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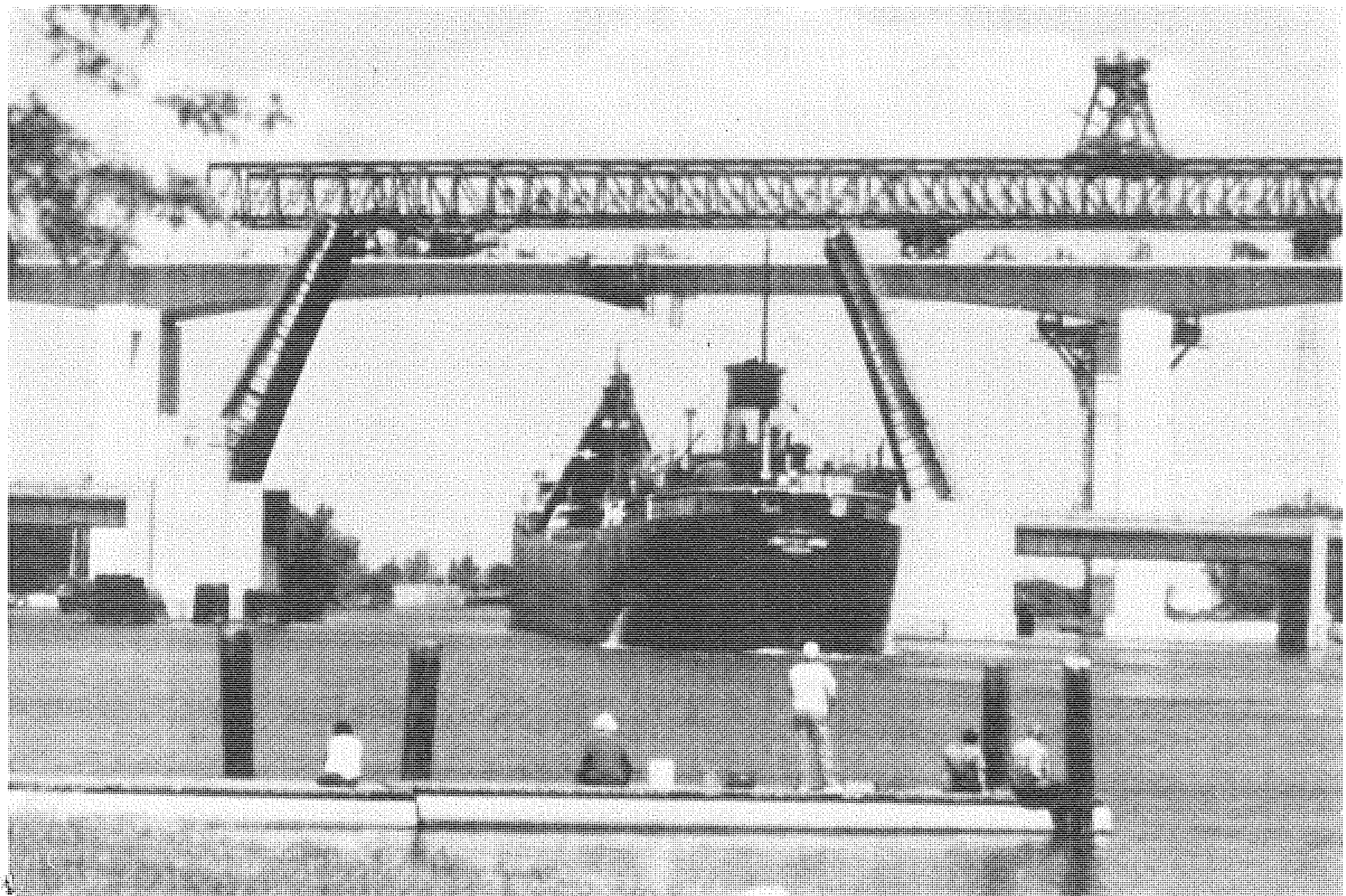
MONITORING MOVEMENT OF THE ZILWAUKEE BRIDGE



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Monitoring movement of the
Zilwaukee Bridge

MONITORING MOVEMENT OF THE ZILWAUKEE BRIDGE

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Research Laboratory Section
Testing and Research Division
Research Project No. 82 TI-849
Research Report No. R-1250

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SUMMARY

Shortly after the August 28, 1982 Zilwaukee Bridge accident, the Testing and Research Division's Structural Research Unit was asked to monitor movements of the damaged structure to determine whether or not it was stable. Monitoring began in the fall of 1982, continued during repair operations in the summer of 1983, and ended when the superstructure was rotated back into place on March 22, 1984.

The structure proved to be stable throughout the period, with virtually the only movement being caused by temperature changes. It was demonstrated that the monitoring could be done with a considerable degree of accuracy. The Sperry tilt sensing system proved to be quite accurate, and correlated closely with the mechanical measuring devices used.

This report describes how the measurements were taken and analyzes the significant findings. Long-term trends concerning temperature-induced movement are established.

INTRODUCTION

Project Description

The Zilwaukee Bridge consists of two parallel segmental concrete bridges which will carry north and southbound Interstate 75 traffic over the Saginaw River (Fig. 1). When completed, the new bridge will replace an existing low-level bascule bridge which was built about 1960 (Fig. 2).

Each bridge is approximately 1-1/2 miles long and consists of three 12-ft driving lanes, one 12-ft truck-climbing lane, and two 11-ft shoulders. This length is necessary to obtain a 125-ft clearance over the shipping channel and maintain the 3-percent maximum grade which is an Interstate system standard.

The southbound bridge consists of 26 spans and the northbound 25, varying in length from 131 to 392 ft, the longest spanning the river. The two mainline decks are made up of 1,592 concrete segments, averaging 73 ft 6 in. in width and 140 tons in weight, with 1,656 segments overall. The segment depths range from 8 ft at midspan to 20 ft over the piers (Fig. 3).

The superstructure is supported on twin reinforced hollow concrete columns. The columns are supported by concrete footings on steel H-piles, driven to bedrock or to refusal (Figs. 4 and 5).

A long steel truss launching girder was used to place the segments and hold them while grout and a temporary prestress force were applied.

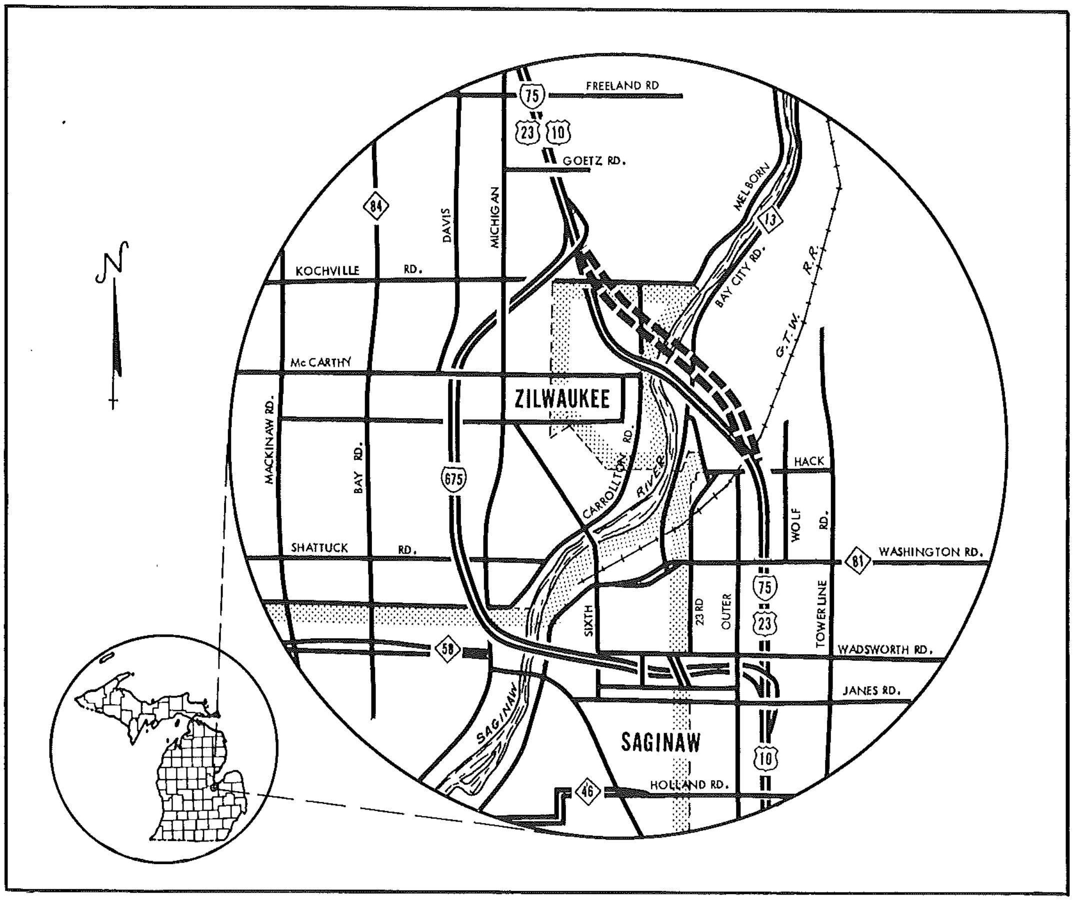


Figure 1. Site of the segmental concrete bridge at Zilwaukee.

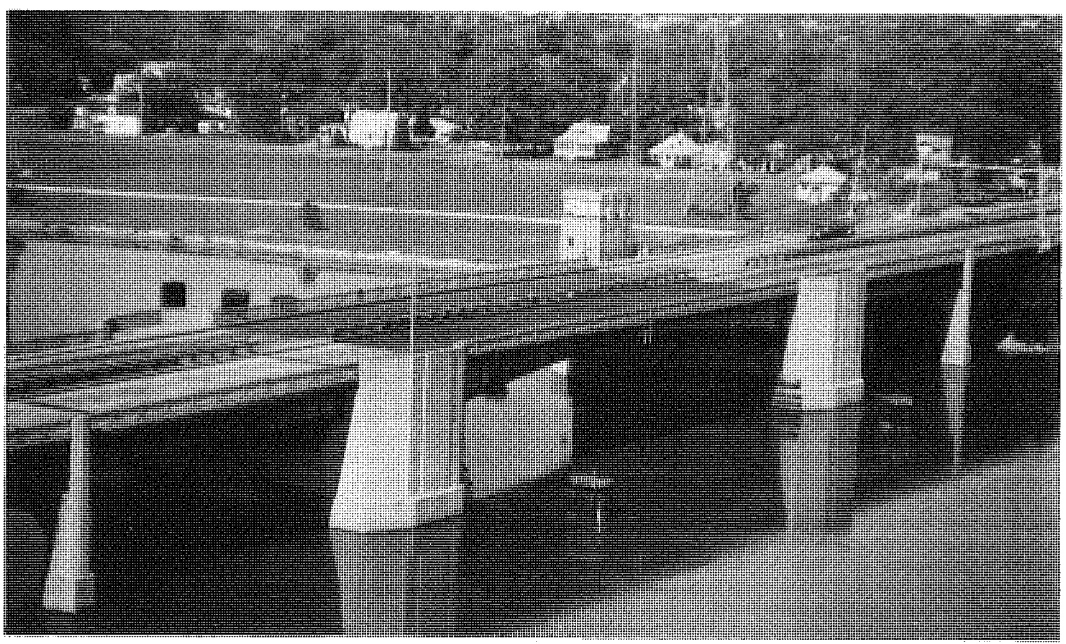
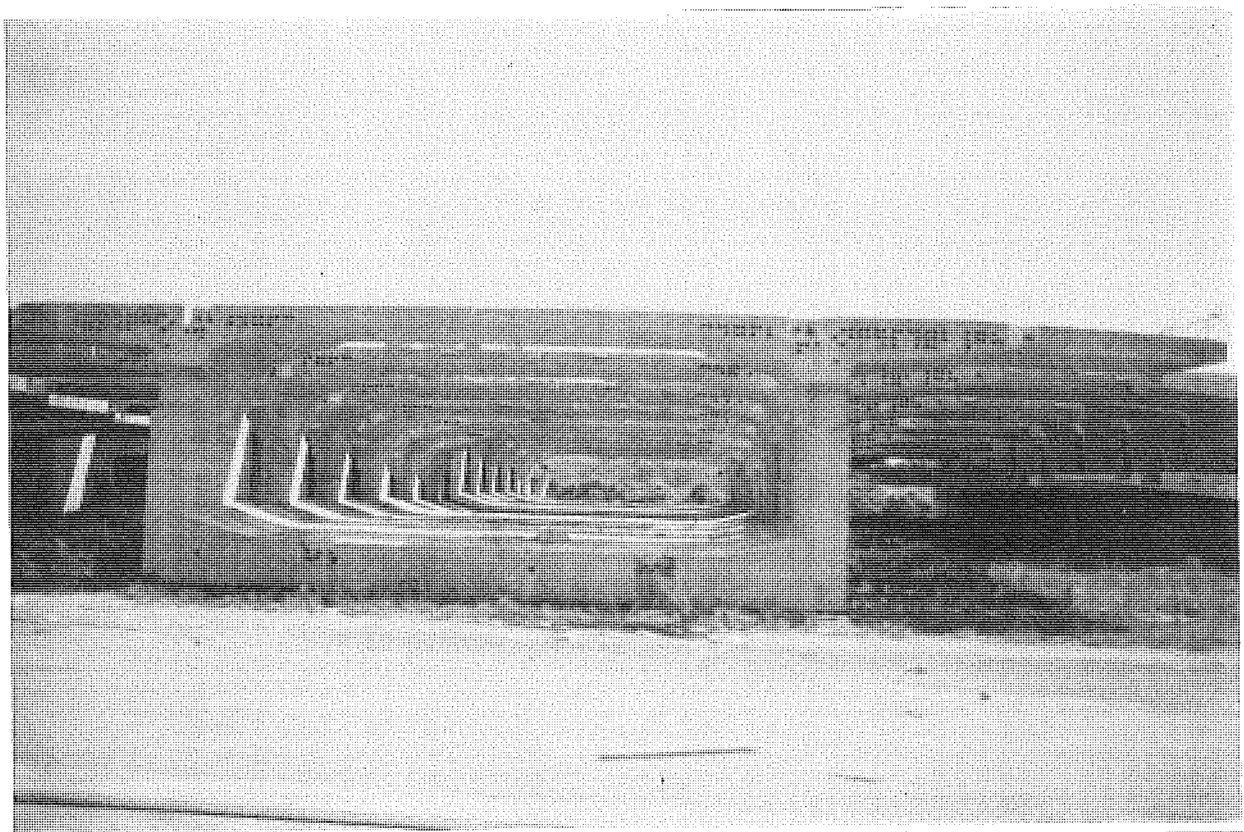


Figure 2. Existing low level bascule bridge.



