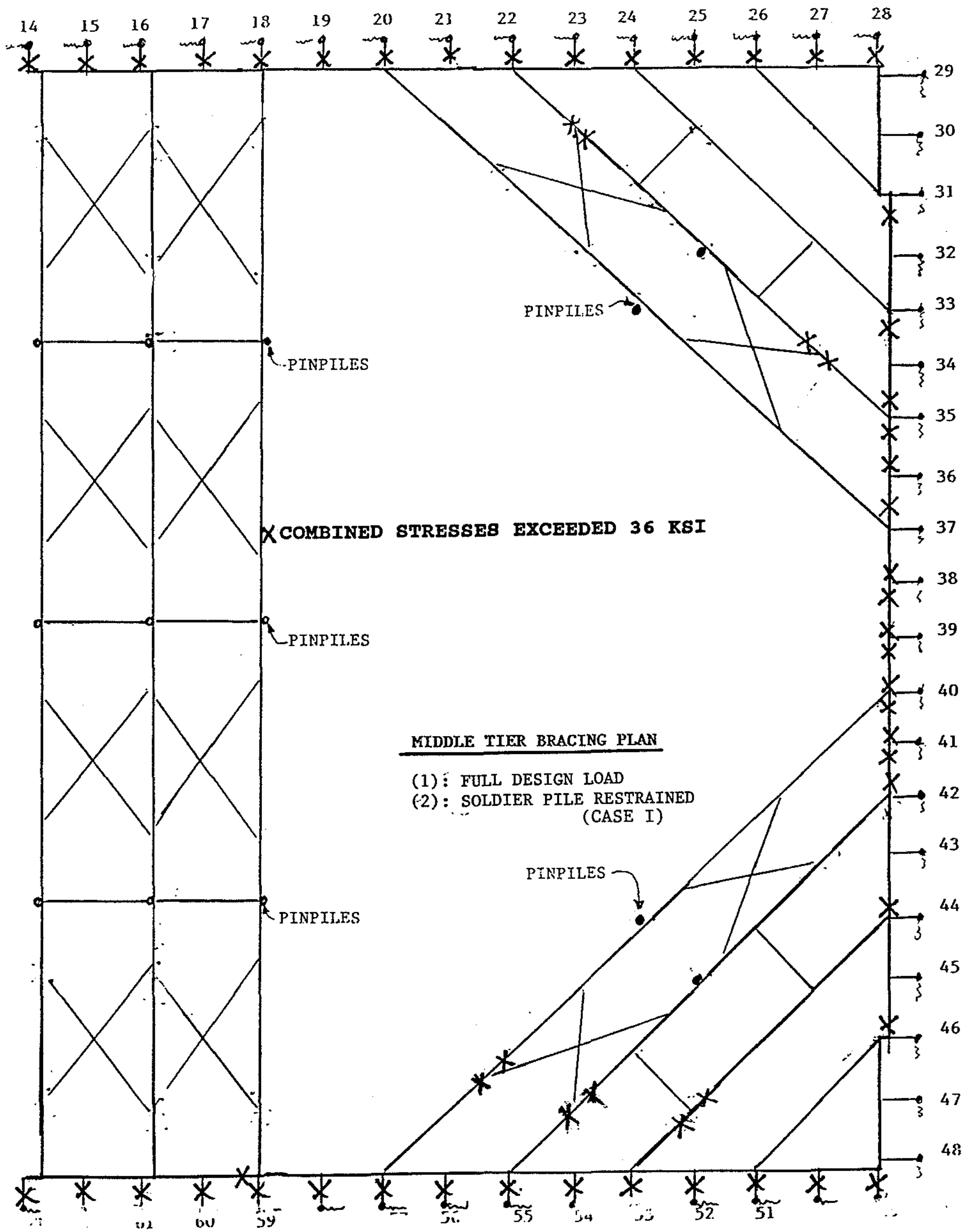


TOP TIER BRACING PLAN

(1): FULL DESIGN LOAD
(2): SOLDIER PILE RESTRAINED (CASE I)

X: COMBINED STRESSES EXCEEDED 36 KSI

FIGURE 4.10



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PINPILES
PINPILES
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PINPILES
PINPILES
COMBINED STRESSES EXCEEDED 36 KSI
MIDDLE TIER BRACING PLAN
(1): FULL DESIGN LOAD
(2): SOLDIER PILE RESTRAINED
(CASE I)
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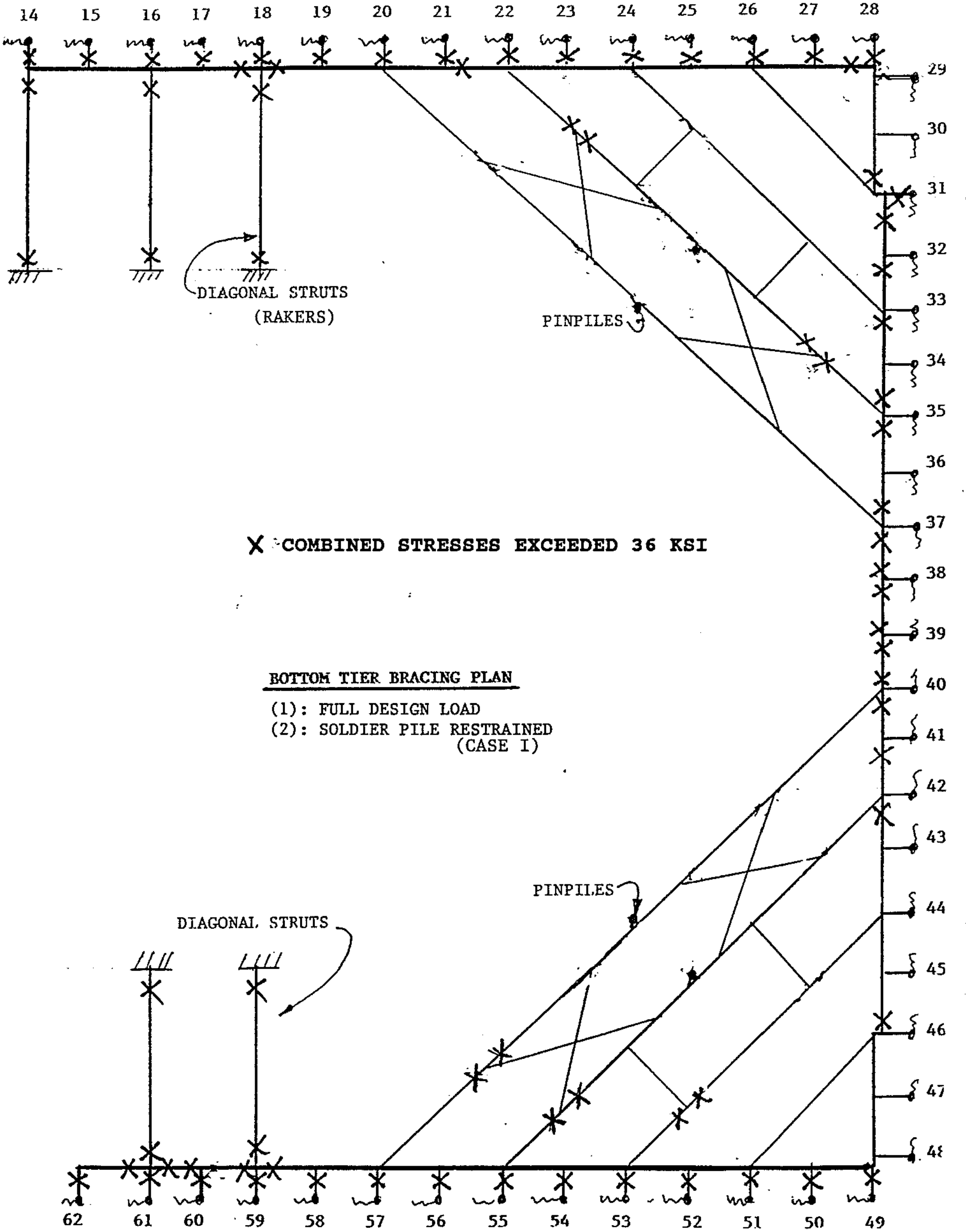
FIGURE 4.11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DIAGONAL STRUTS
(RAKERS)
PINPILES
X COMBINED STRESSES EXCEEDED 36 KSI
BOTTOM TIER BRACING PLAN
(1): FULL DESIGN LOAD
(2): SOLDIER PILE RESTRAINED
(CASE I)
PINPILES
DIAGONAL STRUTS
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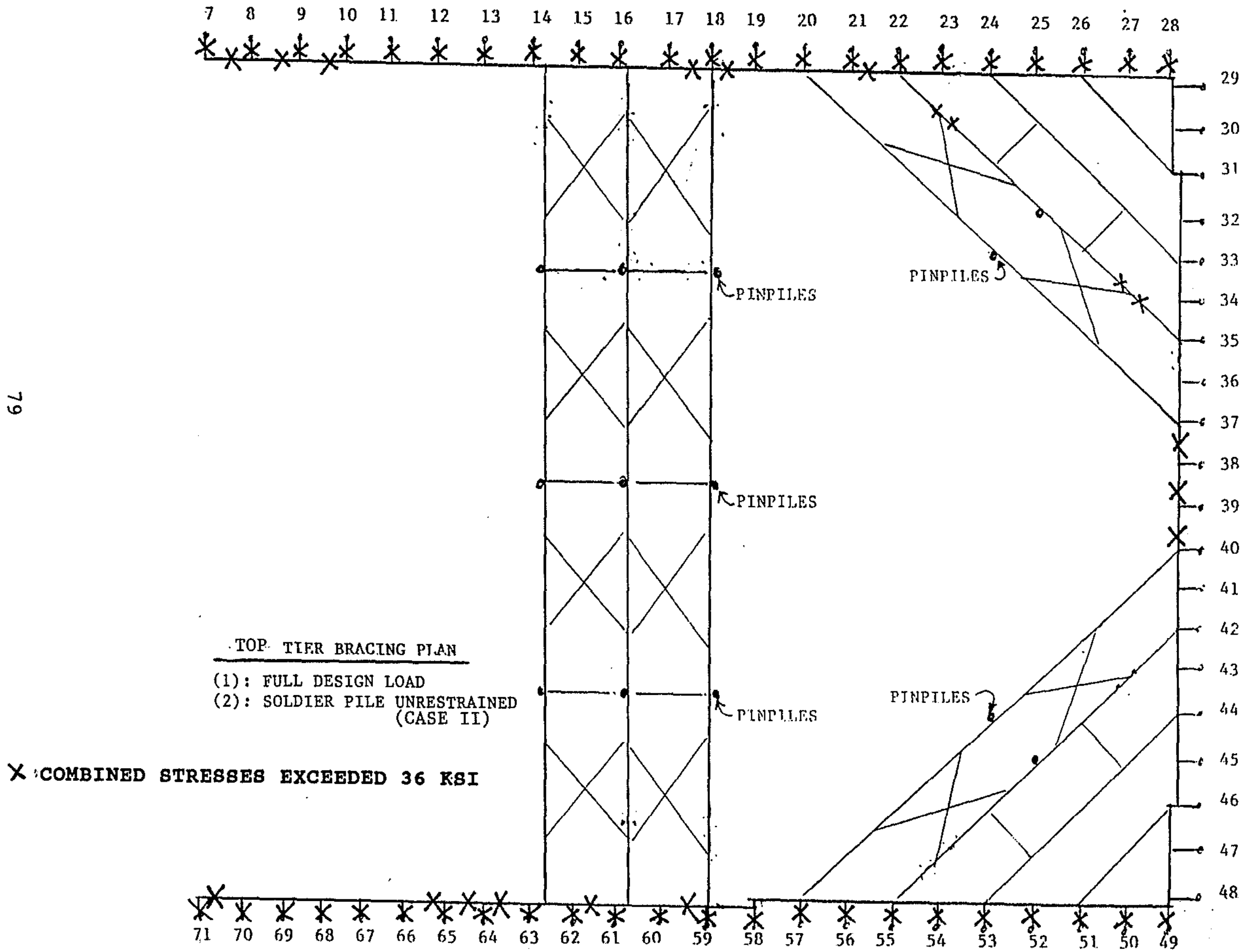
FIGURE 4.12