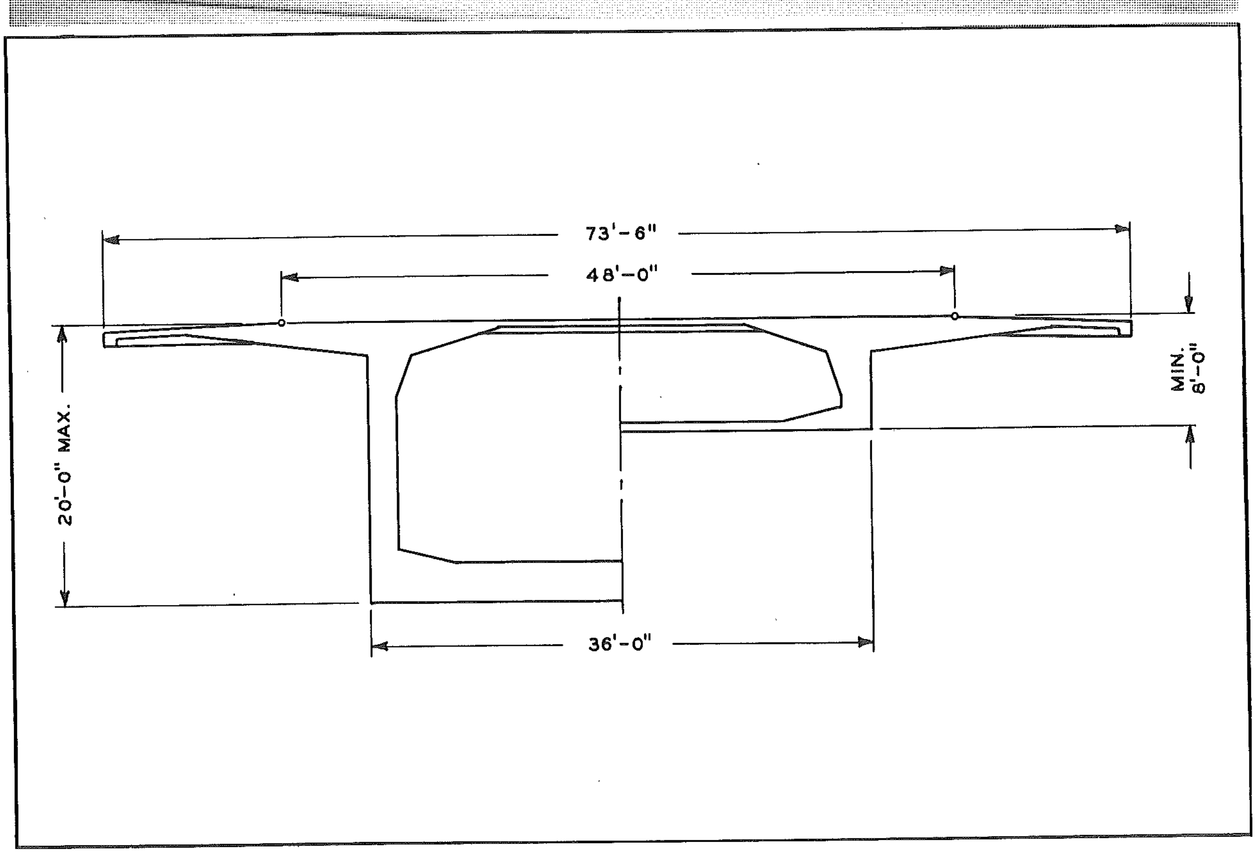
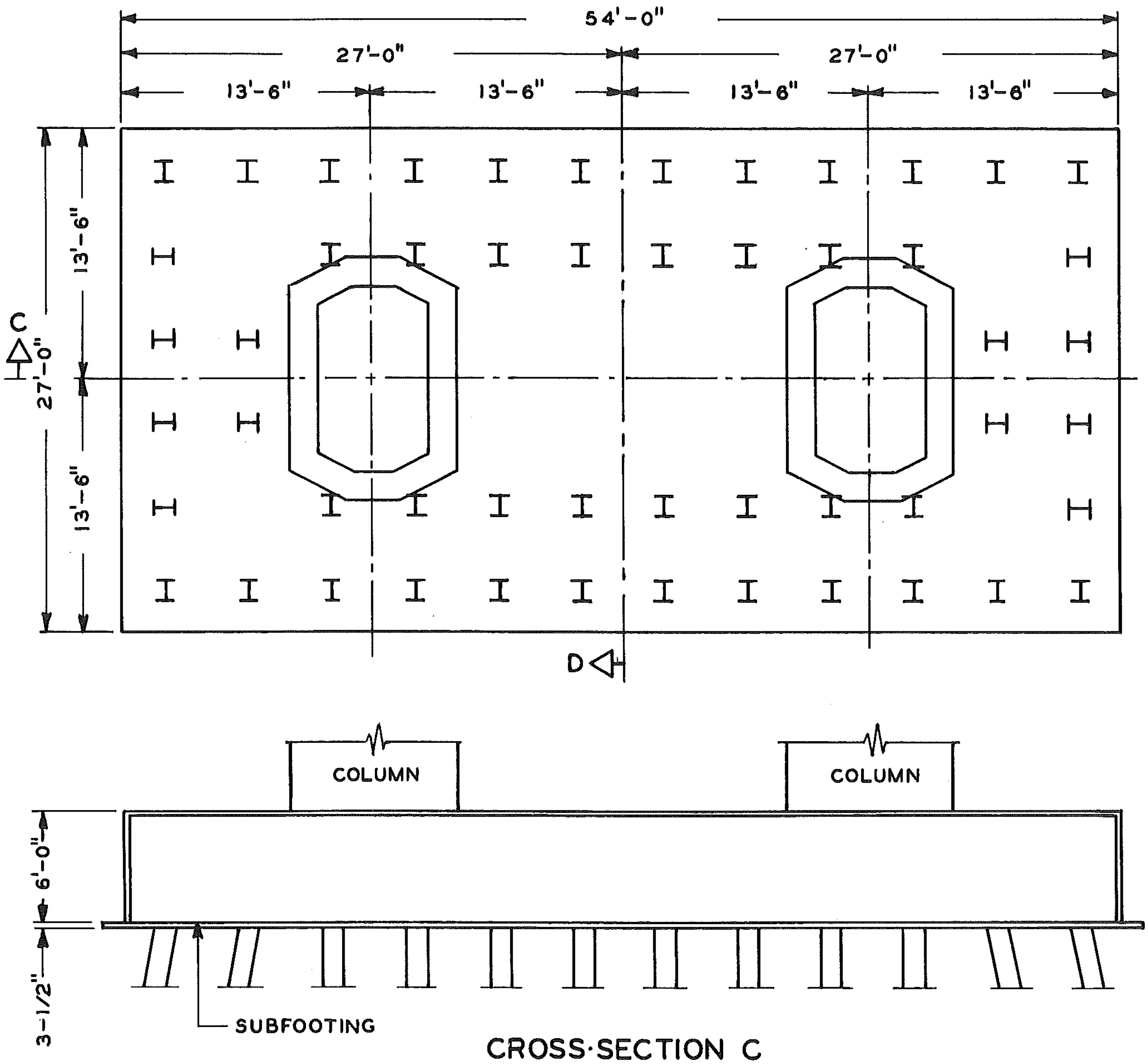


Figure 3. Cross-sectional views of concrete segments.

Figure 4. Steel H-piling was driven to support the column footings (left). A partially completed footing site is shown above.



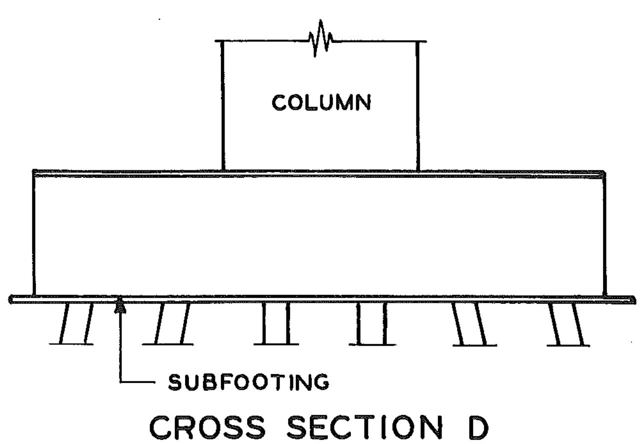


Figure 5. Layout of footing and columns.

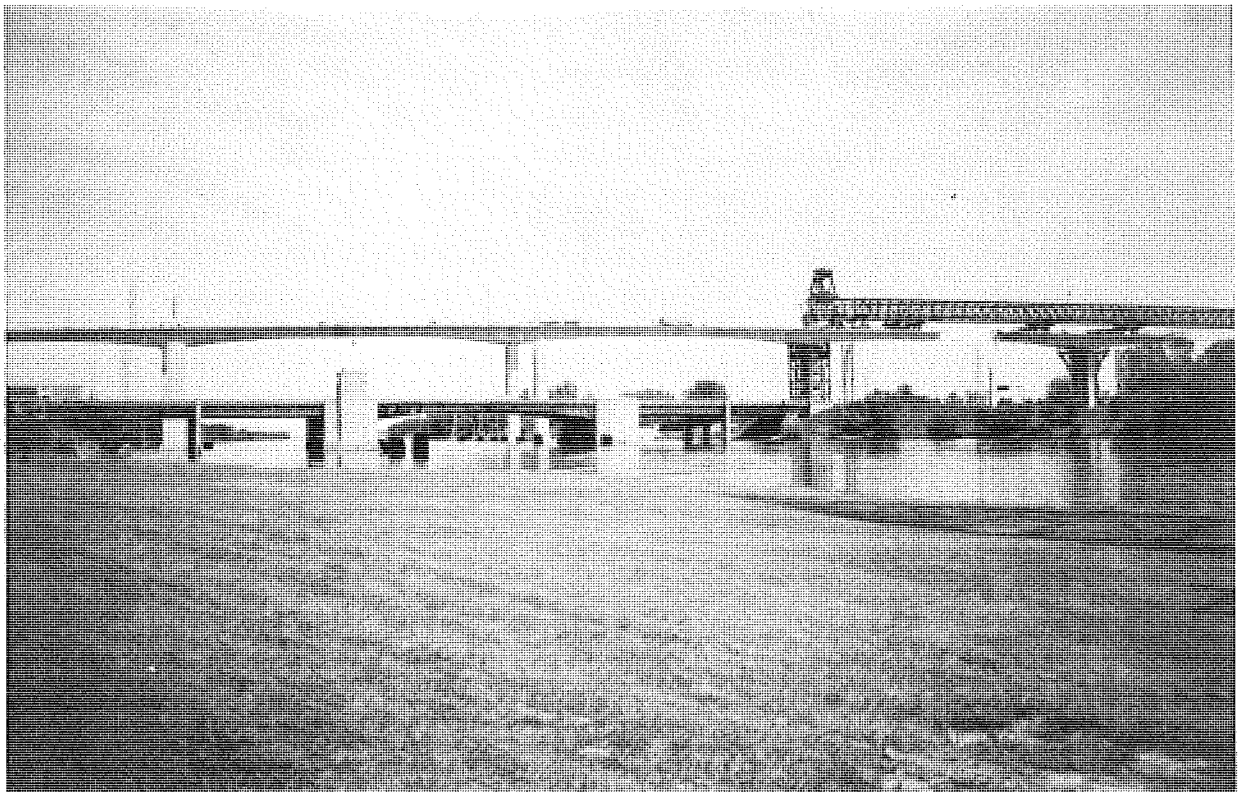


Figure 6. Launching girder (bridge completed through span 12).

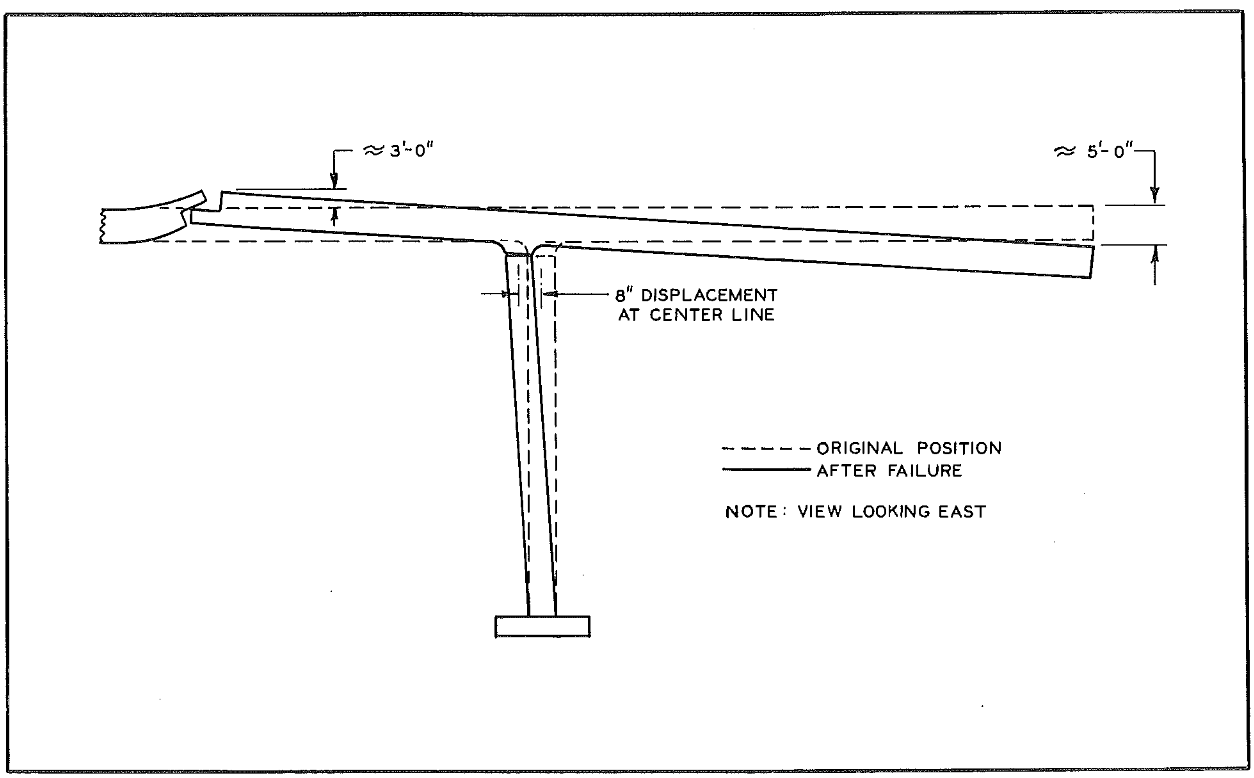


Figure 7. Position of deck before and after accident.

This process is described in detail in Appendix A. The launching girder is approximately 940 ft long and has an estimated weight of 1,200 tons (Fig. 6). Segment placement started on April 10, 1981 on cantilever 24N of the northbound structure and proceeded southward. Construction of the river span was completed in the summer of 1982.

Failure

Early in the morning of August 28, 1982, as a segment was being lifted by the gantry, a failure occurred. At the time, span 12 was completed, which included an expansion joint located 130 ft north of pier 11N; and span 11 was partially completed, cantilevering south of pier 11N approximately 170 ft.

Crushing occurred at the expansion joint; the south end of the cantilever deck deflected downward more than 5 ft, while the other end rose about 3 ft at the expansion joint. The top of the twin columns moved longitudinally to the north about 8 in. (Fig. 7). Excavation around the base of the footing and columns revealed that the footing had failed. Overall damage to the bridge is shown in Figure 8.

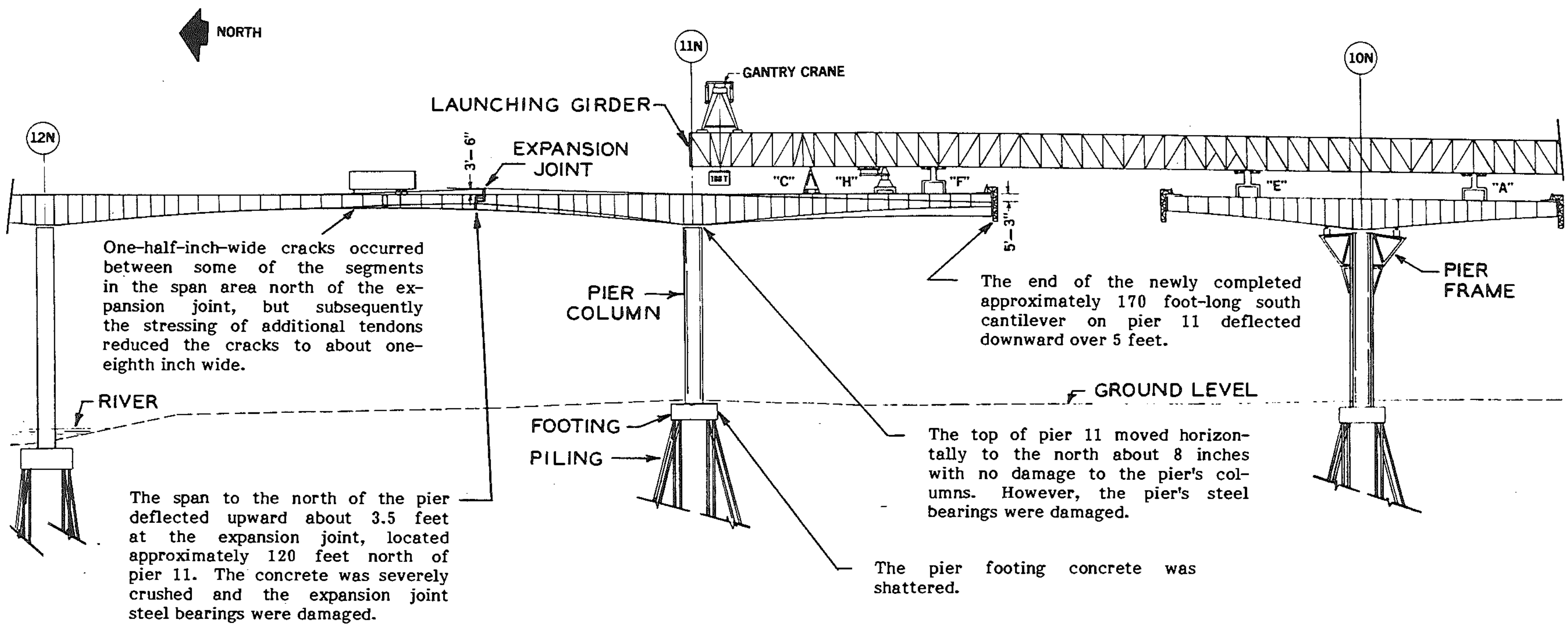
Physical damage to the superstructure included severe crushing of the spacer blocks and adjacent concrete located in the span 12 expansion joint (Fig. 9). Cracking occurred in the bottom of the box section between the expansion joint and pier 12N, and in some cases, cracking extended upward through the shear key. Cracking and spalling were evident on the bottom outside of the segments located at the damaged expansion joint (Fig. 10). The full extent of the damage to the expansion joint bearings was not determined at this time, but some plate bending in the pot bearings did occur.

The columns did not appear to be damaged, other than being out of plumb. The bearings at the top of the column were severely damaged and would have to be replaced (Fig. 11).

Through visual observation and ultrasonic testing, a crack on the top surface of the footing between the two columns appeared to run downward at a slight incline for several feet and then turned sharply downward to the bottom of the north edge of the footing (Fig. 12). Cracking was apparent on all four of the 6-ft high vertical faces of the footing. Downward slanting fractures of the footing were also found at the column-footing intersections proceeding down and away from the columns (Fig. 13).

Function of the Stuctural Research Unit

Shortly after the accident, the Testing and Research Division's Structual Research Unit was asked to monitor movements of the damaged structure. Initially, the primary objective was to document the movements of the structure and to determine whether it had stabilized. Natural movements of the bridge due to thermal expansion and contraction were monitored and patterns established. It was found that the structure



Source: Prepared by GAO from information provided by FHWA and MDOT. The United States General Accounting Office Report to the Honorable Donald W. Reigle, Jr., United States Senate, "Delays and Increased Cost Result from the Zilwaukee, Michigan Bridge Project Mishap," June 27, 1984.

Figure 8. Overall view of site where damage occurred (6).

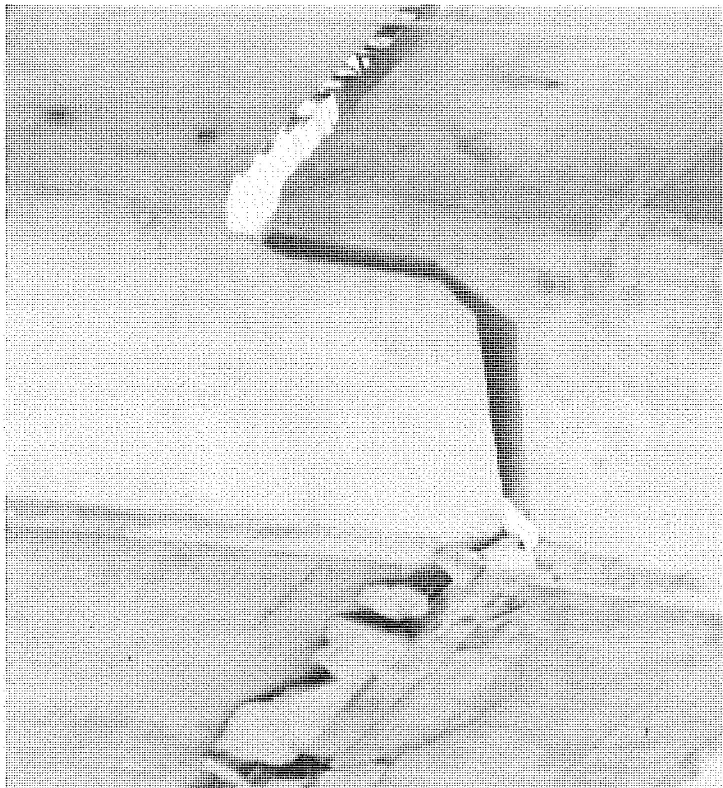


Figure 9. Crushing adjacent to spacer block in the bottom of the span 12 expansion joint.

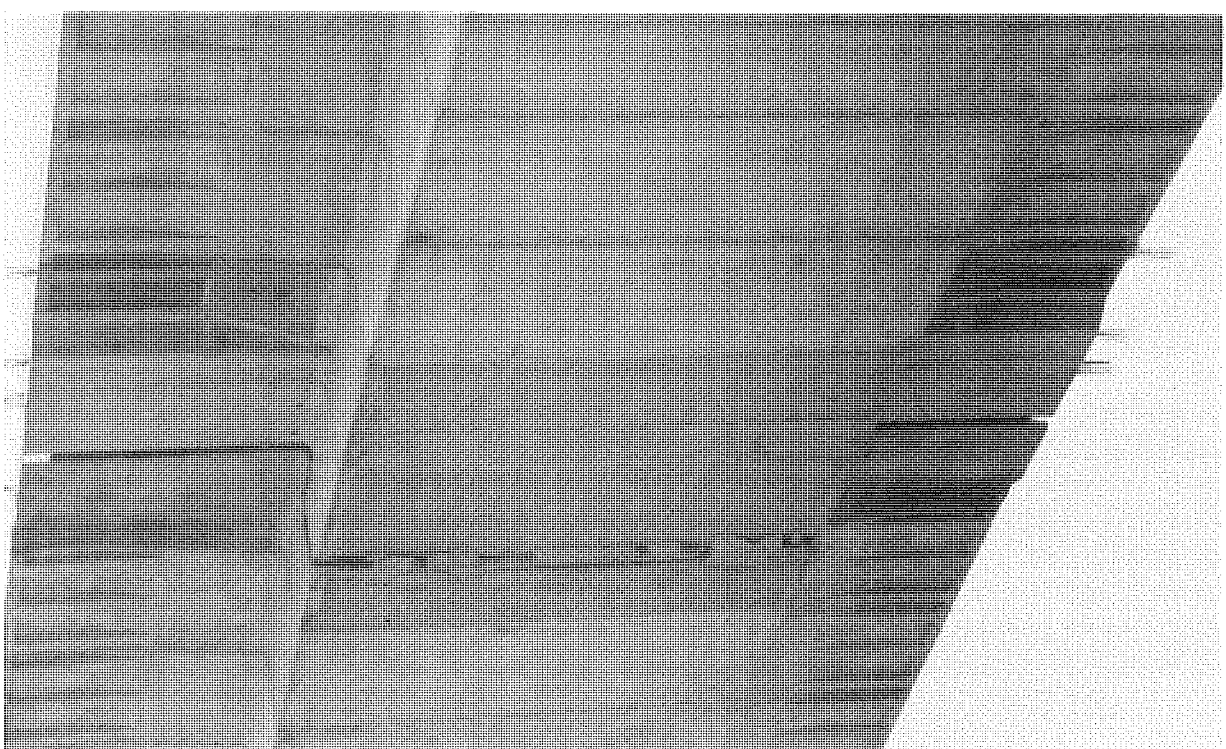
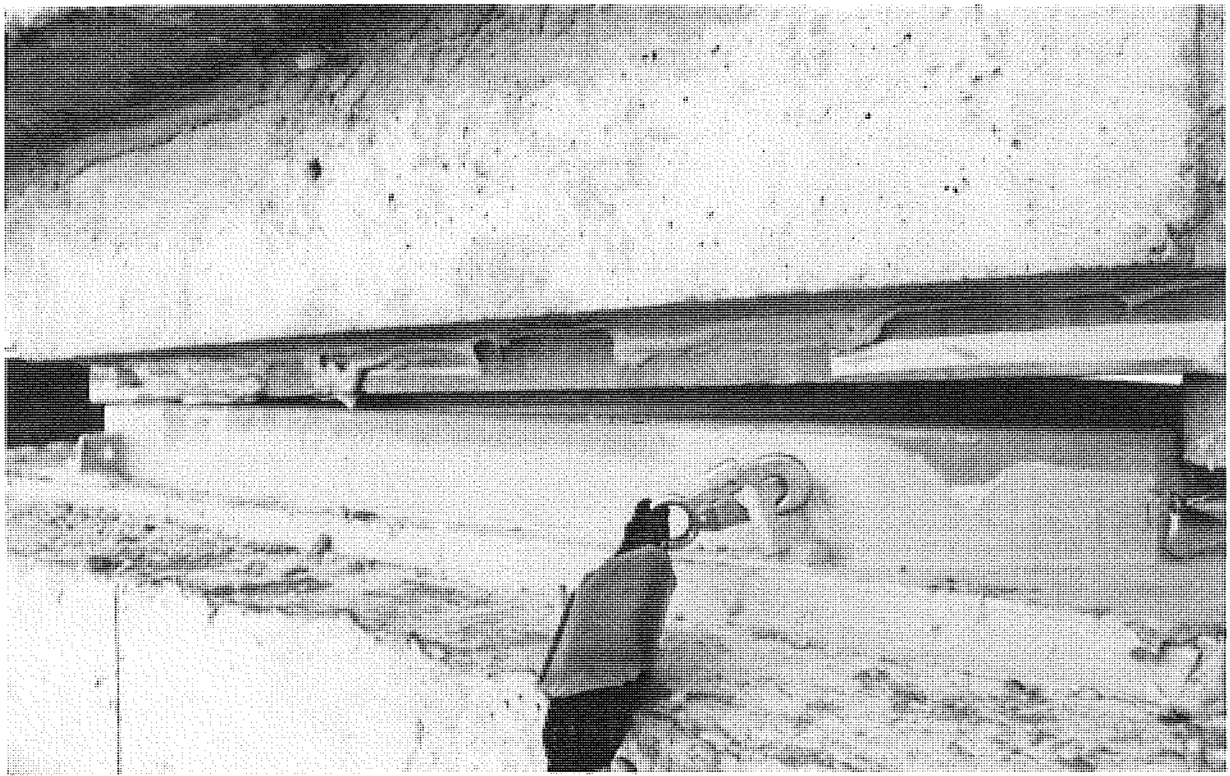


Figure 10. Spalling at the bottom of segments at the damaged joint.



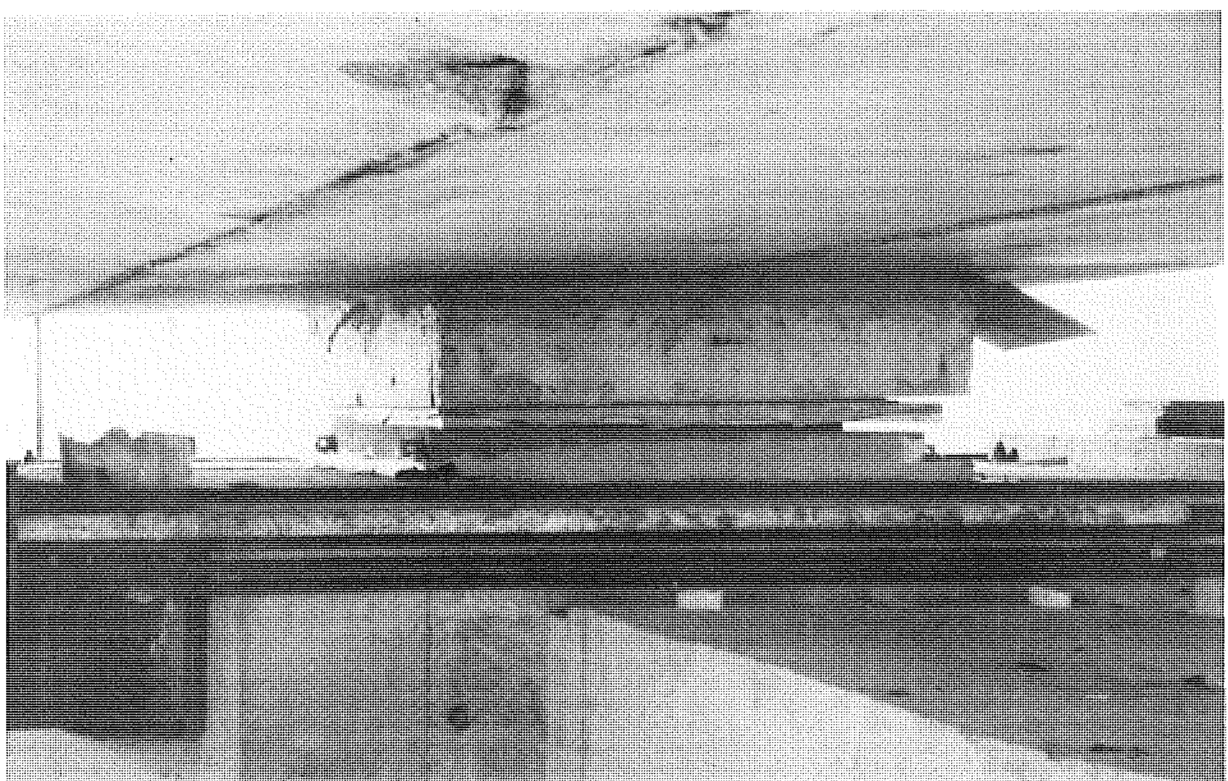


Figure 11. Damaged bearings over east (above) and west (below) columns.