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71 70 69 68 67 66 65 64 63 62 61 60 59 58 57 56 55 54 53 52 51 50 49
PINPILES
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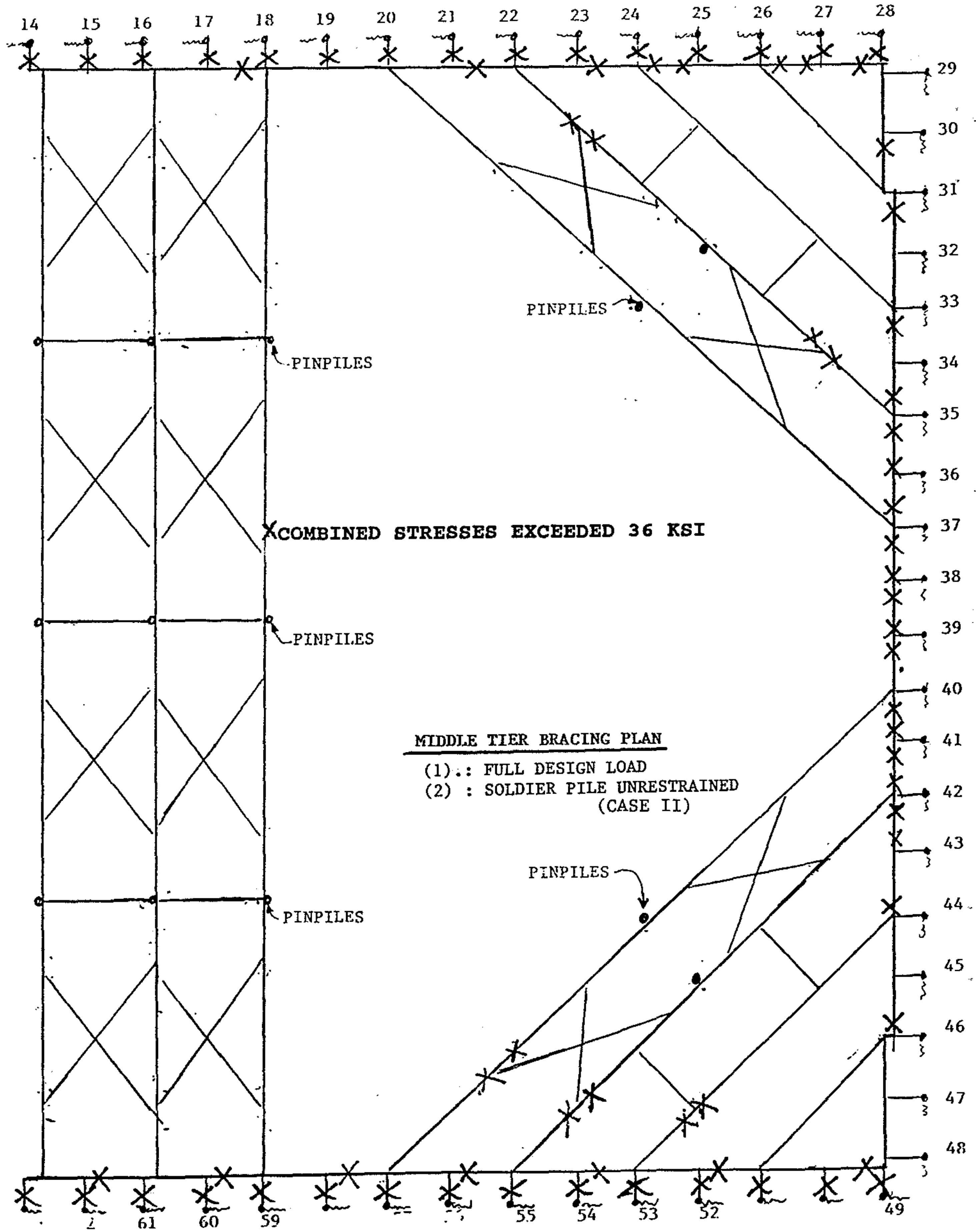
PINPILES
PINPILES
TOP TIER BRACING PLAN
(1): FULL DESIGN LOAD
(2): SOLDIER PILE UNRESTRAINED (CASE II)
X COMBINED STRESSES EXCEEDED 36 KSI

FIGURE 4.13



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PINPILES
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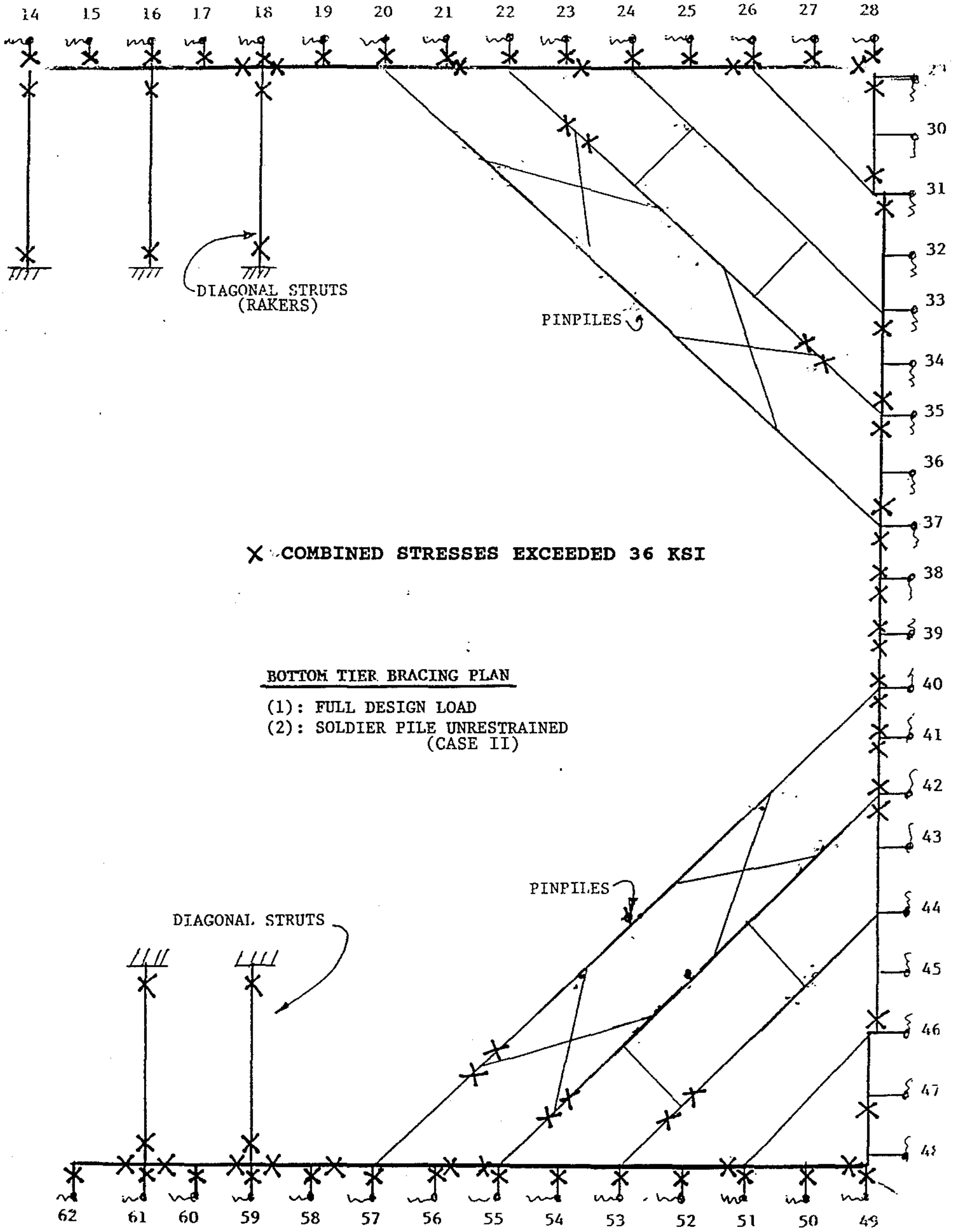
PINPILES
X COMBINED STRESSES EXCEEDED 36 KSI
MIDDLE TIER BRACING PLAN
(1) : FULL DESIGN LOAD
(2) : SOLDIER PILE UNRESTRAINED
(CASE II)

FIGURE 4.14



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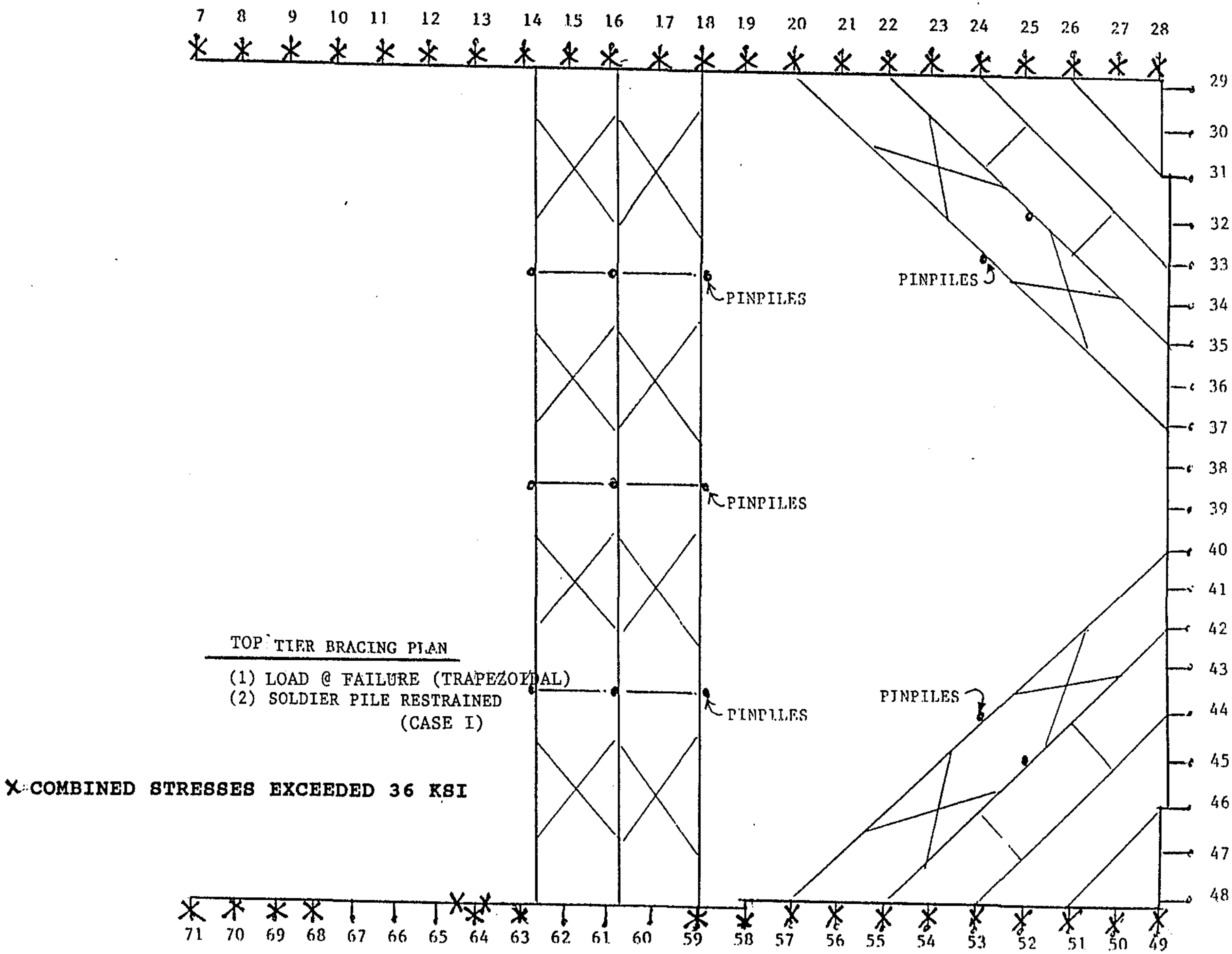
29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48
62 61 60 59 58 57 56 55 54 53 52 51 50 49
DIAGONAL STRUTS
(RAKERS)
PINPILES
X COMBINED STRESSES EXCEEDED 36 KSI
BOTTOM TIER BRACING PLAN
(1): FULL DESIGN LOAD
(2): SOLDIER PILE UNRESTRAINED
(CASE II)
PINPILES
DIAGONAL STRUTS