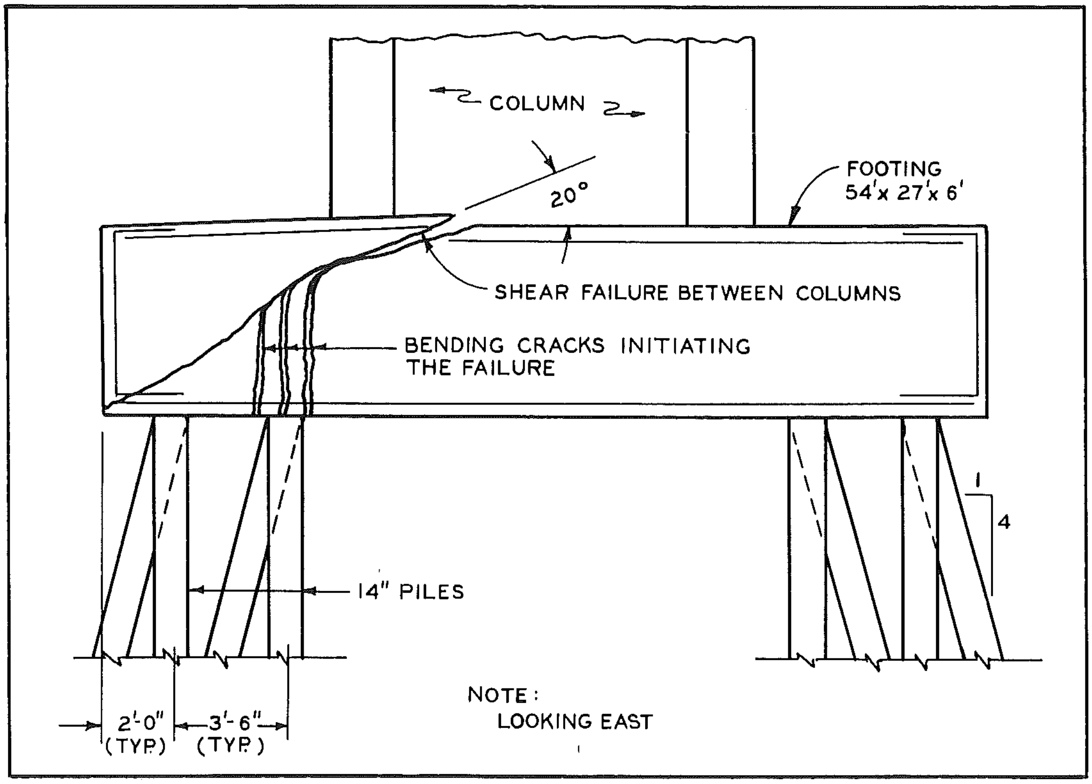


Figure 12. Diagram showing damage to footing of pier 11.

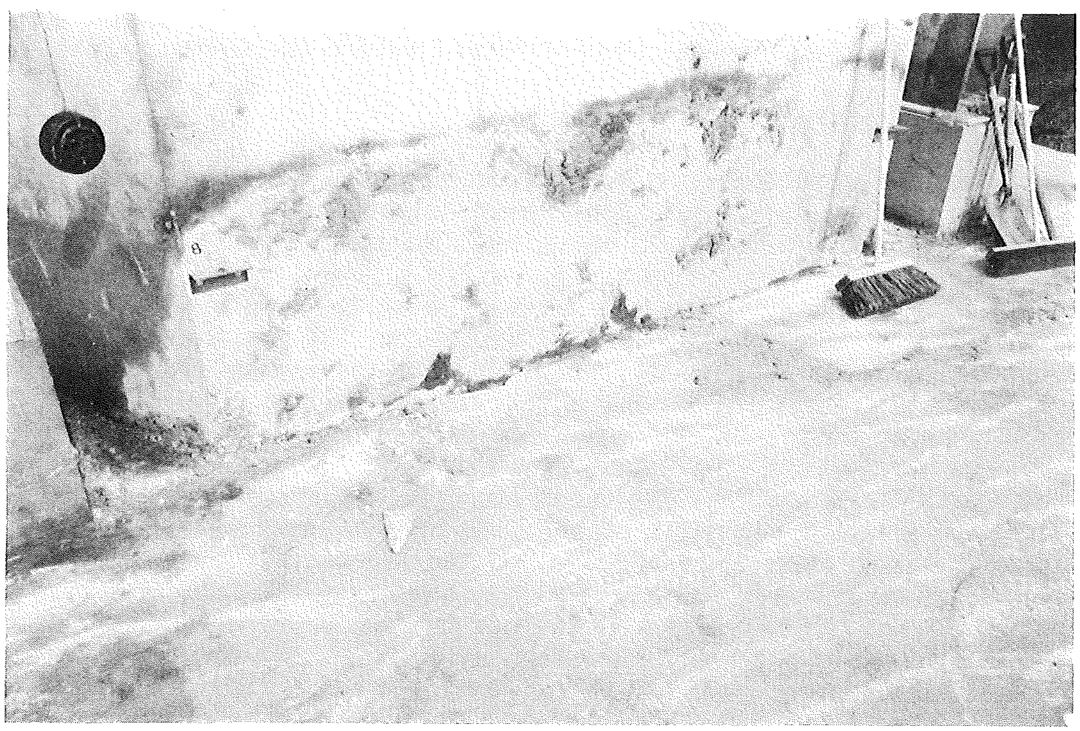


Figure 13. Fractures at column-footing intersection.

moved under the influence of temperature changes, but that there was no significant tendency toward additional major displacement.

This monitoring went beyond the initial objectives. Testing and Research participation continued during assessment of damages, design of repair procedures, freezing of the earth, and during repairs. Involvement ended when the deck had been rotated back to its original position. Because of this, the report will be divided into three separate phases.

Phase I will include monitoring for stability and the tracking of trends that occur with long-term seasonal temperature change through the winter of 1982-83. Phase II starts when the retrofit construction began in late May of 1983 and continued until the two outside portions of the new footing were completed. Phase III consists of the monitoring done during jacking and rotation. A summary of each type of measurement is included for each phase. Appendix B describes in detail the procedures used when monitoring.

PHASE I

This phase began on September 7, 1982, when Structural Research personnel were called to the Zilwaukee bridge site to determine the stability of the structure. Measurements, as described in Appendix B, were taken daily through September, and then approximately every week through the winter. There were no construction activities on the bridge during this time. The last measurements of Phase I were taken on May 5, 1983.

Footing Elevations

Elevation readings were used to determine the degree of movement at the footing. Reference points were established on the edges of the top of the footing (see page 53 of Appendix B for details of installation).

During the first two weeks readings were taken two or three times daily. From September 7 through September 13, the footing measurements appeared to be quite predictable with the average vertical movement of all four corners being 0.060 in., varying from 0.044 in. on the southeast corner to 0.067 in. on both northern corners.

For the remainder of the monitoring period through May 5, 1983 the footing remained relatively stable with an average movement of 0.043 in. The range was from 0.025 in. on the northeast corner to 0.064 in. on the southwest corner. The footing rocked very slightly with temperature changes in the bridge superstructure. It became evident that the northern portion of the footing was effectively disconnected from the rest of the footing, and that there was an extremely slight displacement of the columns downward, relative to the footing.

Column Elevations

Column elevations were taken in the same manner and for the same reason as the footing elevations. Reference points were attached to the columns so that elevation changes could be monitored (see page 55 of Appendix B).

Column elevations appeared to be fairly stable, and compatible with the footing elevation changes. Relative vertical movement of the column versus the footing could not be detected using elevation readings. Since it was believed that this type of movement did exist, pins were placed in the footing and the column base and the relative displacement measured as described in the following section.

Column-Footing Relative Movement

Reference points were placed and a special attachment made for the vernier calipers so that measurements could be made of any movement between the face of the column base and the top of the footing (see page 59 of Appendix B). Readings were taken approximately every other week from November 4, 1982 through May 5, 1983. During this monitoring period, there was a significant trend in the fact that the northern pair of pins of both columns showed more movement than the southerly sets. The north pins moved a total of 0.060 in. as opposed to 0.021 in. for the south. This would seem to indicate that the column was 'hinged' to the footing on the south side.

Longitudinal Column Movement

Reference points were set vertically at 10-ft intervals on the side of the column, so that displacement of those points could be read with the theodolite (see page 59 of Appendix B). Readings were taken at least daily--and usually more frequently--during the week of September 13, and approximately once a week for the remainder of Phase I. These measurements were among the most useful of all the readings taken on the structure. It was found that column movement varied directly with temperature of the superstructure. The top target movement ranged from -2.00 in. (toward the river) at 20 F to +0.22 in. (away from the river) at 87 F. Note that the column top was shifted approximately 8 in. towards the river when the instrumentation was installed. This meant the column moved toward the river during contraction of the superstructure, and away from the river as temperatures rose and the bridge expanded. The straight line relationship of temperature to target reading is shown in Figure 14.

Further, all of the remaining targets showed a movement with temperature in such fashion that the data points could be represented by a straight line, indicating that the column was not bending significantly as it moved. It also appeared that the effective point of rotation was located about 17 ft below the top of the damaged footing.

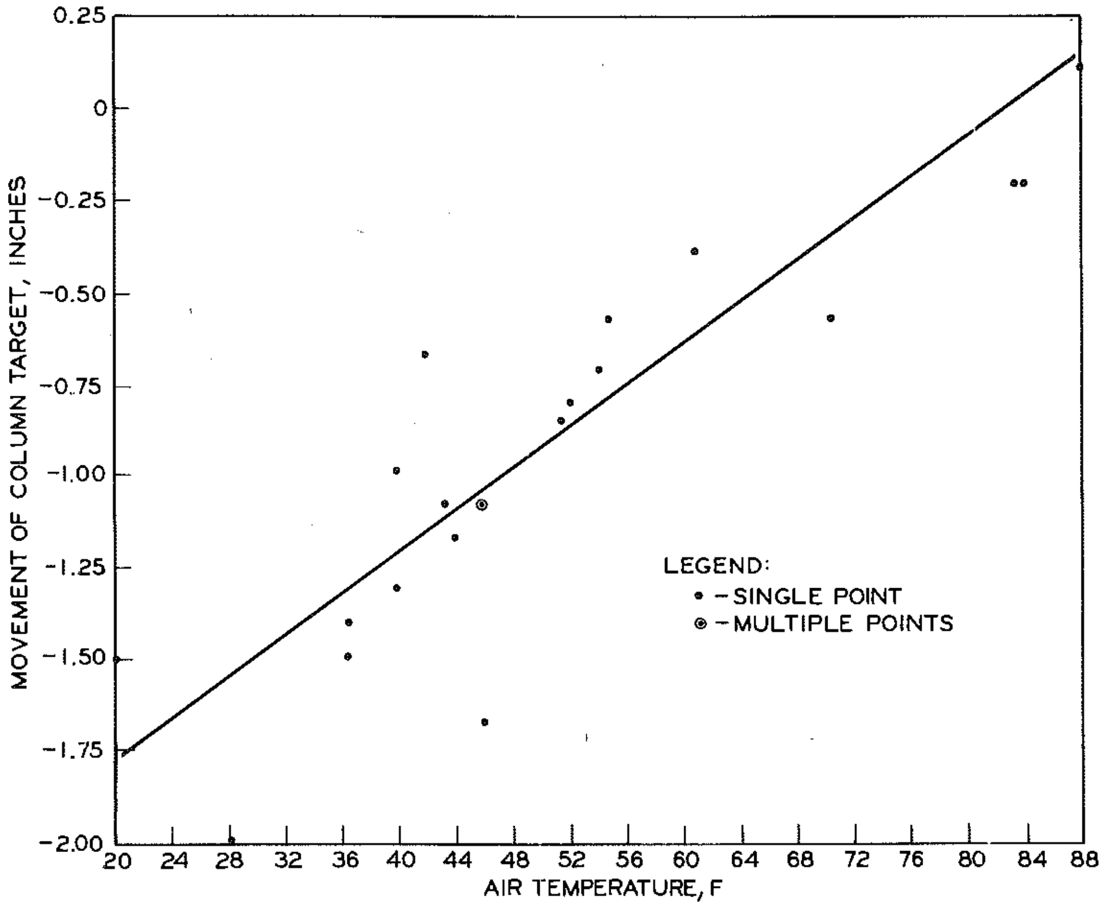


Figure 14. Linear relationship of column target readings to air temperature.

Movement of Cracks in Footing Sides

Reference points were set to measure crack widths in the vertical faces of the footing (see page 63 of Appendix B). The first readings were taken when the pins were set on October 29, 1982. An effort was made to read these cracks every week; however, the footing was covered with plastic during the winter making such readings difficult. Only eight sets of readings were taken during Phase 1, approximately two to three weeks apart.

Because of these limited data, it was hard to draw any conclusions other than the crack openings were relatively stable. The average of all the readings of all the cracks was 0.000 in. with an average range from -0.004 in. to +0.007 in. One crack did show an opening of 0.018 in. when measured on a cool forty degree day. Since it was the intent of these measurements to determine whether the crack openings were getting increasingly wider, and they did not, the information was useful.

Crack on Top of Footing

Reference pins were set to indicate both vertical and horizontal movement of the major fracture between the columns in the top of the footing (see page 63 of Appendix B). Measurements were taken about every other week during Phase I starting November 9, 1982. During this monitoring, the crack opening exhibited very little net movement with the west side of the crack closing an average of 0.008 in. and the east side closing an average of 0.010 in. Both sides showed an average increase in faulting of less than 1/16 in. Here again, the intent was to monitor for changes that would indicate instability of the fractured base. Net movements were found to be extremely small.

Cracks on Bottom of Segments

Reference pins were set inside the supersturcture, on the floors and walls, as discussed in Appendix B and shown on page 63. Readings were first taken on September 9, 1982 and twice a day the first two weeks. Then readings were usually taken every other week until January 5, 1983. A final set of readings was taken on March 17, 1983. Initial crack widths averaged 0.050 in. ranging from 0.020 in. to 0.150 in. Pin readings across the segment joints and cracks showed little movement with the maximum movement occurring on January 5, 1983, showing a closure of 0.042 in. from the original readings. In general, all the cracks and joints showed no signs of opening, and in some cases closed up. The maximum measured opening of a crack or joint was 0.008 in. Here again, small changes were noted with the motion of the structure, but no progressive opening of the fractures occurred.

Expansion Joint Movements

Pins also were set for measurements across the bottom of the damaged expansion joint (see page 66 of Appendix B). The pins at the expansion joint did not show any pattern of movement with temperature change, as the joint is tied together on top. The maximum joint opening was 0.109 in. on the east side of the expansion joint and 0.111 in. on the west side on September 10, 1982 when the temperature was 66 F. The maximum joint closure on the east side of the expansion joint was 0.052 in. when the temperature was 75 F and 0.055 in. on the west side when the temperature read 44 F. Motion here indicates rotation or twisting of the superstructure at the damaged expansion joint.

Readings also were made about every two weeks throughout Phase I on the first undamaged expansion joint north of the river. While the damaged and tied expansion joint showed very little movement, the functioning expansion joint moved with temperature changes, ranging from an opening of 3-1/4 in. on the east side and 3-1/8 in. on the west side at 20 F to a closure of 1/4 in. on the east side and 9/32 in. on the west side at 85 F.

Superstructure Deck Profile

Complete deck profiles (described on page 67 of Appendix B) were taken on October 5 and 19. On November 4, 1982, a partial deck profile was obtained. These profiles were taken on days when the crew was making measurements on the launching girder, which was almost daily in November and early December, 1982. This was done to check on the stability of the structure before going up to take measurements on the girder. After that, measurements were taken once a month or so. Because the temperature did not vary much during that time, little movement took place. The measurements confirmed relative stability of the structure, allowing the crew to work on the launching girder, and were also useful in putting together a picture of how the total structure was moving.

Vibrations

During Phase I, vibrations were monitored four times to determine their level during a period of no construction activity, and to compare the amount of vibration at the end of the cantilever with those on the footing. Generally speaking, truck traffic on I 75 which paralled the new bridge caused a vibration of around 0.01 in. per second at both locations. This level is too low to be perceptible without instruments, and indicated no significant amplification of the foundation vibration by the superstructure.

Phase I Summary

During this phase, it was determined that the entire structure was fairly stable. The elevation of the footing and the columns remained relatively constant and none of the footing cracks were increasing in size. The segment joints and cracks in the superstructure showed very little movement. Vibrations were insignificant.

During temperature changes, movement was noticeable in the column and to some extent in the deck, as the superstructure expanded and contracted. The targets showed the column moved south during the heat of the day and to the north as the temperature cooled. The pins set to measure relative footing-to-column movement proved that most of this movement took place on the north side of the columns. The undamaged expansion joint north of the river moved as it was designed to, closing as temperature increased and opening as temperature decreased, while the damaged and tied expansion joint showed very little movement. Generally the structure movement was predictable and no significant overall tendencies toward further distortion of the position of the structure occurred. Information was relayed to the Construction and Design Divisions, and the consultants, as the work progressed.

PHASE II

This phase involved monitoring of movements during construction repair. Repairs consisted of installing freeze-pipes, freezing the ground to ensure stability, drilling and placing concrete filled caissons to bedrock, building a new footing on top of the caissons, fabricating and installing falsework and jacks, and building and installing the tie-down system under the expansion joint.

Monitoring began on May 31, 1983 when the installation of the freezing pipes began and continued on a daily basis until July 7. During this time, the ground freezing began. In July and August monitoring was done on a weekly basis, as the concrete caissons and the two outside portions of the new footing were constructed (Fig. 15). During this time the tie-down block was poured and the cables at the expansion joint snugged. In September, monitoring was done during the four days that the original construction pier frame was being removed.

Figure 16 shows the new footing which rests on the concrete caissons, Figure 17 shows the tie-down details, and Figure 18 shows the falsework that was erected after the pier frame was removed.

Monitoring was done in much the same fashion as during Phase I with one addition. During Phase II, the contractor purchased and installed a tilt-sensing and warning system recommended by the Department. The Research Laboratory had purchased the same basic system, and was, therefore, familiar with it. Consequently, the Department worked with the contractor in using the system and analyzing the output.

Footing Elevations

During Phase II, construction of the new footing began. Only the two outside portions were to be completed during this phase, allowing the monitoring of the elevations on the old footing to continue. This was essential since the stability of the entire structure depended on the state of the footing. Footing elevations were taken at least daily and usually on an hourly basis from May 31 through June 6, 1983. In general, the footing remained stable during which time the maximum movement for any corner of the footing was 0.02 in. From June 6 until July 7 when measurements were taken at least daily and sometimes at more frequent intervals, the four corners of the footing settled an average of 0.12 in., ranging from a 0.22 in. drop on the southwest corner to 0.05 in. on the northeast corner. Although hourly readings may vary, the trend was a rather steadily downward displacement during the month-long monitoring period.

After July 7, monitoring began on a weekly basis. The northeast corner was covered and could not be monitored; however, the three remaining points could still be used to determine footing stability. The average movement of the remaining three corners was 0.02 in.

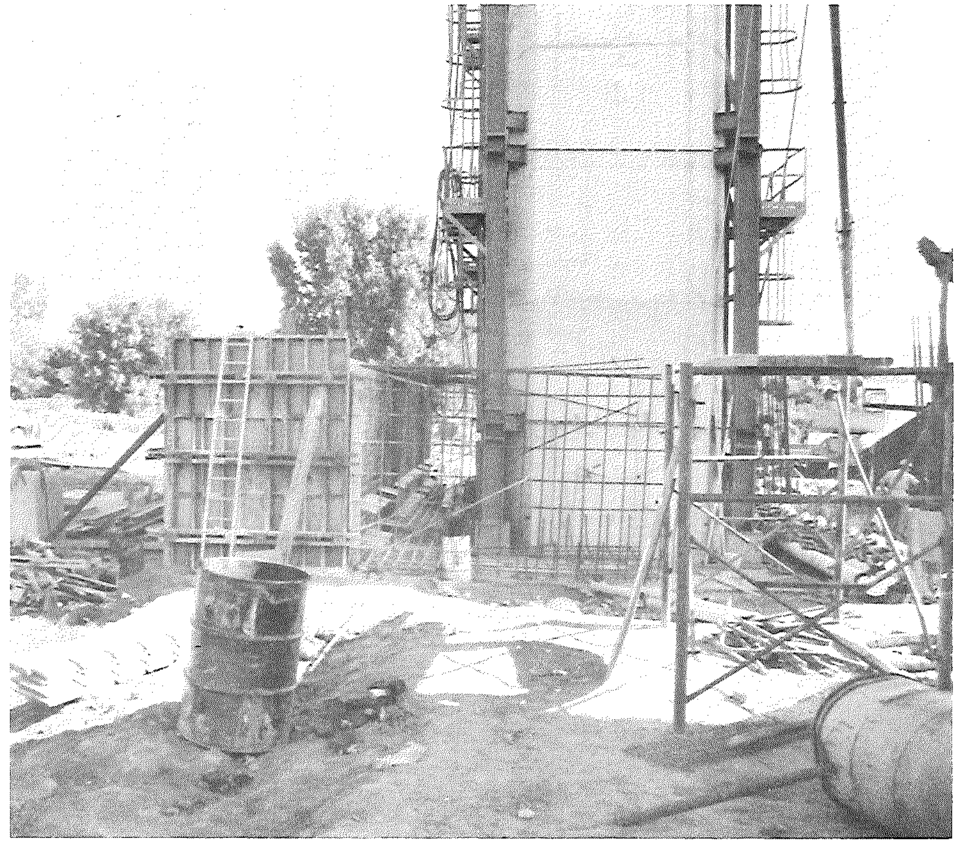


Figure 15. Footing site showing concrete caisson drilling (left). North portion of the new footing is shown after completion (above) with south portion ready for concrete placement.

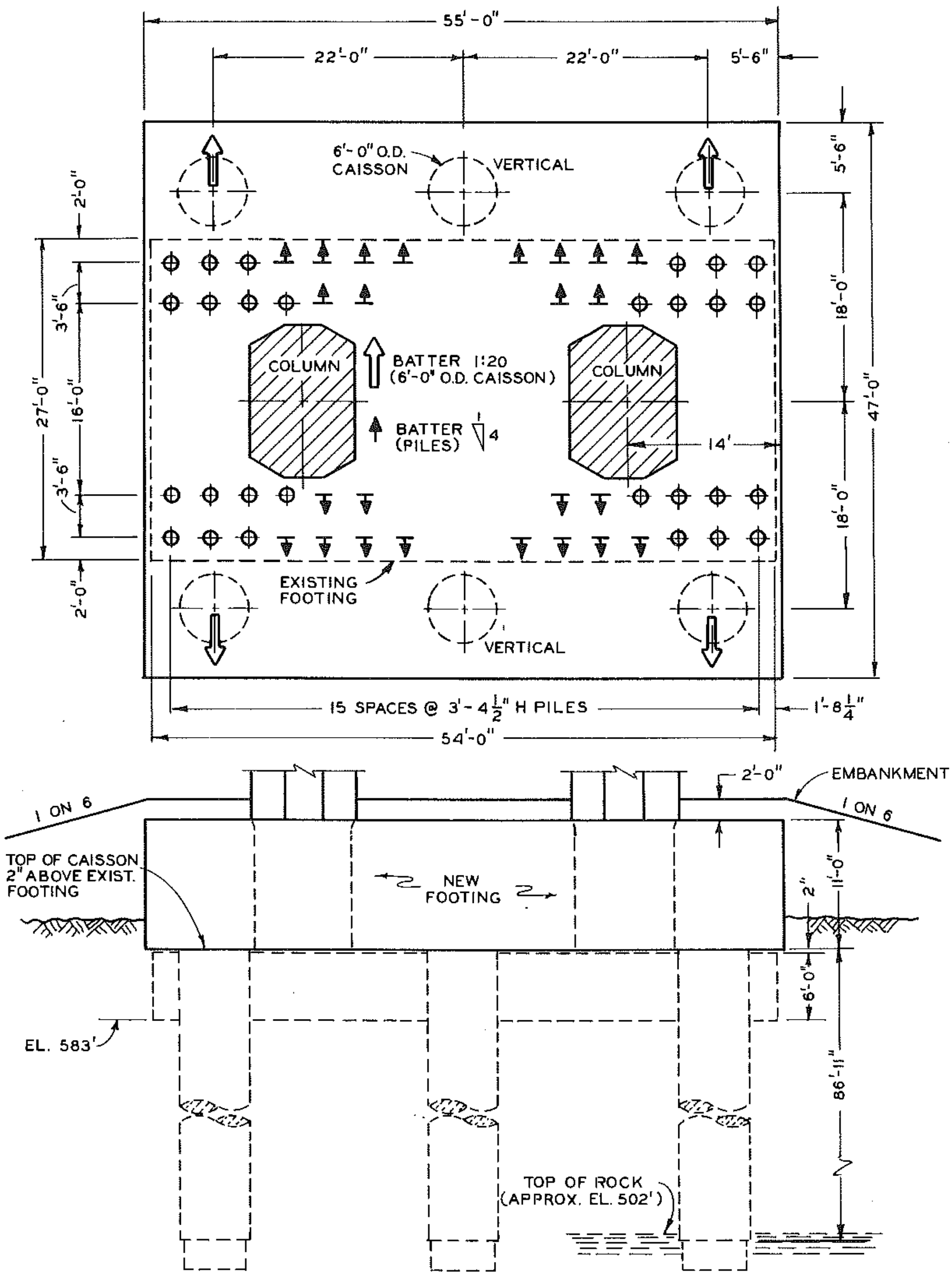


Figure 16. Vertical and horizontal schematic views of the new footing.

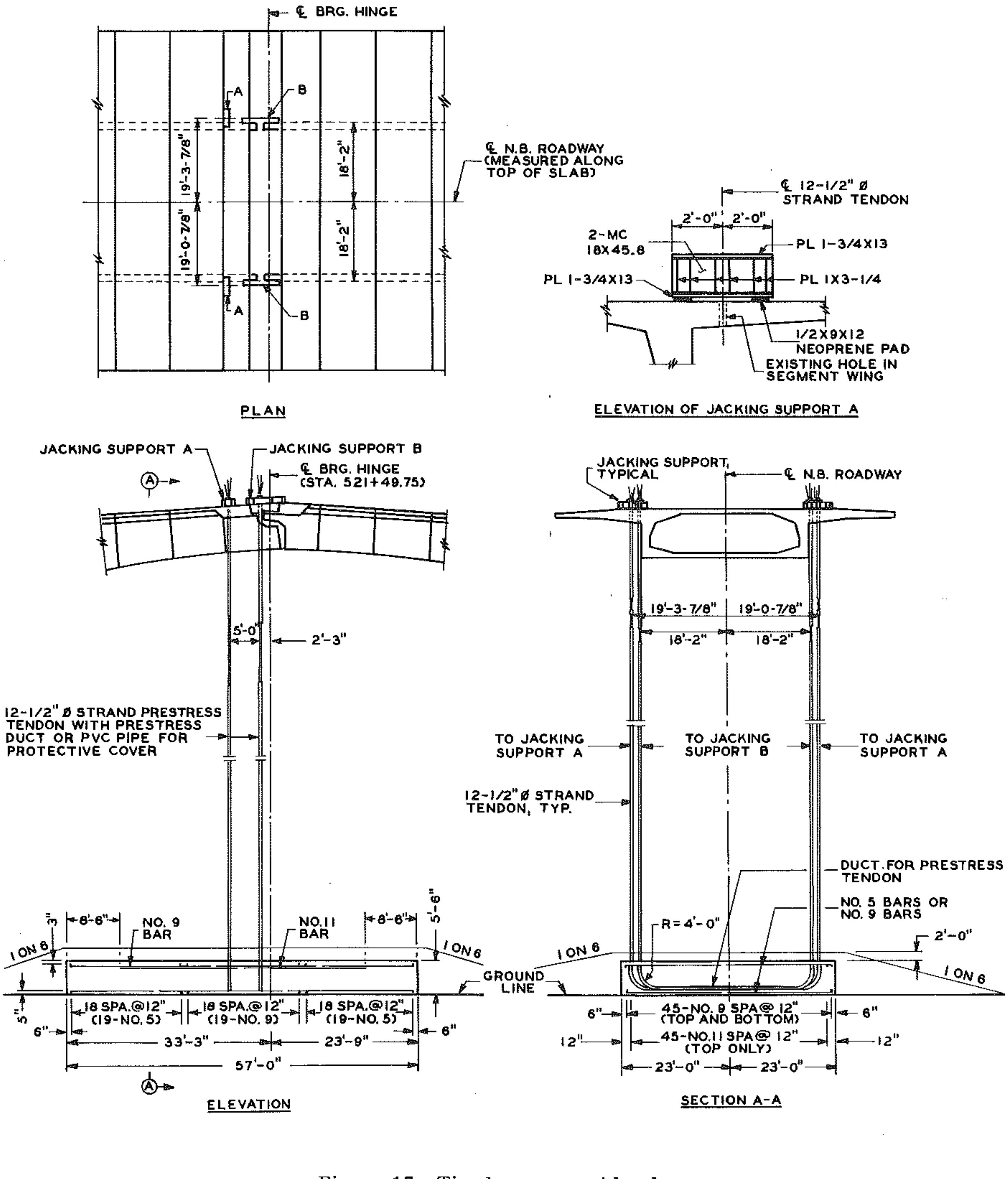


Figure 17. Tie-down assembly plans.

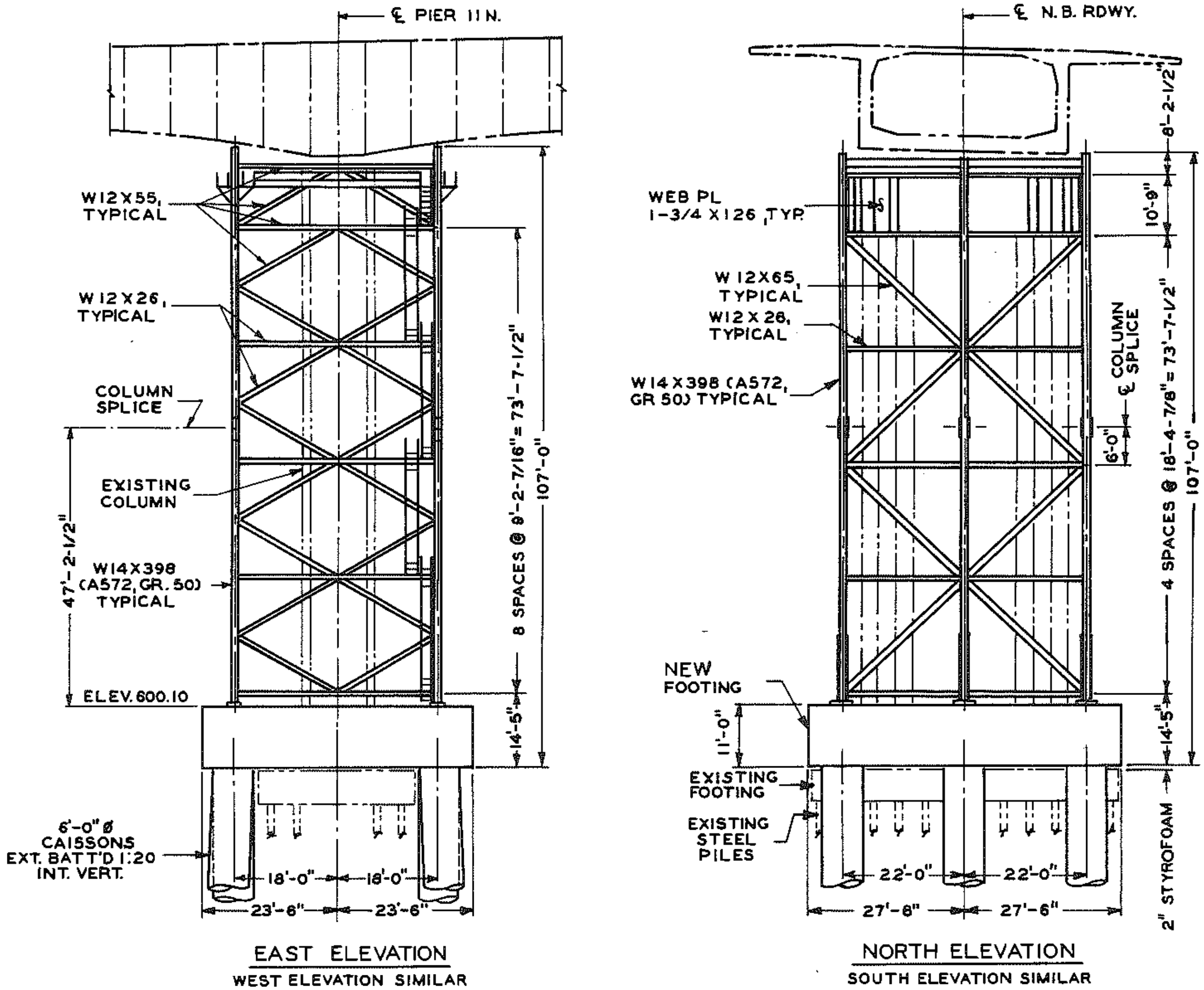


Figure 18. Falsework assembly plans.