FIGURE 4.15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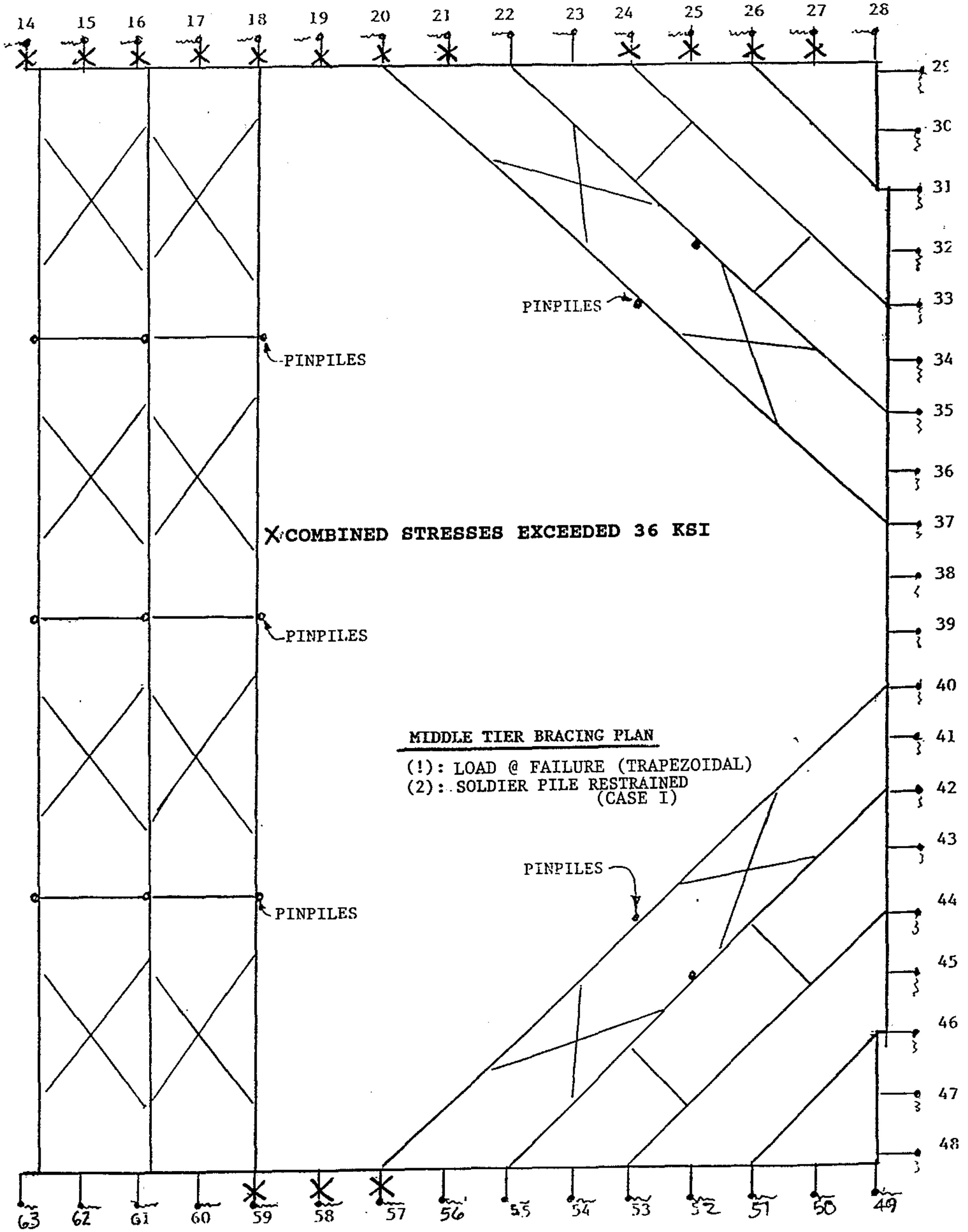
29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48
71 70 69 68 67 66 65 64 63 62 61 60 59 58 57 56 55 54 53 52 51 50 49
PINPILES
PINPILES
PINPILES
PINPILES
PINPILES
TOP TIER BRACING PLAN
(1) LOAD @ FAILURE (TRAPEZOIDAL)
(2) SOLDIER PILE RESTRAINED
(CASE I)
X COMBINED STRESSES EXCEEDED 36 KSI

FIGURE 4.16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PINPILES
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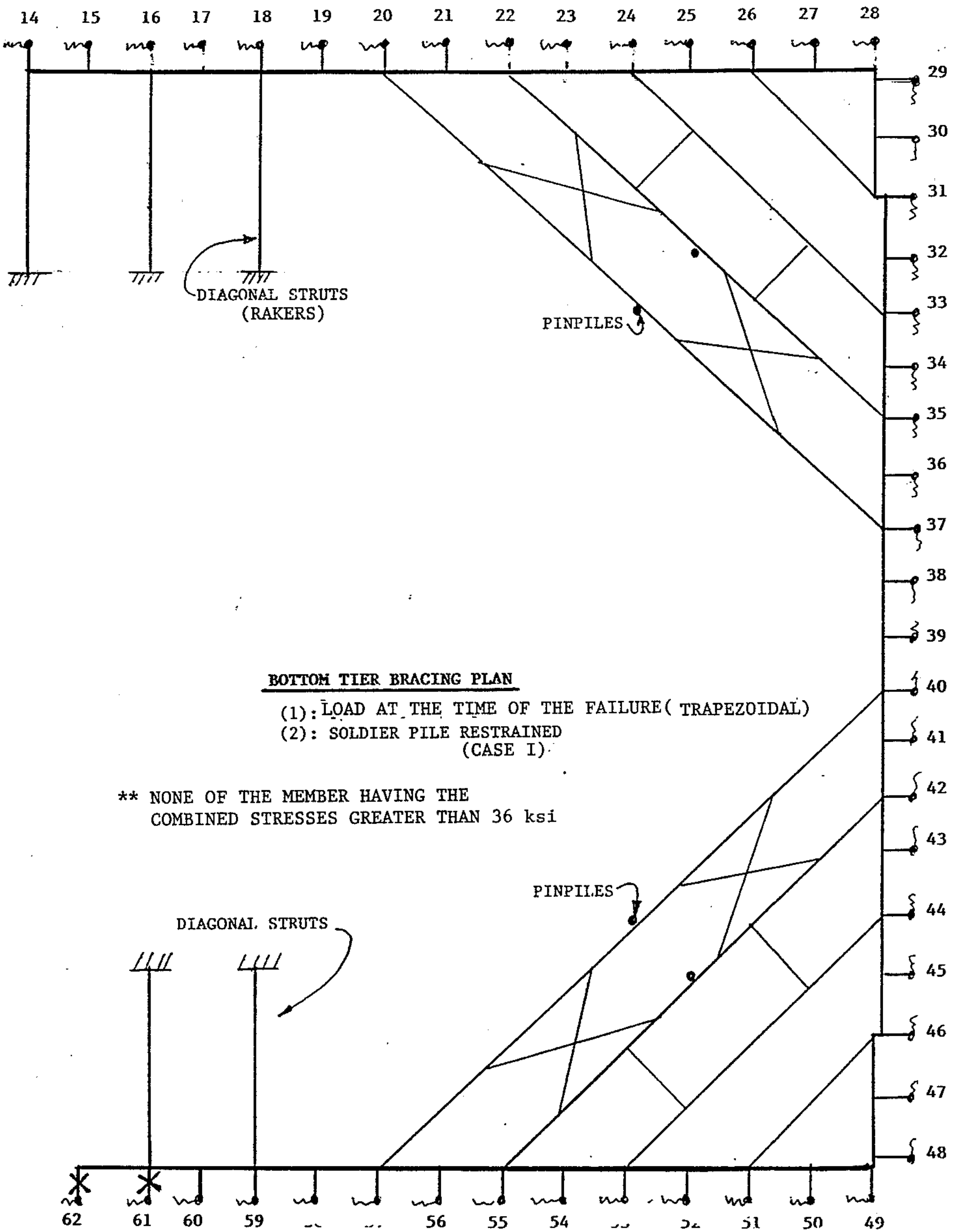
PINPILES
X COMBINED STRESSES EXCEEDED 36 KSI
MIDDLE TIER BRACING PLAN
(1): LOAD @ FAILURE (TRAPEZOIDAL)
(2): SOLDIER PILE RESTRAINED
(CASE I)

FIGURE 4.17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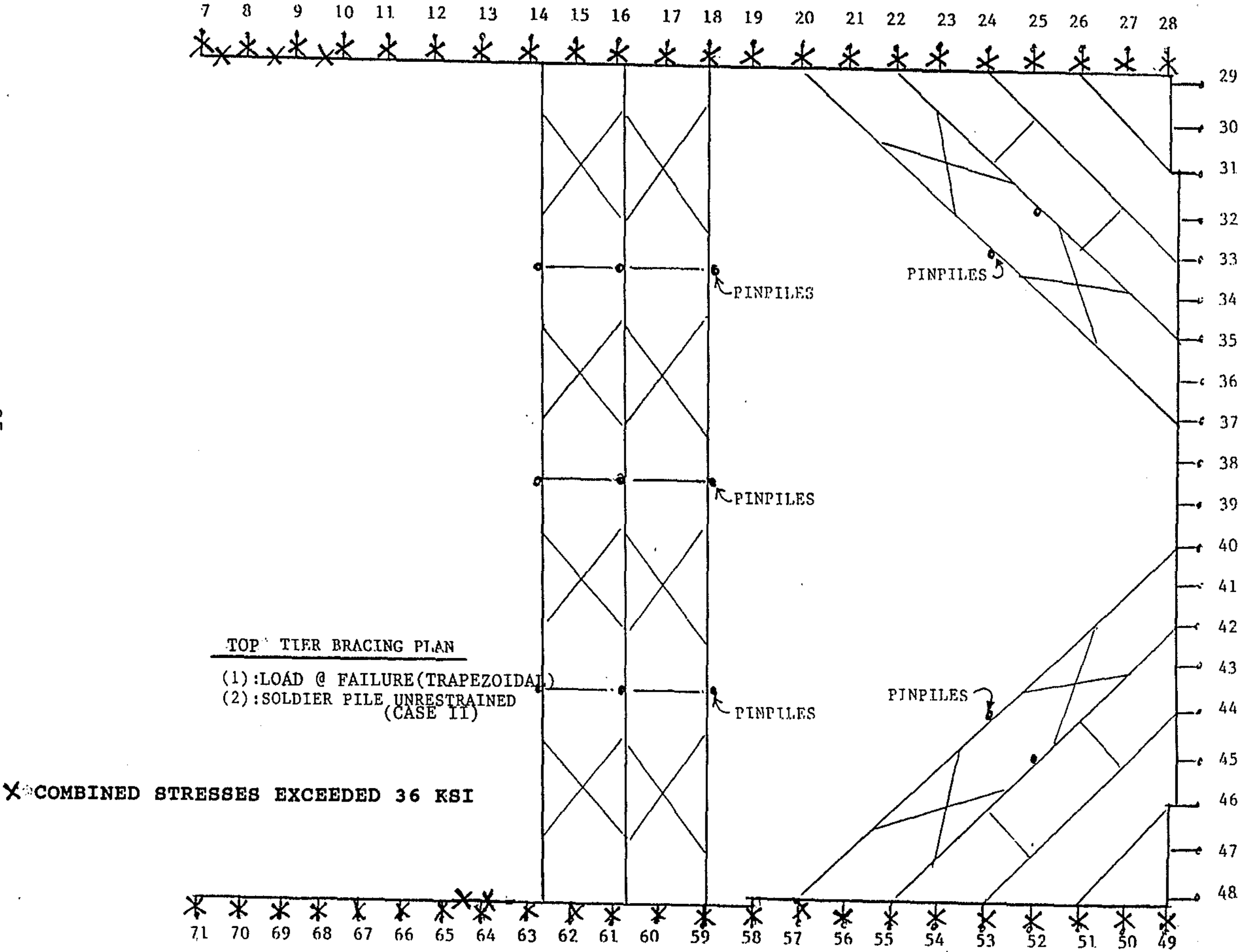
DIAGONAL STRUTS
(RAKERS)
PINPILES
BOTTOM TIER BRACING PLAN
(1): LOAD AT THE TIME OF THE FAILURE(TRAPEZOIDAL)
(2): SOLDIER PILE RESTRAINED
(CASE I)
** NONE OF THE MEMBER HAVING THE
COMBINED STRESSES GREATER THAN 36 ksi
PINPILES
DIAGONAL STRUTS

FIGURE 4.18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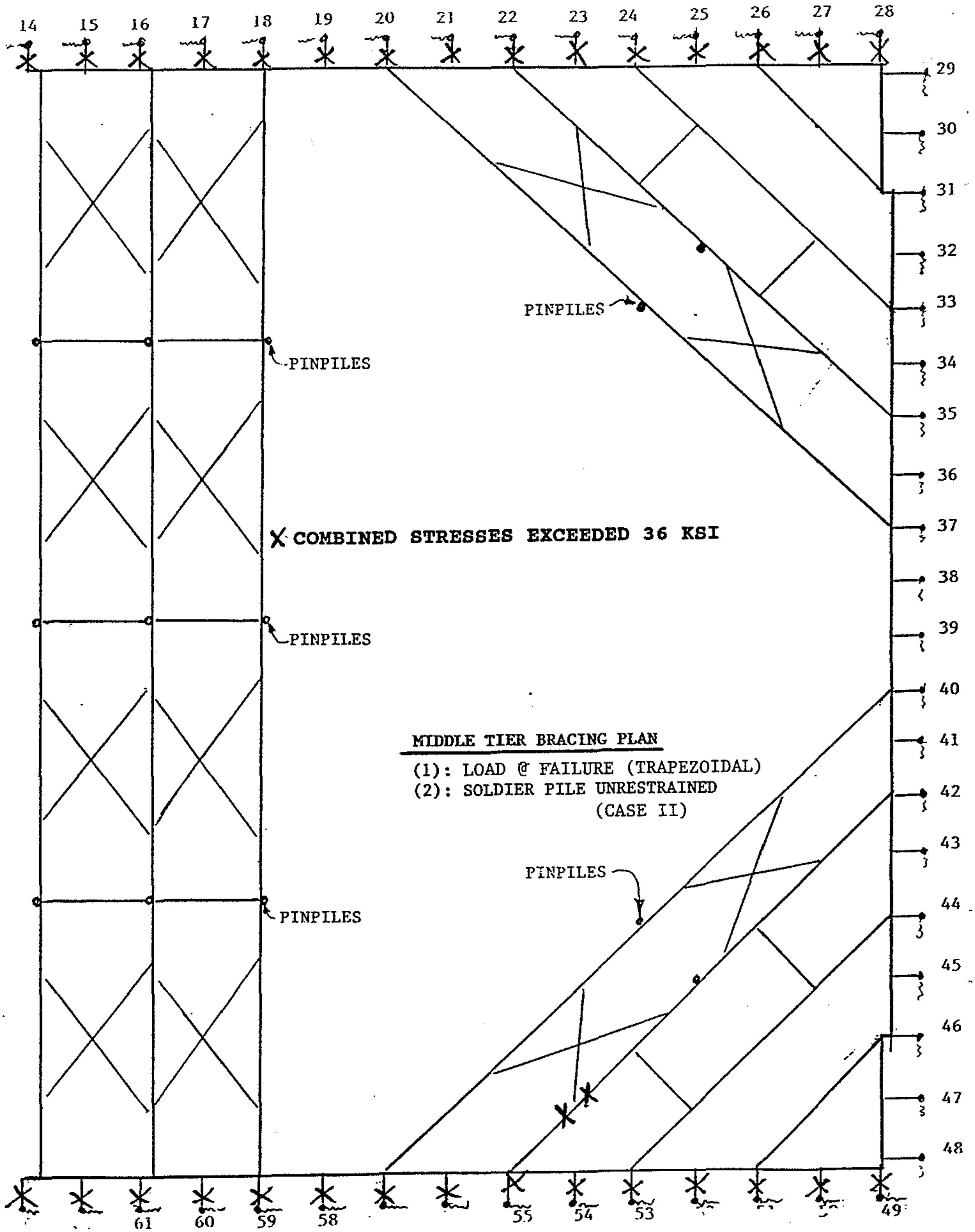
29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48
71 70 69 68 67 66 65 64 63 62 61 60 59 58 57 56 55 54 53 52 51 50 49
PINPILES
PINPILES
PINPILES
PINPILES
PINPILES
TOP TIER BRACING PLAN
(1):LOAD @ FAILURE(TRAPEZOIDAL)
(2):SOLDIER PILE UNRESTRAINED (CASE II)
X :COMBINED STRESSES EXCEEDED 36 KSI

FIGURE 4.19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PINPILES
X COMBINED STRESSES EXCEEDED 36 KSI
MIDDLE TIER BRACING PLAN
(1): LOAD @ FAILURE (TRAPEZOIDAL)
(2): SOLDIER PILE UNRESTRAINED
(CASE II)
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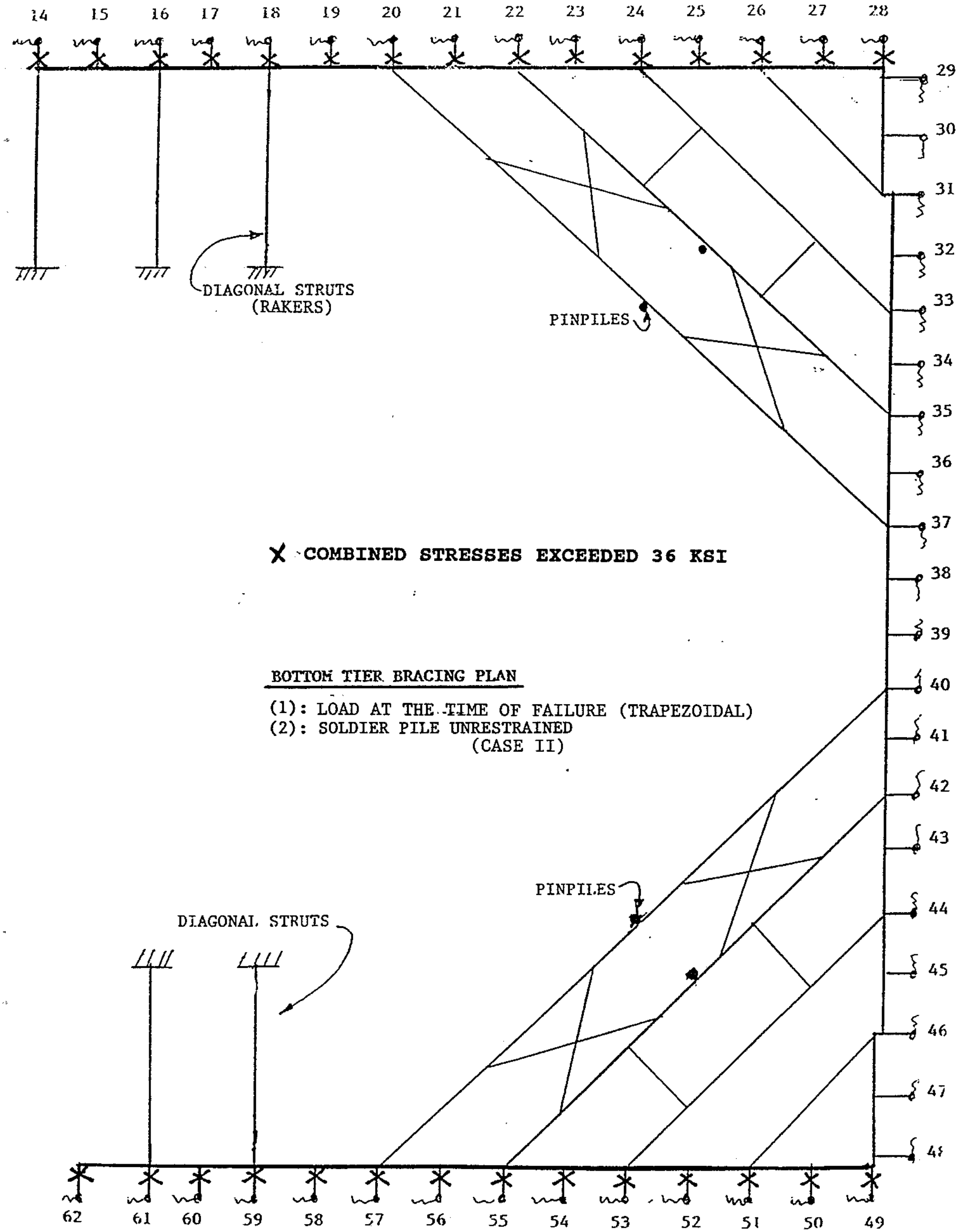
FIGURE 4.20