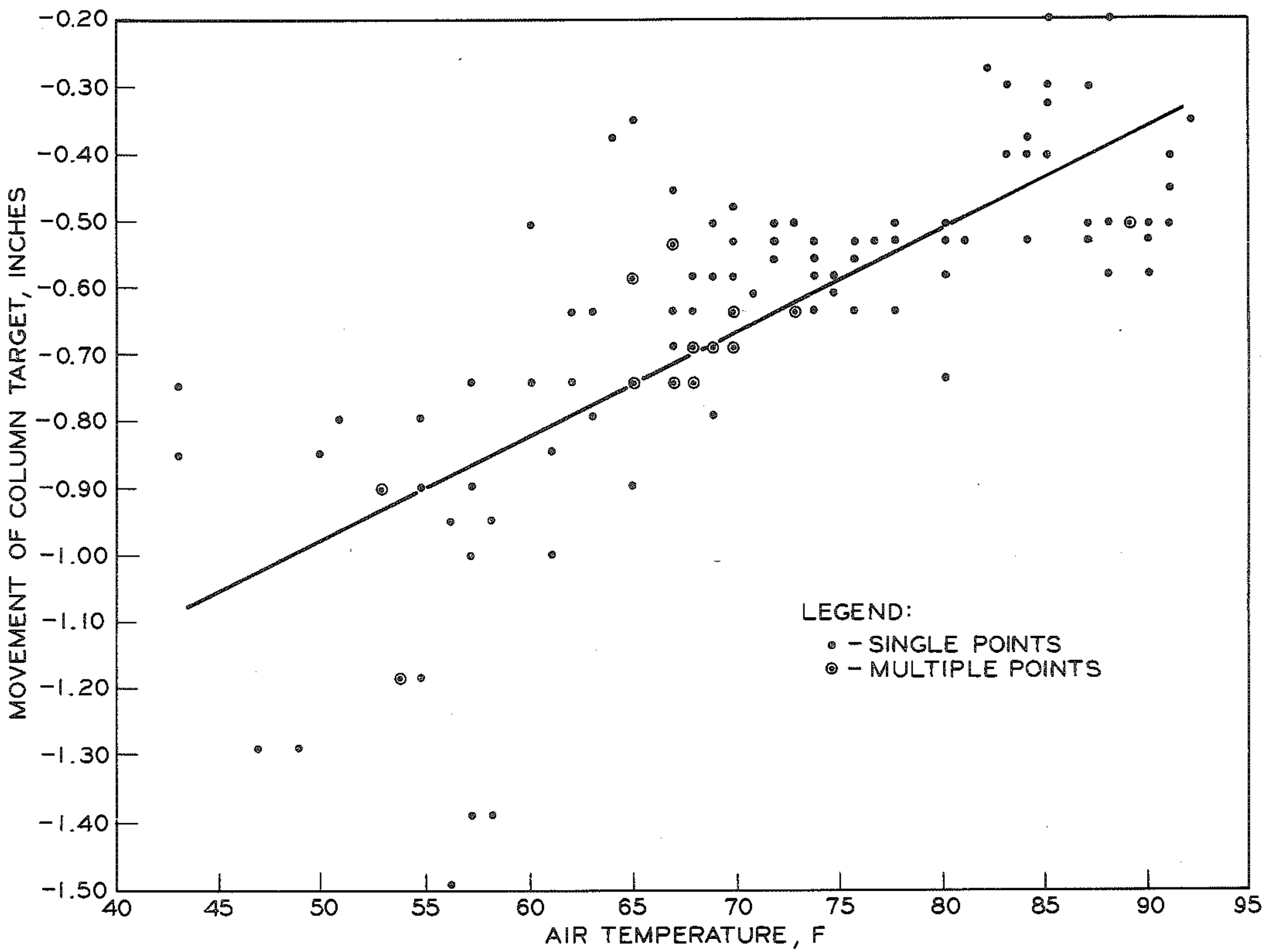


Figure 19. Linear relationship of target at top of column versus air temperature, Phase II.

Column-Footing Relative Movement

These measurements were taken at least daily from June 6 until July 7, 1983, and then on a weekly basis until September. The same trend that was established in Phase I continued. During the day as the temperature rose, the northern pin readings usually increased while the southerly pins usually remained stable. The northern pins increased an average of 0.008 in. while the southerly pins increased an average of 0.001 in. Although these numbers are relatively small, the trend does show that the column moved toward the south during the expansion which took place in the afternoon and appeared to be 'hinged' to the footing on the south side of the column.

Longitudinal Column Movement

Readings were taken approximately every daytime hour during the first two weeks of repair construction. From that point until July, measurements were taken two or three times a day. Readings after July 7 were taken on a weekly basis until early September, 1983.

The same trends established in Phase I continued in Phase II. The column moved away from the river as the temperature increased and toward the river as it decreased. The column targets also continued to move in a fairly straight line, indicating very little additional bending was taking place in the column as it moved with temperature.

The only thing that differed from the Phase I trend was that during the winter the top of the column never moved back past the initial zero reference point established by the Research Laboratory after the accident. The top target ranged from -1.5 in. at 56 F in May, 1983 to -0.1 in. at 90 F in mid-July, indicating a very slight net motion of the structure toward the river.

Figure 19 shows the linear relationship between column movement and temperature for Phase II. Phase I and II data are combined in Figure 20. Both figures show a strong linear relationship, indicating no significant change in the support condition during the monitoring period.

Crack on Top of Footing

Readings began on June 6, 1983 and were taken approximately every hour during the following two weeks. After that, readings were taken at least daily until July 7, and then weekly until September. The footing crack remained stable and of constant width throughout Phase II. This shows that the fractured pier cap pieces did not change relative positions during the monitoring period, which covered the time of major construction repair activities.

Expansion Joint Movements

Although these readings were taken only five times, the trend established in Phase I continued in Phase II. The damaged expansion joint closed

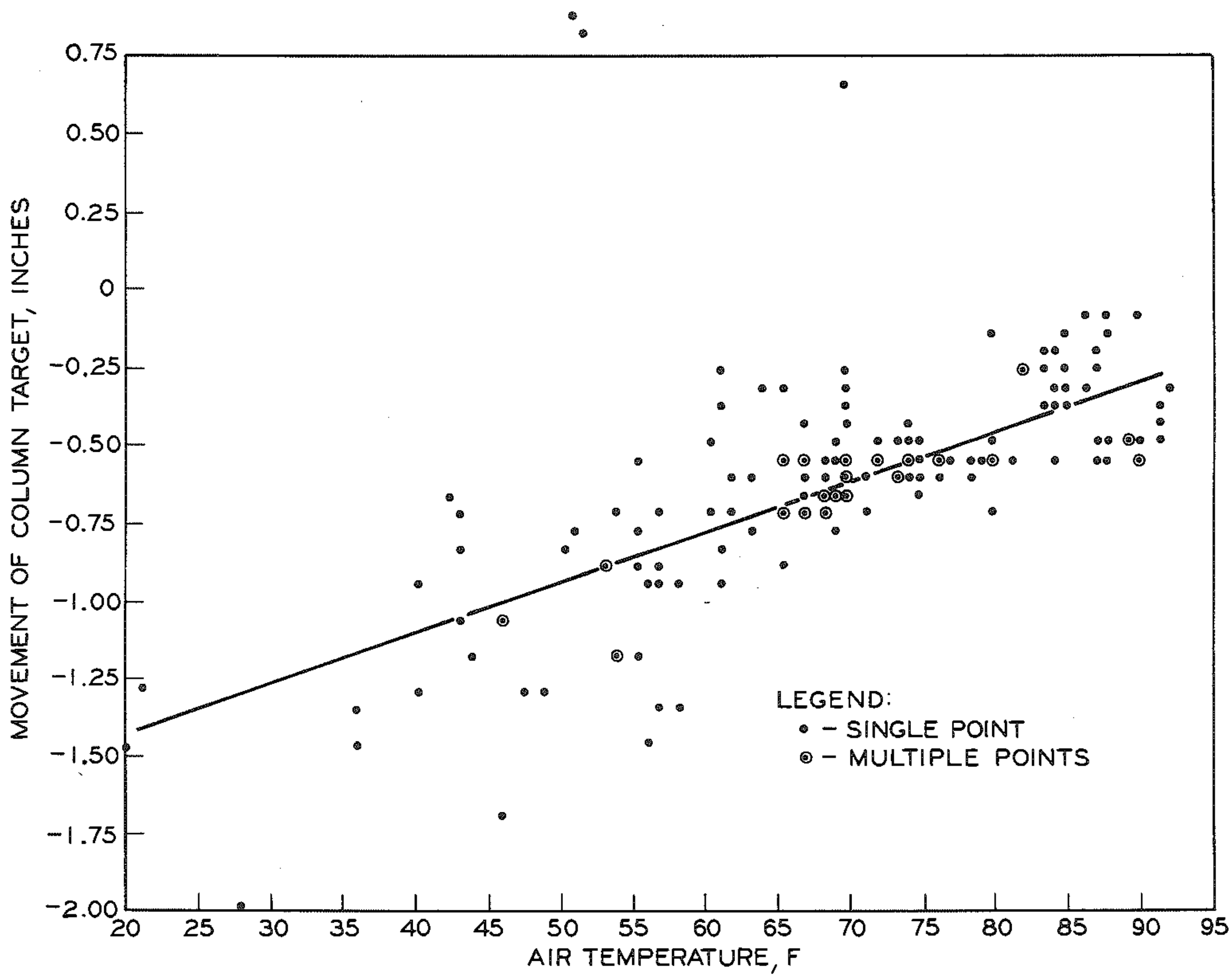


Figure 20. Linear relationship of target at top of column versus air temperature, Phases I and II.

1/8 in. at the bottom on both sides from June until late November, 1983 and the functional expansion joint north of the river opened 4 in. on both sides for the same time period. Closure at the damaged joint indicates some additional crushing and consolidation of the temporary blocks had occurred due to the daily flexure of the joint caused by temperature-induced expansion and contraction of the superstructure.

Superstructure Deck Profile

With the exception of the complete profiles taken on August 23, 1983, partial profiles were taken during Phase II. Specifically, elevations were monitored at a point over pier 12N, a point just south of the damaged expansion joint, and a point at the south end of the cantilever.

Measurements taken during Phase II showed a tendency for the south end of the cantilever to drop as temperature increased during the day. The point just south of the damaged expansion joint showed some movement, but did not vary directly with temperature and for the most part remained stable. When the data from Phases I and II were combined, the long-term trend was for the superstructure to act as a rigid body. As temperature increased during the day, the south end of the cantilever decreased in elevation and the expansion joint end increased in elevation. The opposite trend held true as temperature decreased. This long-term trend is shown by the linear regression analysis in Figures 21 and 22.

The last deck profile taken before the tie-down cables were snugged at the expansion joint was at 12:55 p.m. on August 23, 1983. On this date at around 2:00 p.m., the cables were tightened. The next profile, taken on August 30 at about the same temperature as that taken on August 23, showed a 1/4-in. drop in the deck at the expansion joint and a 1/2-in. rise at the end of the cantilever. No further profile readings were taken after installation and initial application of tension to the cables, since the cables effectively prevent gross rotation of the superstructure about the pier.

Vibrations

Vibrations were monitored continuously during the drilling for the freezing pipes until the start of the concrete caissons drilling operation. This began on June 7, 1983 and continued through July 7. One seismometer was set on the footing and the other on the end of the cantilever to provide continuous monitoring of the structure to check for unusual or excessive motion. Average maximum levels during construction were usually around the range of 0.005 in. per second to 0.010 in. per second. If a bulldozer was operating near the footing, the range could reach 0.02 in. per second. Both seismometers usually showed about the same output, indicating no unusual or excessive motion.

In mid-September the pier frame was removed so the falsework could be erected. Two transducers were placed on the end segment of the can-

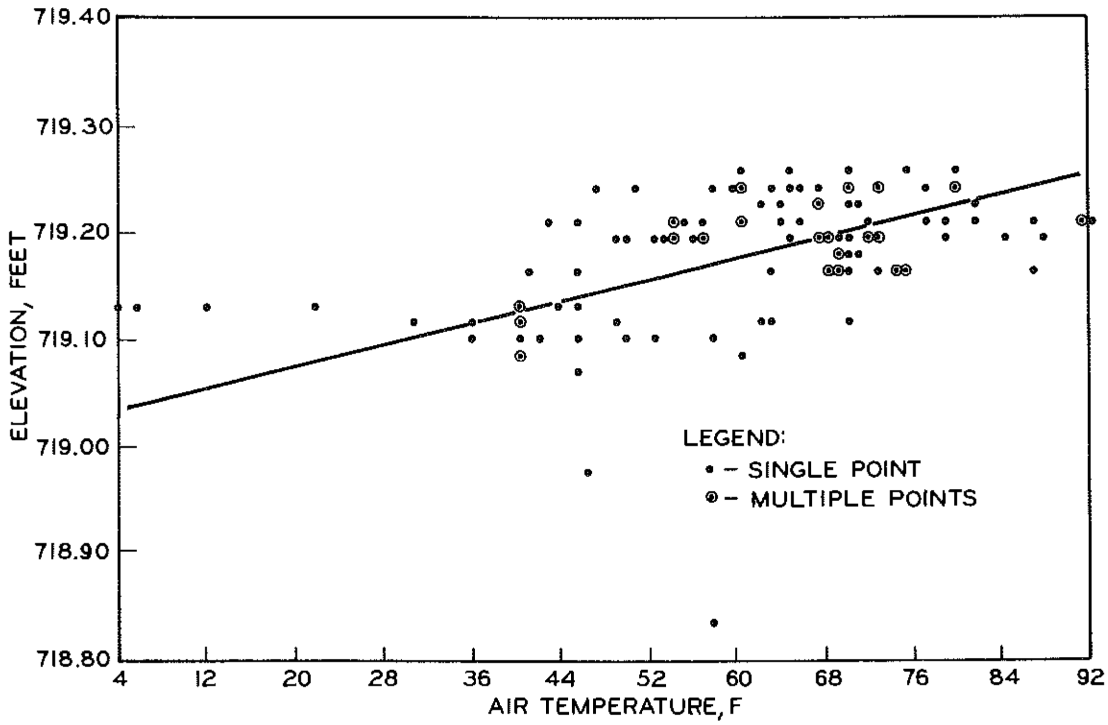


Figure 21. Linear relationship of the point just south of the expansion joint (No. 28) to temperature, Phases I and II.