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DIAGONAL STRUTS
(RAKERS)
PINPILES
X COMBINED STRESSES EXCEEDED 36 KSI
BOTTOM TIER BRACING PLAN
(1): LOAD AT THE TIME OF FAILURE (TRAPEZOIDAL)
(2): SOLDIER PILE UNRESTRAINED
(CASE II)
PINPILES
DIAGONAL STRUTS
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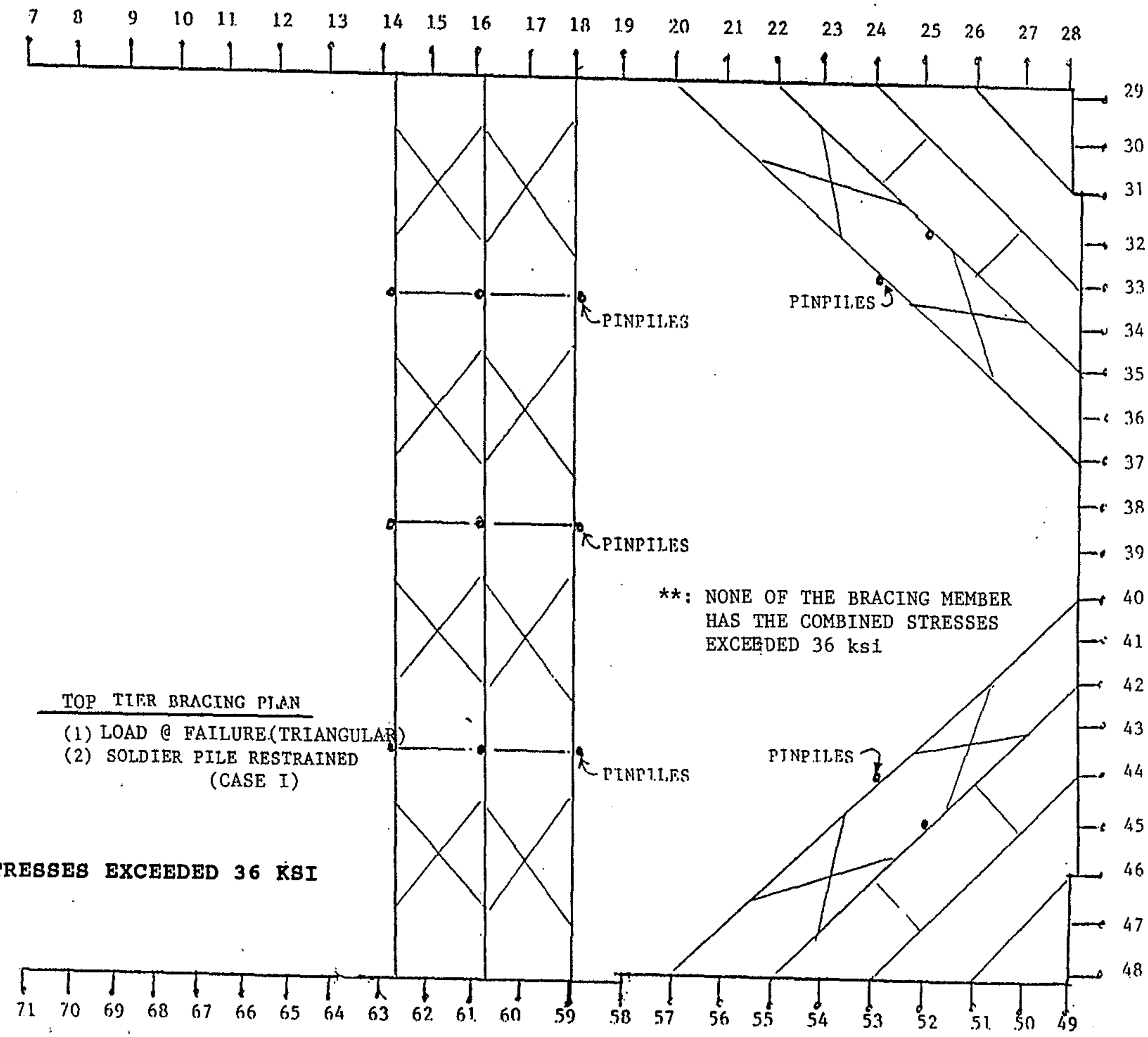
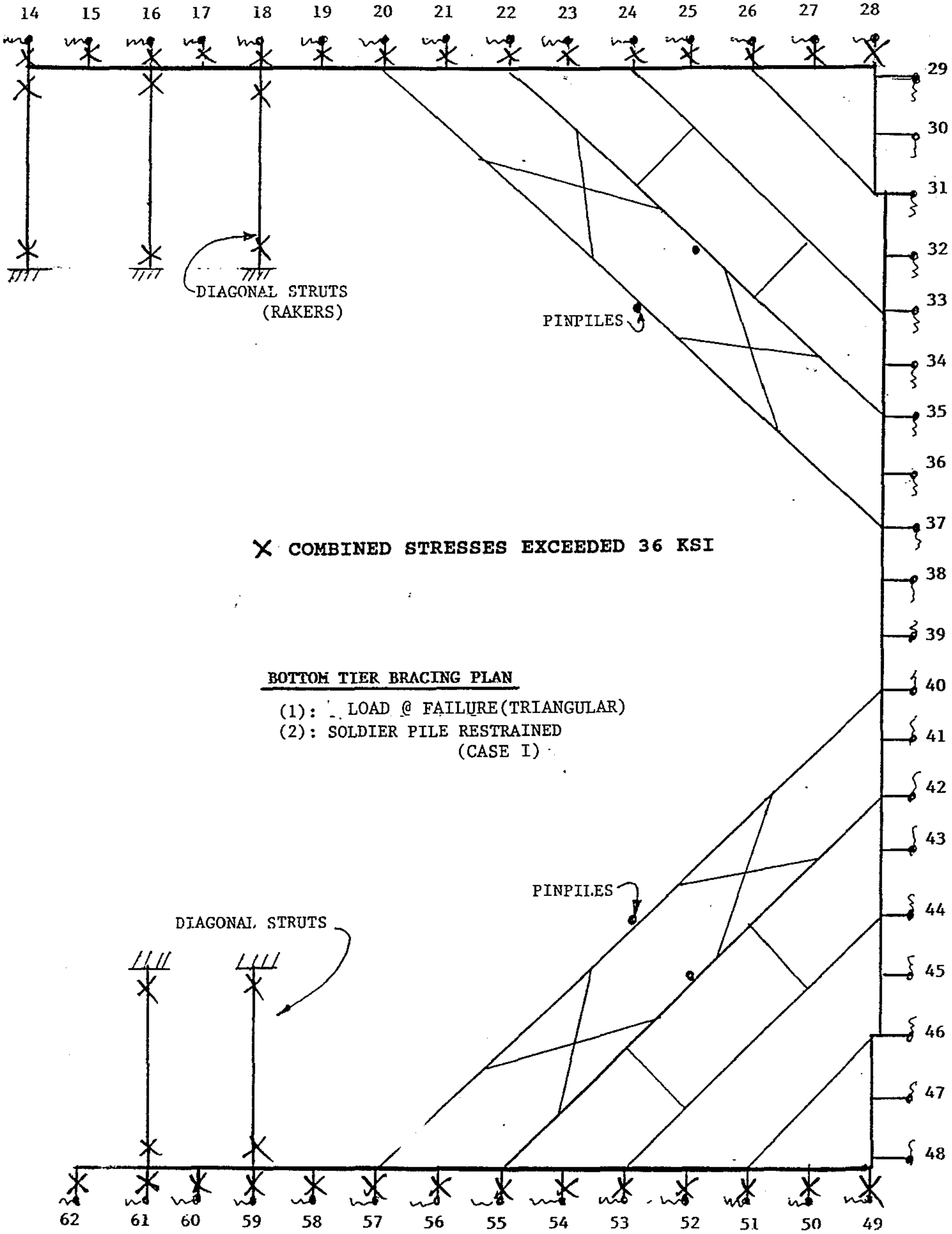
FIGURE 4.21



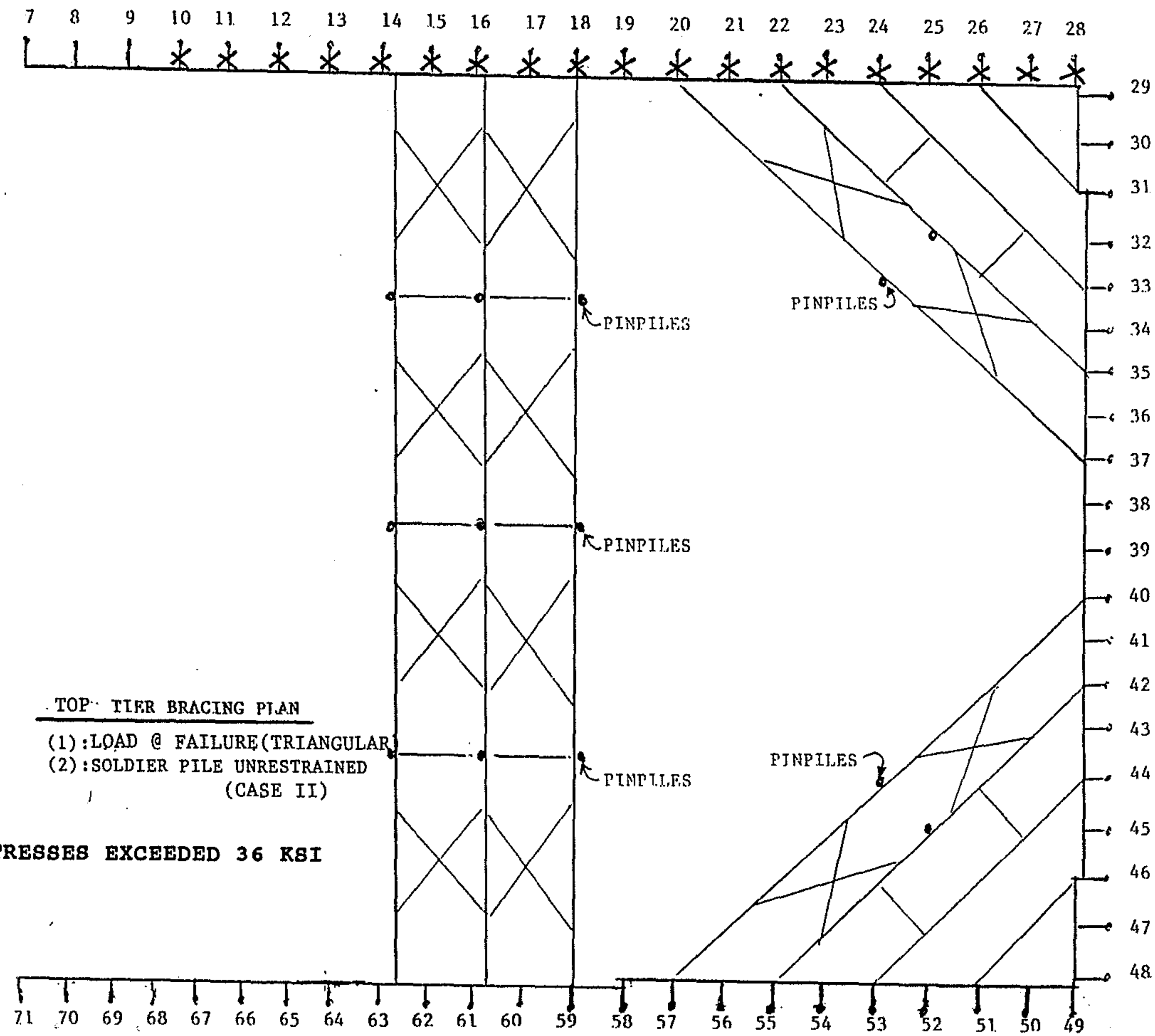
X COMBINED STRESSES EXCEEDED 36 KSI

FIGURE 4.22



14 15 16 17 18 19 20 21 22 23 24 25 26 27 28
29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48
62 61 60 59 58 57 56 55 54 53 52 51 50 49
DIAGONAL STRUTS
(RAKERS)
PINPILES
COMBINED STRESSES EXCEEDED 36 KSI
BOTTOM TIER BRACING PLAN
(1): LOAD @ FAILURE(TRIANGULAR)
(2): SOLDIER PILE RESTRAINED
(CASE I)
PINPILES
DIAGONAL STRUTS

FIGURE 4.23



X COMBINED STRESSES EXCEEDED 36 KSI

FIGURE 4.24

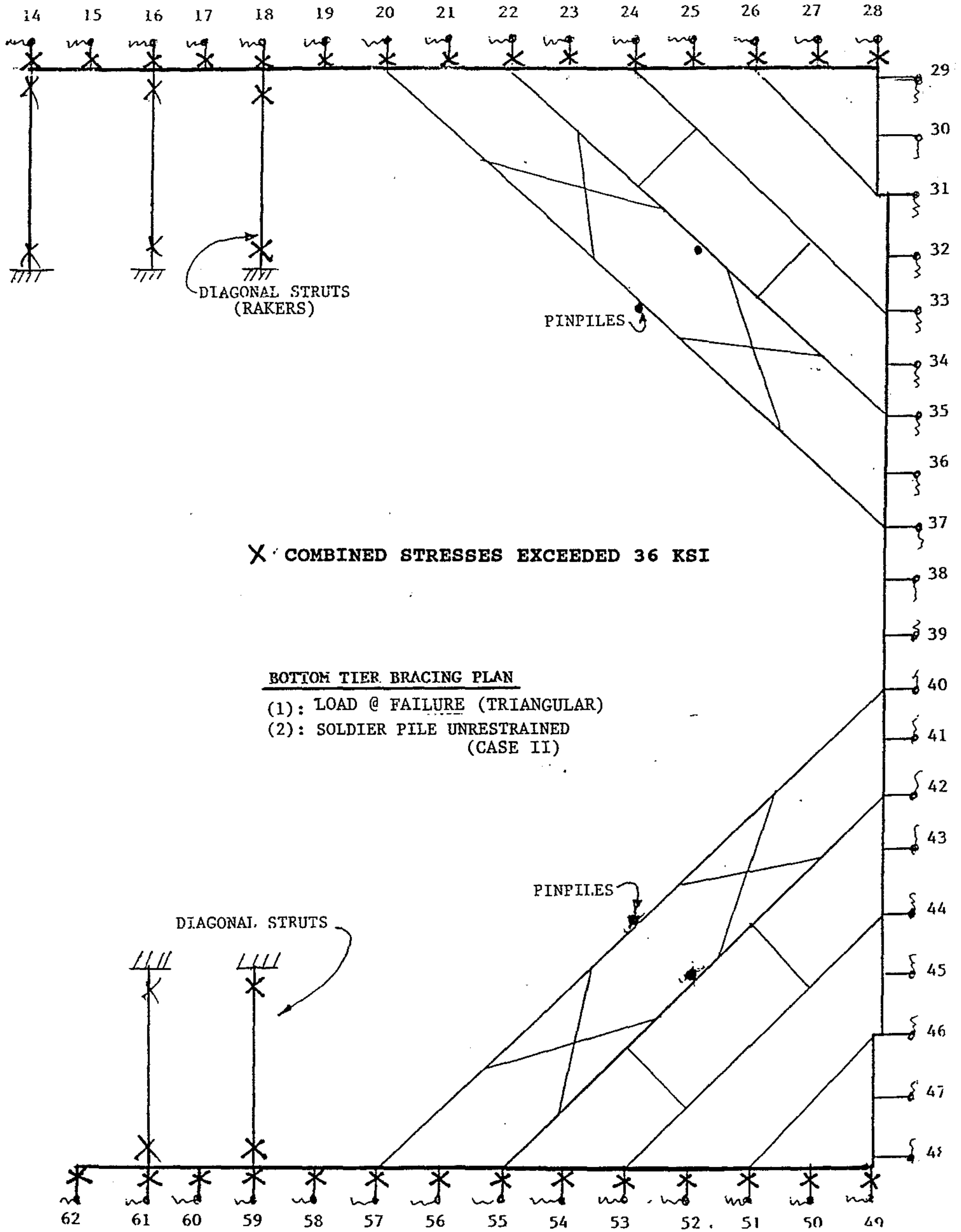
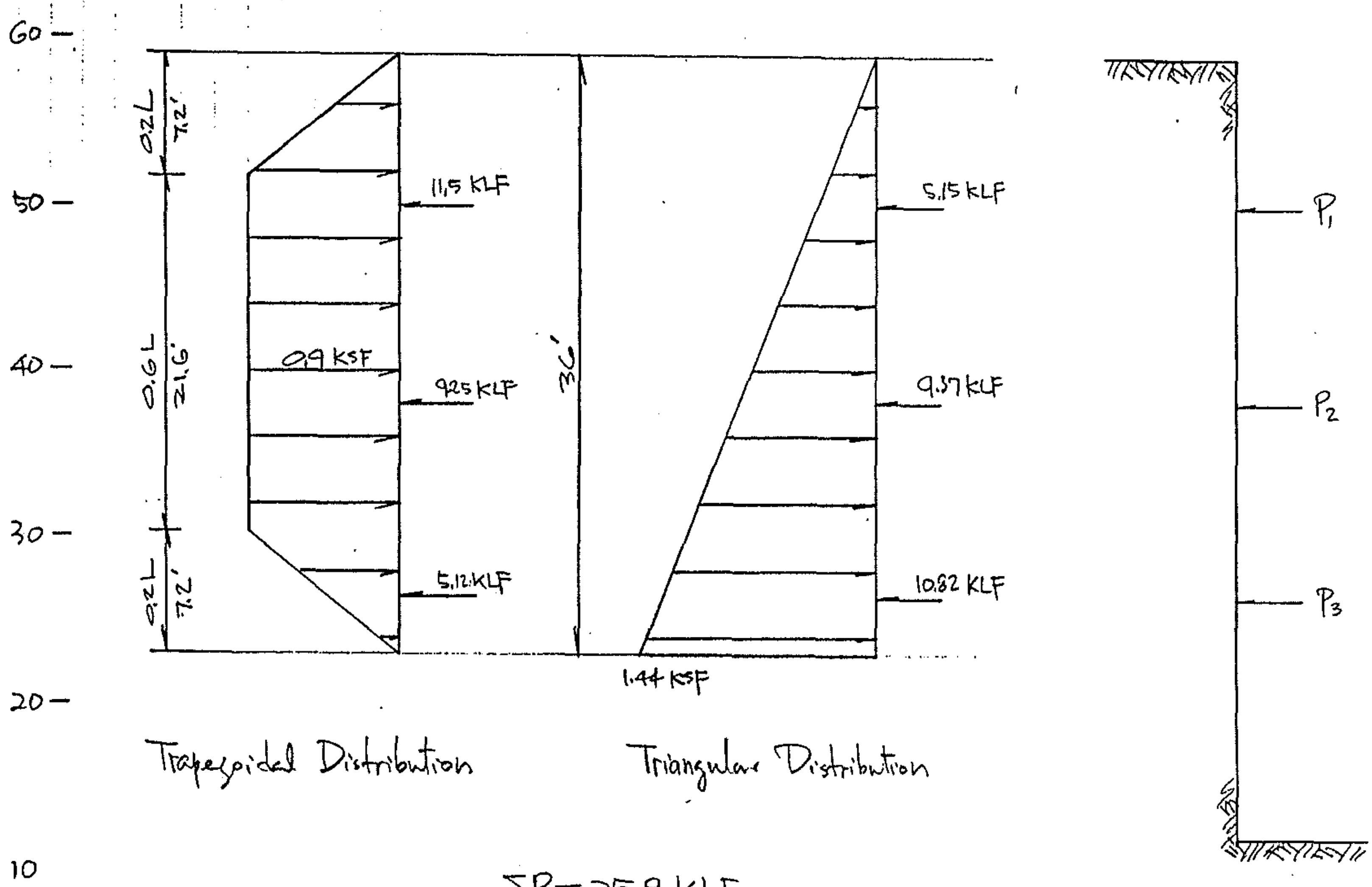

14 15 16 17 18 19 20 21 22 23 24 25 26 27 28
29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48
62 61 60 59 58 57 56 55 54 53 52 51 50 49
DIAGONAL STRUTS
(RAKERS)
PINPILES
X COMBINED STRESSES EXCEEDED 36 KSI
BOTTOM TIER BRACING PLAN
(1): LOAD @ FAILURE (TRIANGULAR)
(2): SOLDIER PILE UNRESTRAINED
(CASE II)
PINPILES
DIAGONAL STRUTS

FIGURE 4.25

Soldier Beams 14 through 63



$\Sigma P = 25.9$ KLF

Note: For soldier beams 14 through 23, earth pressure starts at El. 60.

SOIL PRESSURE IMMEDIATELY PRIOR TO THE COLLAPSE

FIGURE 4.26

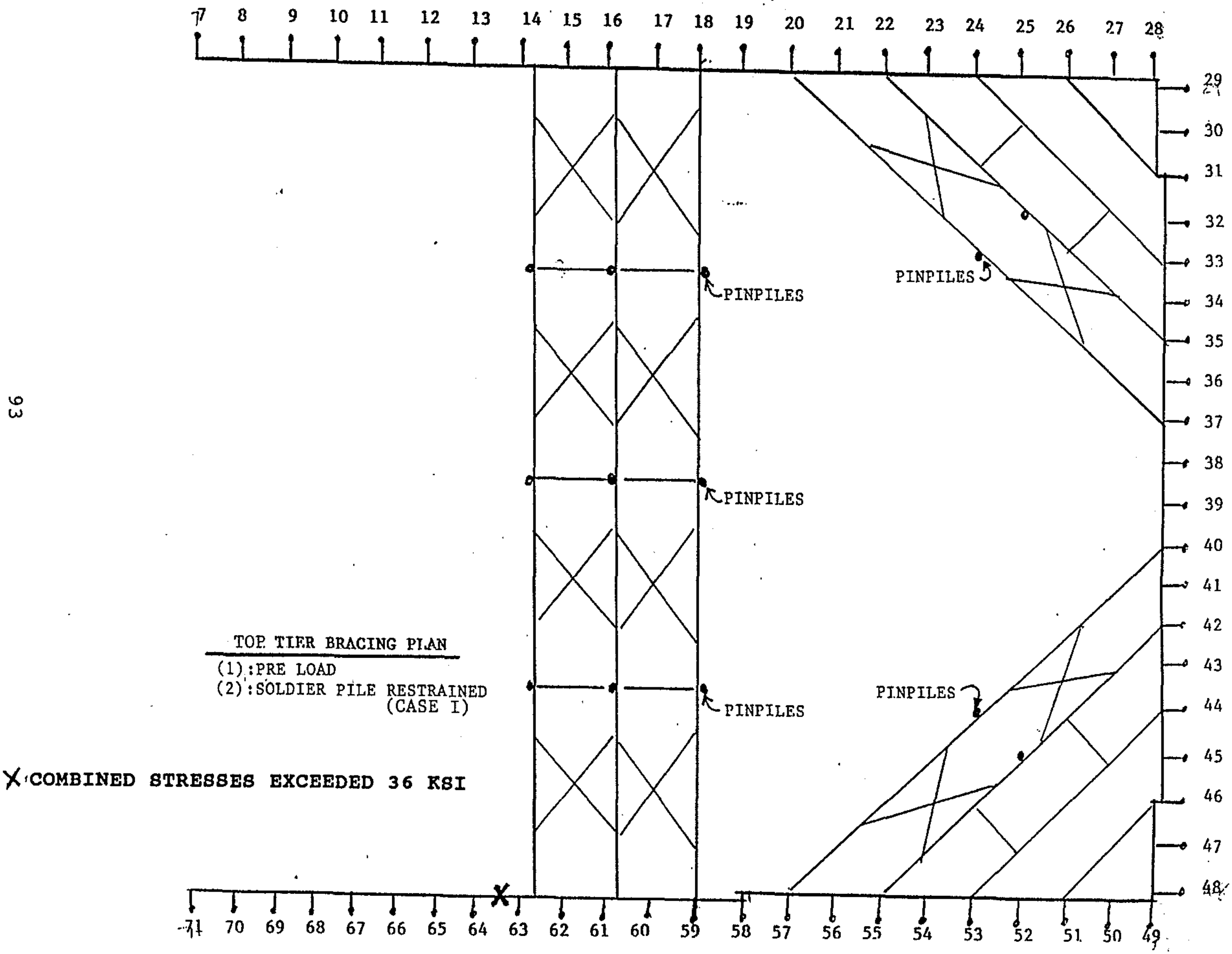
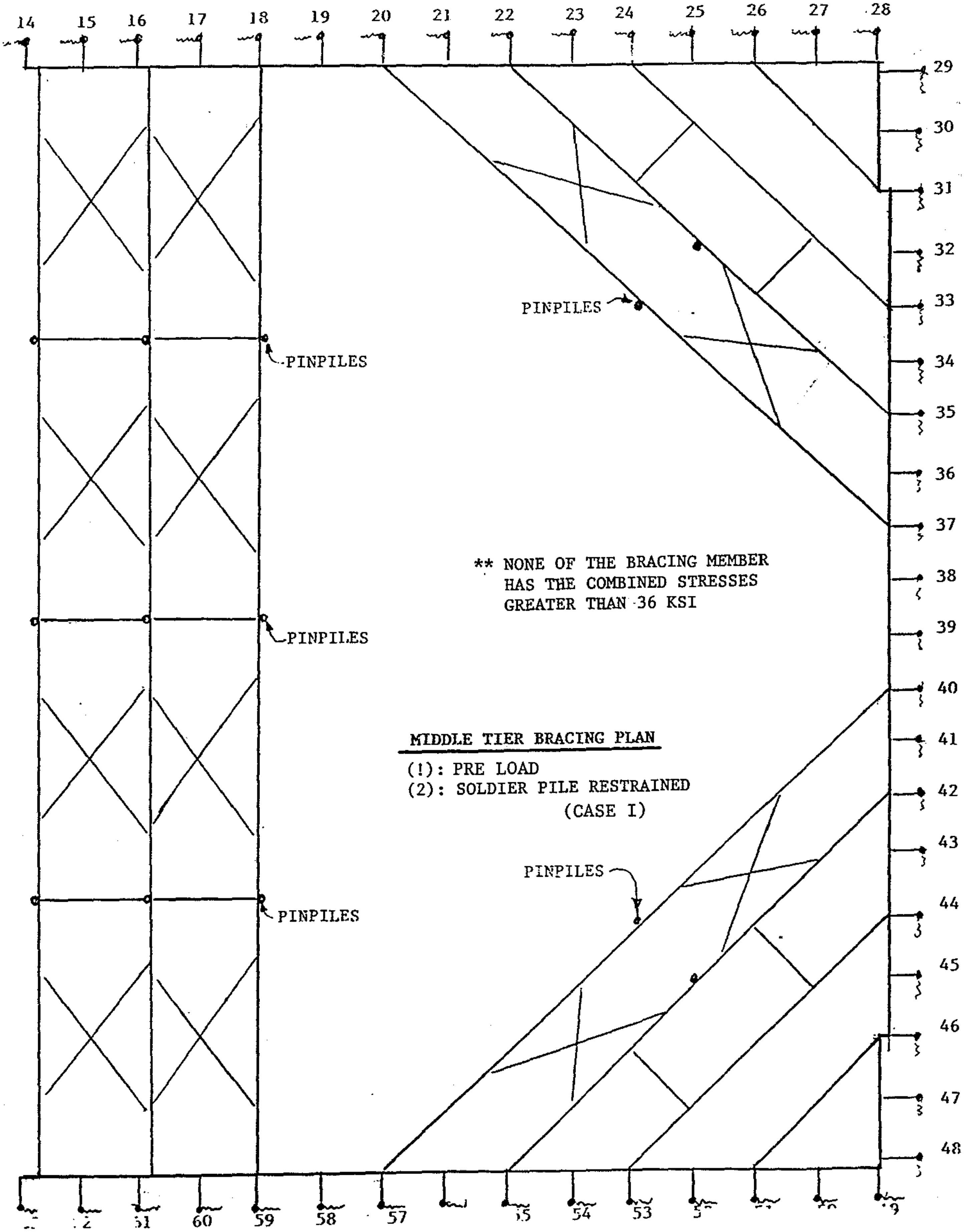