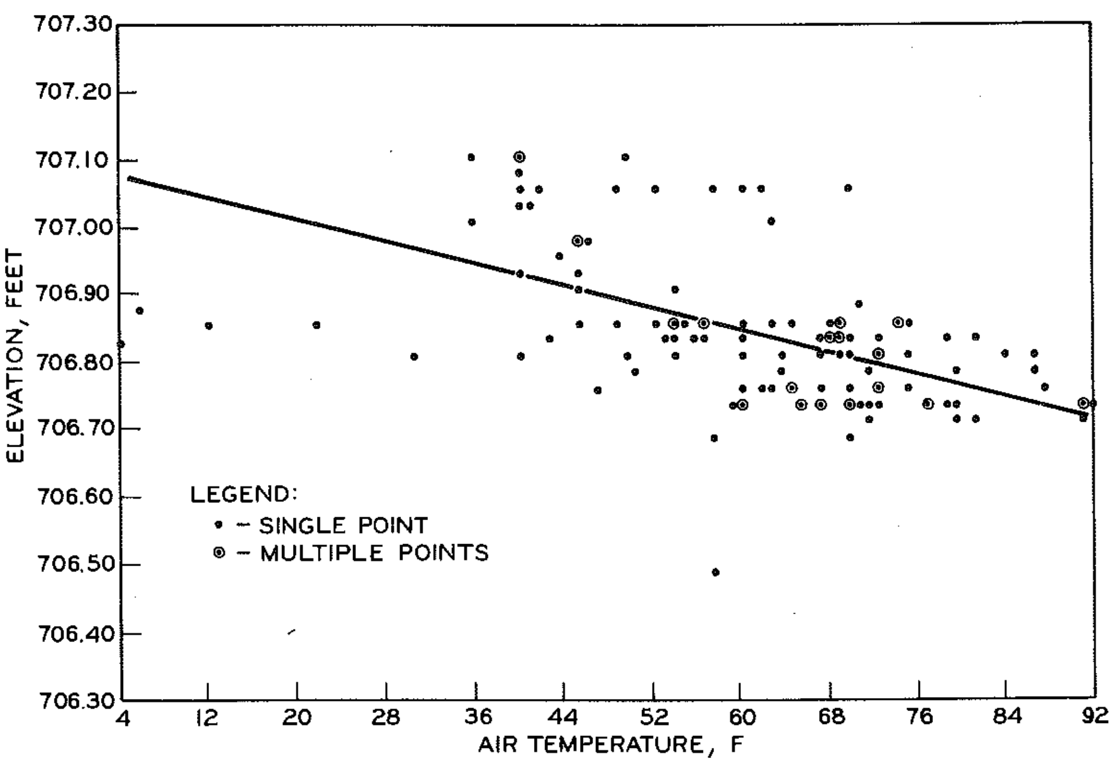


Figure 22. Linear relationship of the point at the end of the cantilever (No. 40) elevation to temperature, Phases I and II.

tilever deck; one in the middle of the segment and the other over the web (Fig. 23). During the removal and 'walking' down of the pier frame, vibrations reached maximum levels of 0.11 and 0.13 in. per second. This level is well into the perceptible range but still not excessive or indicative of any problem.

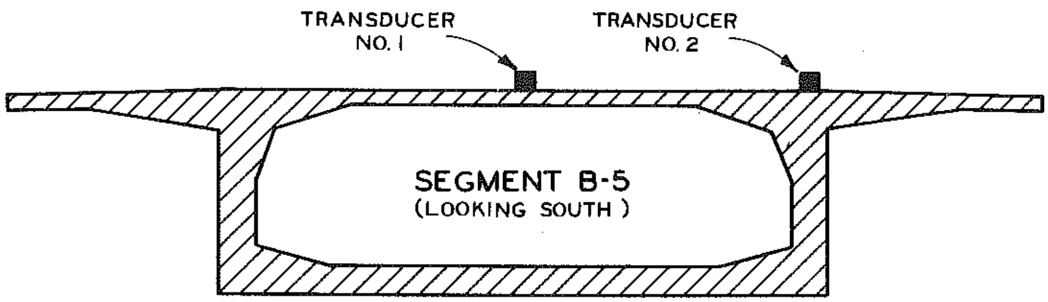


Figure 23. Location of transducers during pier frame removal.

On September 20, the day after the pier frame was removed, the vibrations in the deck were constant throughout the day at 0.01 to 0.02 in. per second. This was also detected by the tilt sensors and discussed in the next section. It was determined that the strong, constant wind blowing that day, along with the removal of the pier frame, caused this to occur. The next morning the wind was calm and there were no vibrations detected. When the wind picked up that afternoon, the same vibrations as those of the previous day were noted.

Tilt Sensors

Extensometers were installed on the structure by the contractor at various locations at the start of repair work. These were attached to an alarm system which would warn workers to leave the area if any unusual movements took place. Primary sensors were installed on the footing's corners, referenced to anchors augered into the earth below. However, digging, filling, and insertion of freeze pipes in the area caused the soil to shift, giving faulty readings on the sensors. Since there was no suitable anchor point available for steady reference, the contractor was directed to put up a tilt sensing system that used the earth's gravity as a reference and, therefore, was independent of minor shifts in the surrounding soil. A Sperry tilt sensing system was obtained. Essentially the sensors consist of level bubbles with electrical wiring and electronic circuitry to detect rotation. Four sensors were installed: two were mounted on the west column about 30 ft above the damaged footing (one for measuring longitudinal movement and the other for measuring transverse movement), one on the footing, and one inside the superstructure on the floor of the segment just over the west column of pier 11N (Fig. 24). The tilt measuring system was connected with the major data logging system so that positions were monitored continuously and data recorded at regular intervals. Thus, any motion of the structure would be indicated immediately. Outputs from the tilt sensors were fed to the alarm system.

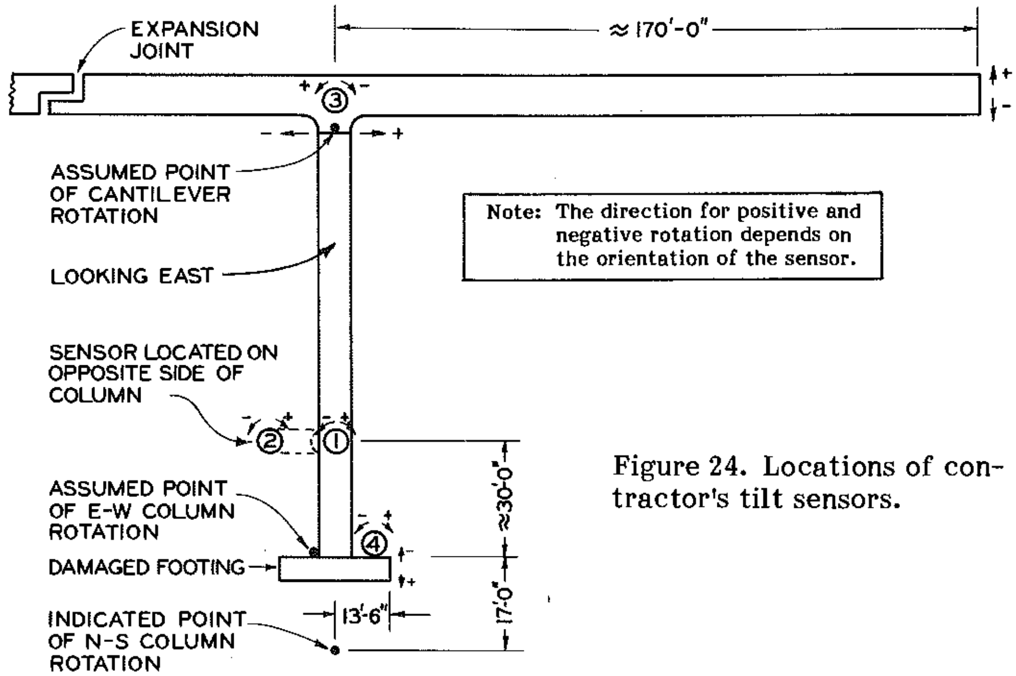


Figure 24. Locations of contractor's tilt sensors.

At first there were problems with false alarms. After trying several different options such as interchanging tilt sensors and replacing the contractor's sensors with MDOT sensors, it was determined on August 31, 1983, that electromagnetic interference caused by a portable two-way radio entered the system through the unshielded cable running from the Sperry console to the Acurex Data Logger. This cable was replaced with a shielded one and there were no other false alarms, other than once when the footing sensor was bumped by a worker.

The monitoring and analysis of the tilt sensor data were primarily the responsibility of the contractor. The data were, however, periodically reviewed by Department personnel. On September 9, 1983 the contractor no longer had a person monitoring these data on a full-time basis. At this point, the Structural Research Unit took over the responsibilities of plotting and analyzing these data. This was done on a weekly basis.

The sensors for measuring motion of the footing and transverse movement of the column remained fairly stable. The sensor for measuring longitudinal movement showed that the column would move away from the river as temperature increased and toward the river as temperature decreased. The tilt sensor measurements of longitudinal column movement correlated quite closely with those taken with the theodolite on the targets mounted on the east column. This showed that the tilt sensor measuring column and footing movement were reliable and that the east and west columns appeared to be moving about the same. The displacement trends of the footing and two column sensors continued throughout the monitoring period.

When the tilt sensor inside the deck was first monitored, the daily trends which were established appeared to show some movement not detected by previous superstructure profile measurements. As the temperature increased, the tilt sensor reading also increased. This meant that if the superstructure were rotating as a rigid body, the south end of the cantilever should rise and the expansion joint end should drop in elevation. The deck profile was indicating an opposite movement for the south end of the cantilever and virtually no movement at the expansion joint end of the cantilever.

In order to verify or explain this reaction, the Structural Research Unit on August 10, 1983, put two of its tilt sensors inside the superstructure. One was placed just east of the contractor's sensor over the west column of pier 11N and the other was placed immediately south of the damaged expansion joint. Both tilt sensors over the column showed an increase of 0.06 arc-minutes over a 7 F temperature increase during a rainy, overcast day. This seemed to confirm that the contractor's tilt sensor was operating correctly. The tilt sensor near the expansion joint showed an increase of 0.34 arc-minutes over the same time period. This indicated the superstructure was bending but did not confirm that the superstructure's south end was decreasing in elevation as temperature increased.

On August 23, 1983, tilt sensors were placed inside the superstructure on the floor of the segments. One was placed in front of the contractor's sensor over the pier, one immediately south of the expansion joint, and one at the end of the cantilever. Another was placed over the column at a right angle to the other sensor to see if any transverse movement was occurring. The day was clear and sunny. From 9:00 a.m. to 1:00 p.m., the temperature rose 14 F. The contractor's tilt sensor showed an increase of 0.12 arc-minutes while MDOT's sensor directly in front of it showed an increase of 0.11 arc-minutes. The tilt sensor near the expansion joint showed an increase of 1.04 arc-minutes while the south end of the cantilever showed a decrease of 1.43 arc-minutes. The tilt sensor measuring transverse movement remained fairly stable. This confirmed that the deck was bending as the temperature increased during the day, as would be expected with the top surface of the deck subjected to direct rays of the sun. It also confirmed the profile measurements which showed the south end of the superstructure dropping in elevation as temperature increased.

Between 1:15 p.m. and 2:00 p.m., the tendons from the expansion joint to the tie-down block were being tightened. At this time, the tilt sensor

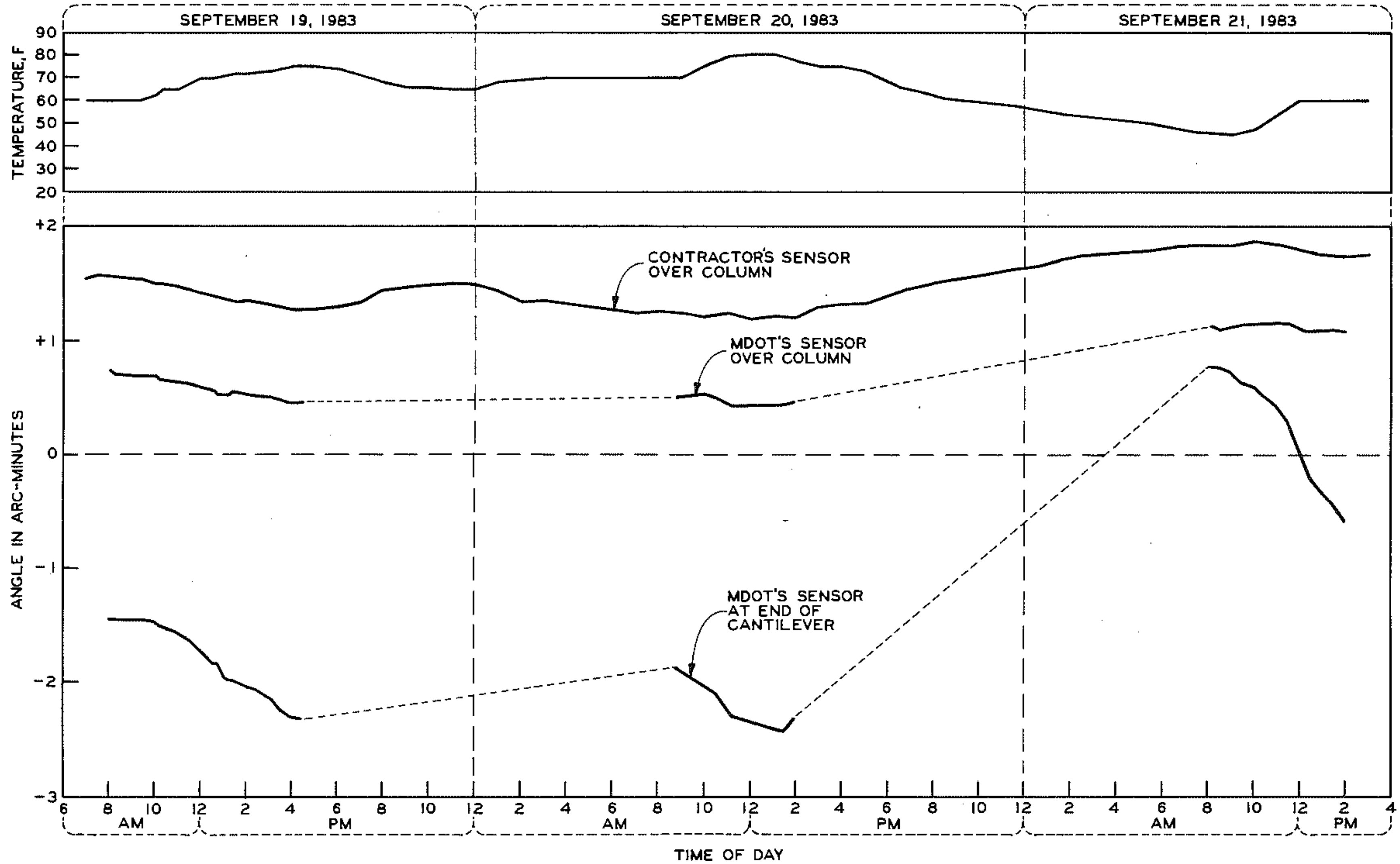


Figure 25. Tilt sensor movement during a sharp temperature drop.

at the expansion joint showed an increase of 1.00 arc-minutes and the tilt sensor at the south end showed a decrease of 0.30 arc-minutes. Once again, the two tilt sensors over the column were very close, with MDOT's tilt sensor increasing 0.51 arc-minutes and the contractor's increasing 0.48 arc-minutes.