7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28
29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48
49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66 67 68 69 70 71
PINPILES
PINPILES
PINPILES
PINPILES
PINPILES
TOP TIER BRACING PLAN
(1):PRE LOAD
(2):SOLDIER PILE RESTRAINED
(CASE I)
X COMBINED STRESSES EXCEEDED 36 KSI

FIGURE 4.27


14
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33
34
35
36
37
38
39
40
41
42
43
44
45
46
47
48
PINPILES
PINPILES
PINPILES
PINPILES
PINPILES
** NONE OF THE BRACING MEMBER
HAS THE COMBINED STRESSES
GREATER THAN 36 KSI
MIDDLE TIER BRACING PLAN
(1): PRE LOAD
(2): SOLDIER PILE RESTRAINED
(CASE I)
61
60
59
58
57
54
53

FIGURE 4.28

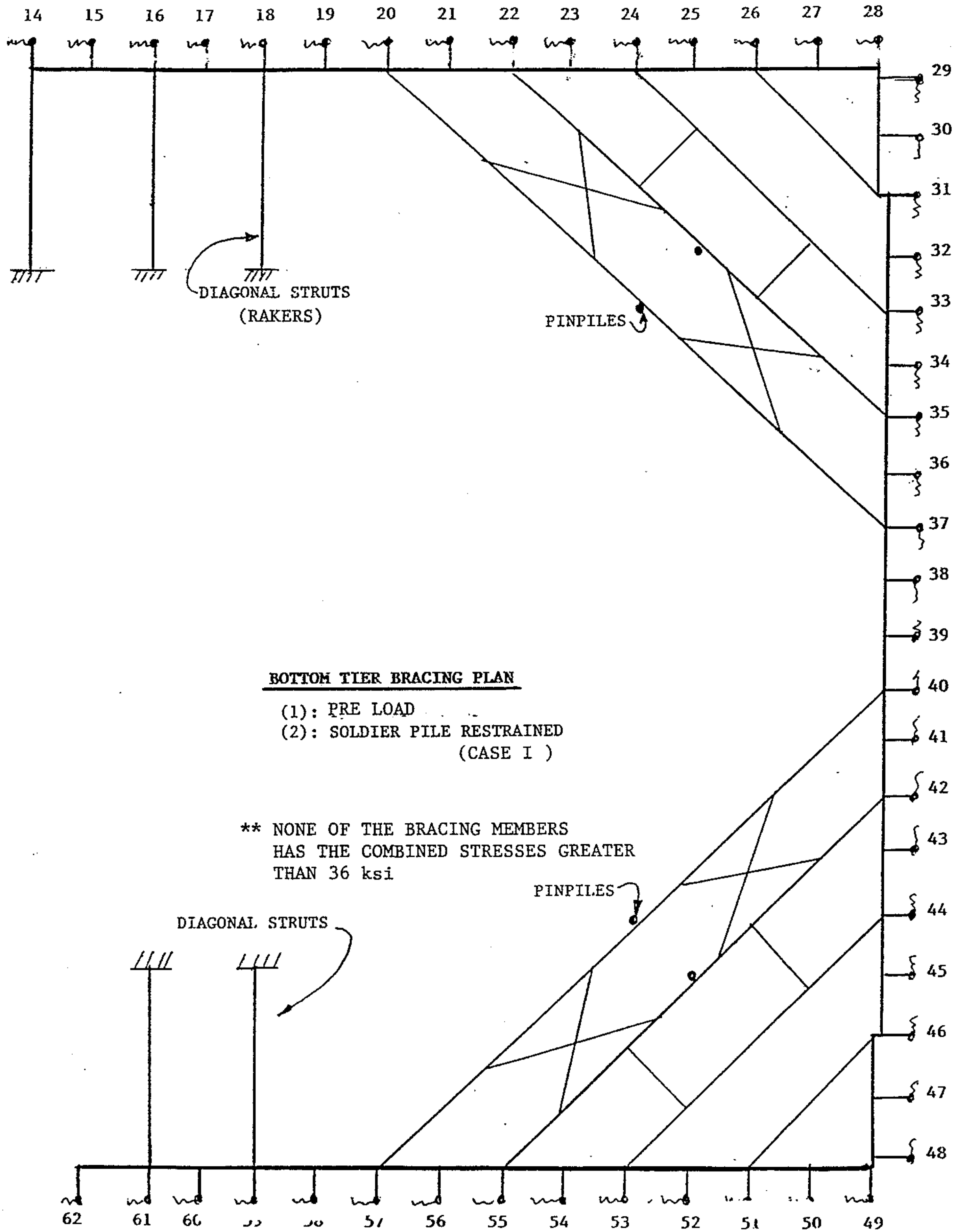
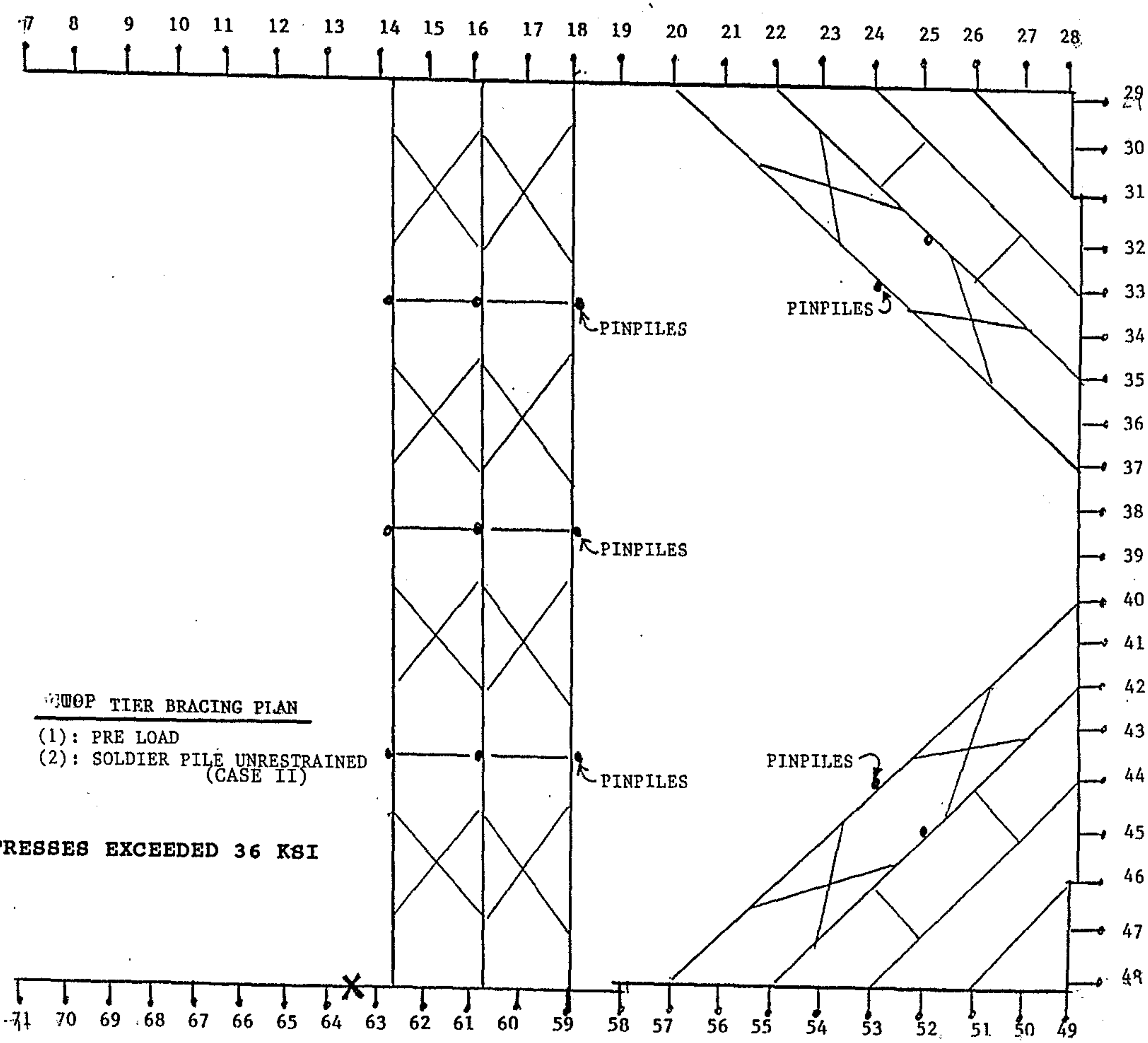

14 15 16 17 18 19 20 21 22 23 24 25 26 27 28
29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48
62 61 56 55 54 53 52 50 49
DIAGONAL STRUTS
(RAKERS)
PINPILES
BOTTOM TIER BRACING PLAN
(1): PRE LOAD
(2): SOLDIER PILE RESTRAINED
(CASE I)
** NONE OF THE BRACING MEMBERS
HAS THE COMBINED STRESSES GREATER
THAN 36 ksi
PINPILES
DIAGONAL STRUTS

FIGURE 4.29



X: COMBINED STRESSES EXCEEDED 36 KSI

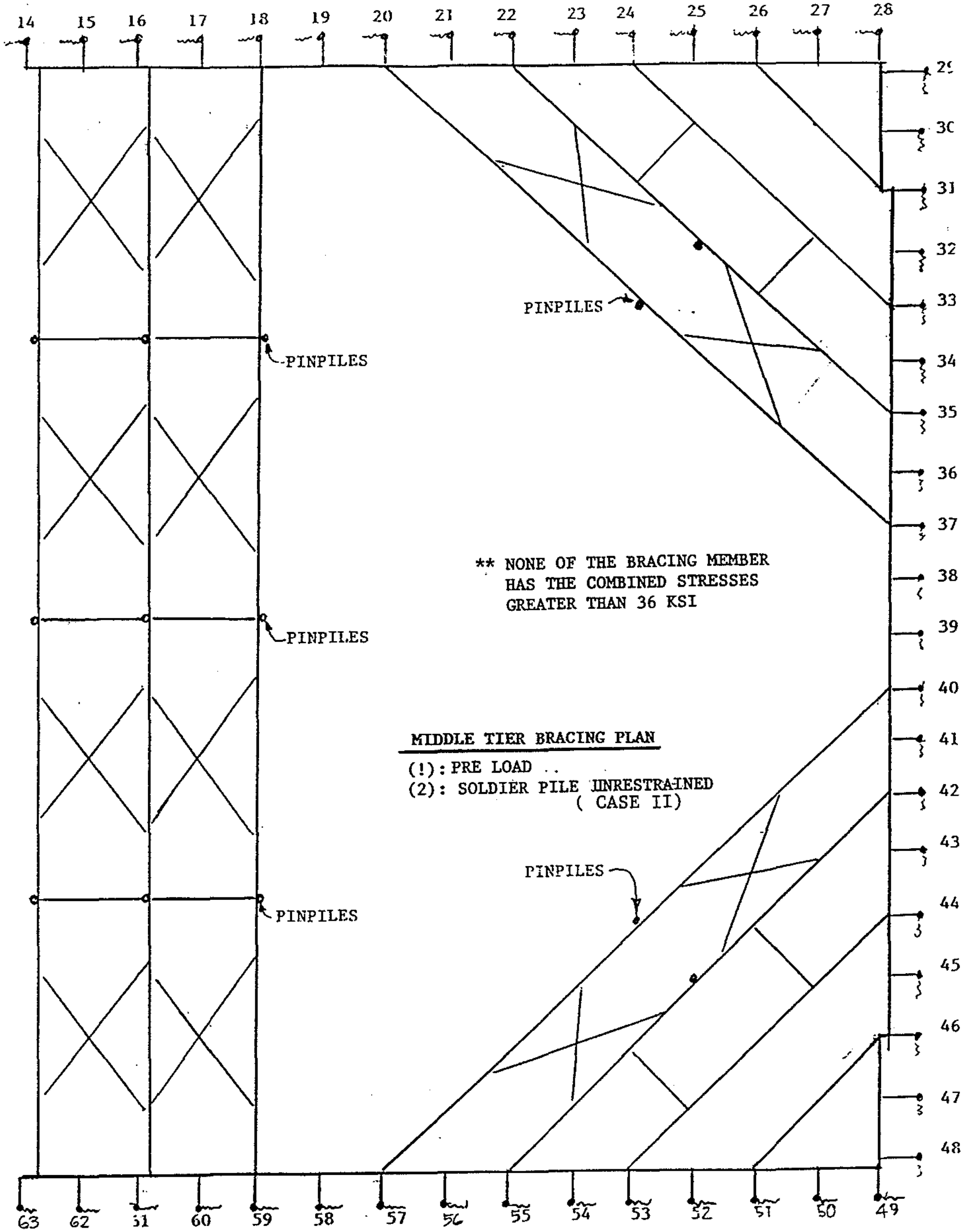

14 15 16 17 18 19 20 21 22 23 24 25 26 27 28
29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48
63 62 61 60 59 58 57 56 55 54 53 52 51 50 49
PINPILES
PINPILES
PINPILES
PINPILES
PINPILES
** NONE OF THE BRACING MEMBER HAS THE COMBINED STRESSES GREATER THAN 36 KSI
MIDDLE TIER BRACING PLAN
(1): PRE LOAD
(2): SOLDIER PILE UNRESTRAINED (CASE II)

FIGURE 4.31

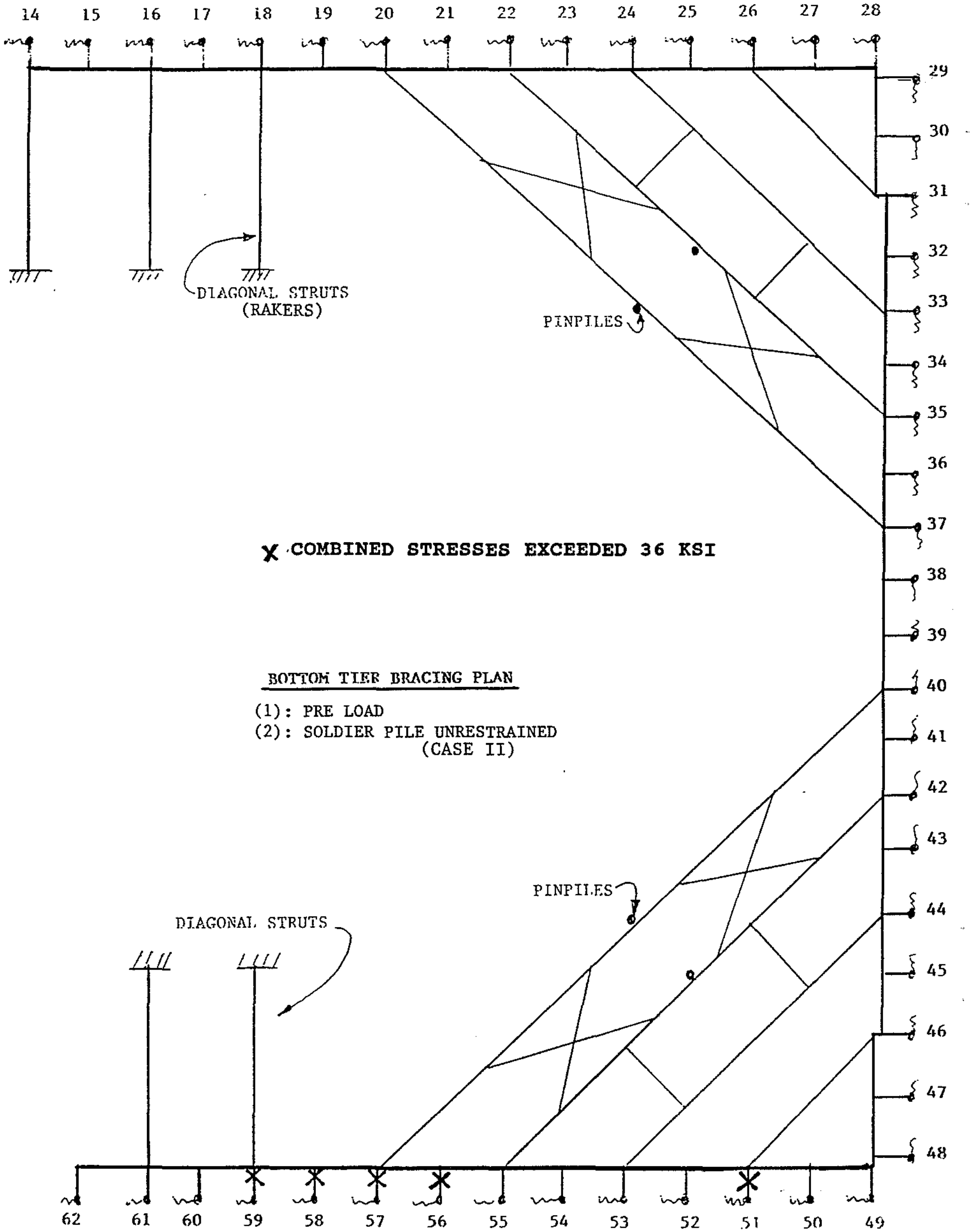

14 15 16 17 18 19 20 21 22 23 24 25 26 27 28
29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48
62 61 60 59 58 57 56 55 54 53 52 51 50 49
DIAGONAL STRUTS
(RAKERS)
PINPILES
X COMBINED STRESSES EXCEEDED 36 KSI
BOTTOM TIER BRACING PLAN
(1): PRE LOAD
(2): SOLDIER PILE UNRESTRAINED
(CASE II)
PINPILES
DIAGONAL STRUTS

FIGURE 4.32

5. CONCLUSIONS

The following conclusions by the Occupational Safety and Health Administration are based on the examination of the collapsed structure, the witness interviews and the geotechnical and structural analysis discussed in this report.

1. The collapse of the excavation occurred due to the failure of certain structural members of the internal support system on the north and south walls.

2. The external support system consisting of tiebacks, soldier beams and tieback wales which were in place on the west wall and part of the north and south walls did not fail.

3. Subsequent to the collapse, all soldier beams, rakers and pinpiles continued to remain embedded below the final subgrade.

4. The full soil pressures developed and used in this report agreed with the expected loads assumed by the shoring subcontractor for the project to design the internal support system and were deemed to be consistent with the local practice.

5. The north and south wall outlookers and their welded connections at all three tiers were not properly proportioned to resist the expected load.

6. The sloping wale on the south wall at the top tier between soldier beams 63 and 64 was not properly proportioned to resist the expected load.

7. The wales at all three tiers on the east wall between soldier beams 37 to 40 were not properly proportioned for the expected load.

8. The failure load, the soil pressure immediately preceding the collapse, was determined to be lower than the expected load for which the structure was designed by the shoring subcontractor for the project.

9. The shoring subcontractor overestimated the passive earth resistance to movement parallel to the north and south walls.

10. It is unlikely that either the groundwater pressure or the construction deficiencies discussed in Section 3.3.4 initiated or contributed to the collapse.

6. REFERENCES

1. Soil Mechanics in Engineering Practice, by Karl Terzaghi and Ralph B. Peck, John Wiley and Sons, Inc., 1968.

2. Adjacent Construction Design Manual, by Office of Engineering and Architecture, Washington Metropolitan Area Transit Authority, 1987.

3. Foundation Design, by Wayne C. Teng, Prentice-Hall, Inc., 1962.

4. Manual of Steel Construction Allowable Stress Design, by American Institute of Steel Construction, Inc., Ninth Edition, 1989.

5. STAAD-III/Integrated Structural Design System, by Research Engineers, 1990.

6. Structural Standard Drawing No. ST-S-9, by Washington Area Transit Authority, 1970.

APPENDIX A

GEOTECHNICAL AND STRUCTURAL COMPUTATIONS

A.1 Computaiton of Total Horizontal Pressure

A.2 Computation of Earth Resistance Tangential to the Excavation Wall

A.3 Computation of Geotechnical Parameters for Computer Analysis

A.4 Estimation of Failure Loads

A.5 Manual Analysis of Internal Support System

A.6 Analysis of Outlooker Welding Connections

APPENDIX B

COMPUTER ANALYSIS RESULTS