From this time, the daily movement of the superstructure tilt sensor located over the column reversed itself. Thus, as the temperature increased during the day, the tilt sensor showed a negative movement. This meant if the deck were rotating as a rigid body, the expansion joint end would rise and the south end of the cantilever would drop in elevation, as the temperature increased. This indicated the motion of the system was now under the influence of the expanding and contracting tie-down tendons as well as the expansion, contraction, and bending of the concrete superstructure. This trend continued through the remainder of the project.

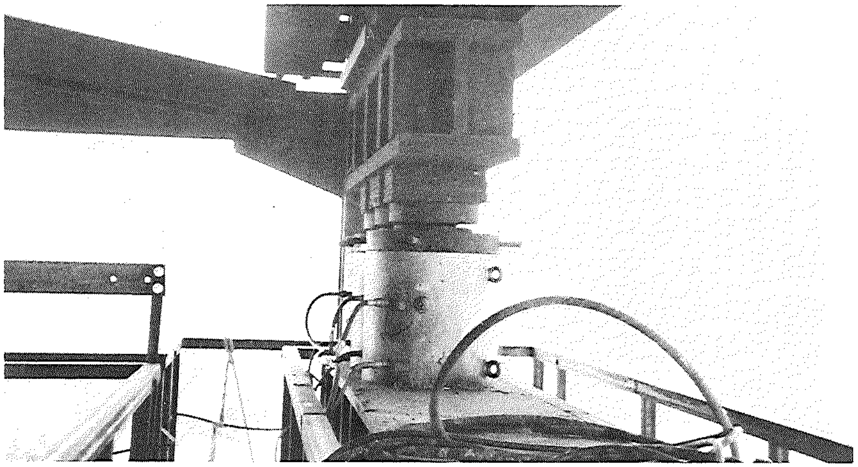
During the period that the pier frame was being removed, two tilt sensors were monitored; one over the column in front of the contractor's and one at the end of the cantilever. On September 20, 1983, the same day that there was a constant vibration assumed to be caused by strong winds, the tilt sensor readings were constantly varying, usually at an average of 0.06 arc-minutes and sometimes by as much as 0.20 arc-minutes. On September 21, the tilt sensor output remained steady with the return of calm conditions.

On September 20, the most significant overnight drop in temperature occurred since monitoring began, and the tilt sensors reflected this change. From noon on September 20 to 9:00 the next morning, the temperature dropped 35 degrees. The tilt sensor at the south end of the cantilever showed a positive angle change of 3.16 arc-minutes, indicating a rise in the south end of the cantilever. The sensor over the column showed a positive angle increase of 0.65 arc-minutes. This sharp contrast is shown in Figure 25.

Phase II Summary

Monitoring during Phase II showed that the structure remained stable during the ground freezing, and the caisson and footing construction. No unusual or unexpected movements took place. Any longer term movements which did occur were temperature related. Most of the same trends established in Phase I continued.

The mechanical means used to measure movement verified those being obtained by the tilt sensors. These sensors proved to be a reliable system for monitoring movements on a continuous basis and provided a reliable warning system of any unusual movements which may have taken place. Since they provide continuous output, they are much more valuable than the mechanical or optical methods used, because they immediately respond to motion that occurs.



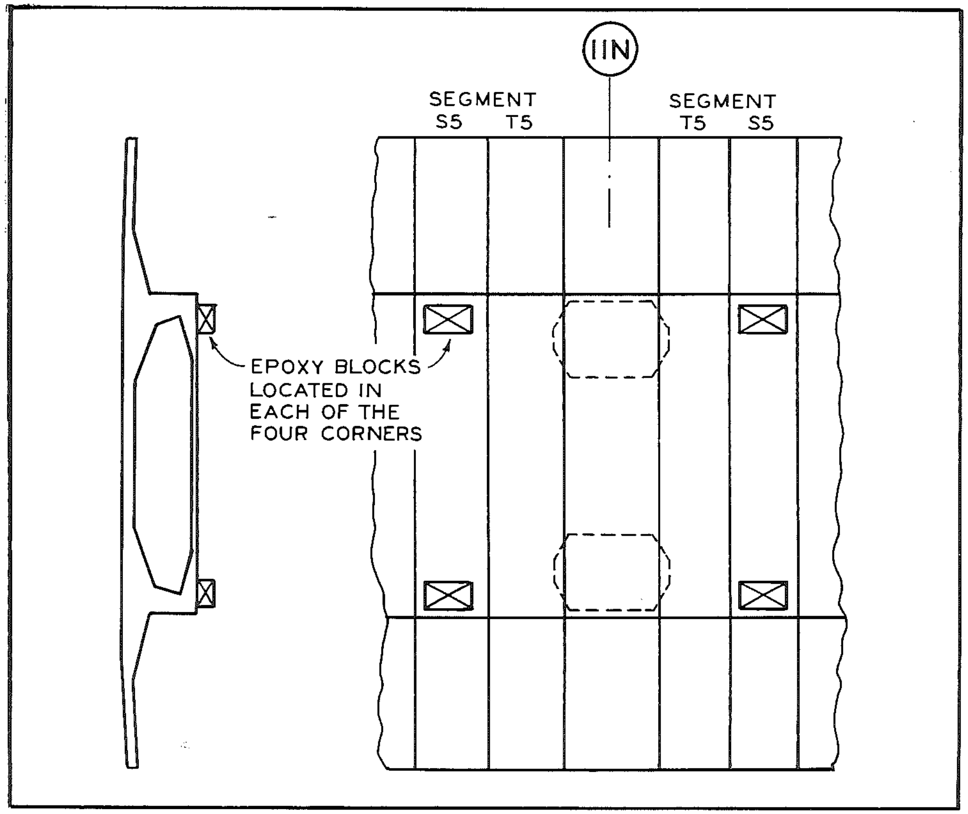


Figure 26. Jacking system and falsework used in the deck lifting operation.

PHASE III

Phase III involved the lifting of the superstructure off the column by means of four sets of three 600-ton capacity jacks placed on top of the newly erected falsework (Fig. 26). After this operation was completed, the middle portion of the new footing was finished and the bearings were replaced. The superstructure was then lowered back onto the new bearings and the deck rotated back into proper alignment. Monitoring was done as needed and usually only during the time in which the described activities were taking place.

Lifting Bridge Off Columns

Initial Lift - The initial lift was done on December 17, 1983 and was intended to relieve just enough pressure from the twin columns so the bush hammering and drilling could be completed without disturbing the superstructure. Bush hammering was done on the surface of the portion of the columns to be embedded in the new footing to ensure bond between the new footing and the columns. Post tensioning tendons were installed in 104 holes drilled through the columns to tie them to the new footing. The primary concern was to determine the effect on the structure from the jacking, drilling, and bush hammering, and to confirm stability. Tilt sensors were monitored during lifting and elevations of the old and new footings were taken before and after lifting.

Elevation readings were taken on the four corners of the new and old footings prior to jacking and after jacking. Very little if any movement was detected in either the new or old footing during this first lift.

During the initial lift, the contractor's tilt sensors were monitored by Structural Research personnel. Three sensors remained in their original position on the columns and superstructure; only the footing sensor was removed. The primary purpose of monitoring the sensors was to make sure the deck was jacked up reasonably straight. The superstructure sensor was monitored constantly and jacking was stopped if the sensor showed a deviation of more than 1/2 arc-minute from the start of jacking. During the first lift the column tilt sensors showed very little movement.

Two seismometers were placed; one on the footing and the other inside the last segment on the south end of the cantilever. During the jack pressurizing, the peak vibrations reached 0.007 in. per second on the footing and 0.030 in. per second in the superstructure. During actual lifting, the average peak readings were zero on the footing and 0.012 in. per second in the deck. At one point during lifting, the falsework steel shifted slightly giving readings of 0.009 in. per second on the footing and 0.030 in. per second in the deck. All the vibration readings recorded during this time are barely perceptible, and indicated a relatively smooth operation.

During the bush hammering and column drilling, average vibrations were 0.005 in. per second on the footing and 0.006 in. per second in the bridge deck which are totally insignificant.

Intermediate Lifts - Two more lifts were done prior to the middle portion of the footing being placed; one on December 19 and the other on December 21, 1983. The same measurements were taken as with the first lift, and elevation readings of the four corners of each column were recorded.

All elevation readings were taken prior to the December 19 lift and after the December 21 lift. The readings indicate no detectable movement of the old footing, the new footing, or the columns. The targets mounted on the east column were also monitored during lifting to verify tilt sensor readings.

The contractor's tilt sensors were again monitored in the same manner as during the initial lift; nothing unusual was detected.

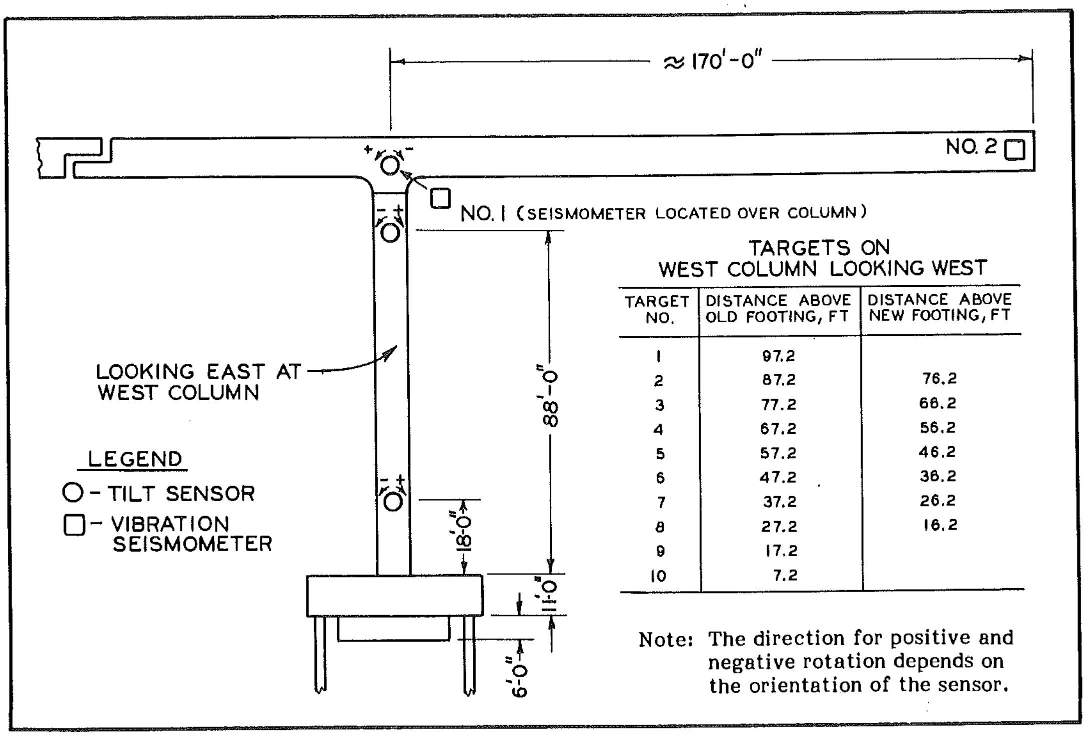
Both seismometers were placed inside the superstructure during the two intermediate lifts. One was placed over the column and one at the south end of the cantilever. Both seismometers averaged about 0.008 in. per second, with the one over the column usually giving slightly higher readings. Again these low values indicate a smooth operation. The jacks were locked in place and the central portion of the new footing was placed.

Final Lift - On January 30, 1984, the final lift of the superstructure took place. This was the most critical because the deck was separated completely from the column.

During this lift, extensometers, tilt sensors, and vibrations were monitored during jacking (Fig. 27). Elevation readings of the new footing and the columns were measured before and after jacking. Extensometers were placed on top of each column to measure relative movement of the deck to the pier. One extensometer was put on the east column and one on the west. Two vibration transducers were placed in the superstructure; one over pier 11N and one at the south end of the cantilever. One tilt sensor was placed in the superstructure directly over pier 11N. The other two were mounted on the west column to measure longitudinal movement; one on the lower portion of the column 18 ft above the new footing and the other near the top, 88 ft above the new footing.

It took about 10 minutes to equalize pressure in the jacks. During this time the extensometers showed very slight movement (0.010 in. each). Peak vibrations remained steady at 0.002 in. per second over the column and ranging from 0.001 to 0.008 in. per second at the end of the cantilever, neither of which were cause for concern.

During the jack pressurizing, the superstructure sensor showed an increase of 0.32 arc-minutes, indicating a fall of 0.15 in. at the expansion joint and a rise of 0.19 in. at the cantilever. The tilt sensor at the bottom of the column indicated a 0.06 arc-minute (0.04 in.) movement away from the river and the sensor on the top showed a 0.15 arc-minute (0.03 in.) movement toward the river. The targets on the east column appeared



TARGET NO.	DISTANCE ABOVE OLD FOOTING, FT	DISTANCE ABOVE NEW FOOTING, FT
1	97.2	
2	87.2	76.2
3	77.2	66.2
4	67.2	56.2
5	57.2	46.2
6	47.2	36.2
7	37.2	26.2
8	27.2	16.2
9	17.2	
10	7.2	

Note: The direction for positive and negative rotation depends on the orientation of the sensor.

Figure 27. Location of monitoring equipment during final lift.

to verify the direction of the column tilt sensor readings although the magnitude of movement on the east column was greater. The target located 76.2 ft above the new footing showing a 0.123 in. movement toward the river and the target located 26.2 ft above the new footing showing a 0.025 in. movement away from the river.

Once the deck started to leave the column, the extensometers measuring relative pier to superstructure movement showed a steady increase. The final readings showed a relative movement of 0.240 in. on the west side and 0.460 in. on the east side.

Peak vibrations averaged around 0.002 in. per second at both locations until the deck separated from the column. At this time, vibrations increased to a maximum of 0.030 in. per second at the south end of the cantilever and 0.072 in. per second directly over the column. These levels are perceptible but no cause for concern.

The superstructure tilt sensor showed a negative movement (south end dropping, north end rising) of 0.20 of an arc-minute as jacking started, but returned 0.18 of an arc-minute just prior to separating from the column. After lift-off, the sensor showed 0.06 of an arc-minute increase. The net change during the entire final lift was 0.04 of an arc-minute indicating that the attitude of the superstructure remained fairly constant during the lift.

After the jacks were shut down, the tilt sensor at the bottom of the column showed a fairly steady movement away from the river until the deck was off the column. This movement was 0.59 of an arc-minute (corresponding to 0.40 in. at the top of the column). The top sensor initially showed the top of the column moving toward the river a total of 0.61 of an arc-minute (corresponding to 0.13 in. at the top of the column). The sensor showed the column slowly start back the other way and then an immediate movement away from the river of 1.21 arc-minutes (corresponding to 0.27 in. at the top of the column) as the deck separated from the column. The total change from the maximum readings was 1.32 arc-minutes (corresponding to 0.30 in. at the top of the column). Targets on the east column verified the tilt sensor readings.

Elevation readings taken on January 25, 1984 showed that the four corners on each of the two columns rose an average of 0.12 in. since the readings were taken after the intermediate lifts in December. Since no readings were taken immediately prior to or after placement of the middle portion of the new footing, it is not known what effect this had on the column elevation. Final elevation readings taken on February 1 showed that these elevations increased by a total of 0.13 in.

Bearing Block Crack Measurement

After the final jacking, it was discovered that the northeast bearing block was cracked (Fig. 28). Four of these epoxy blocks are located above the jacks (as shown in Fig. 26) and support the superstructure. Pins were

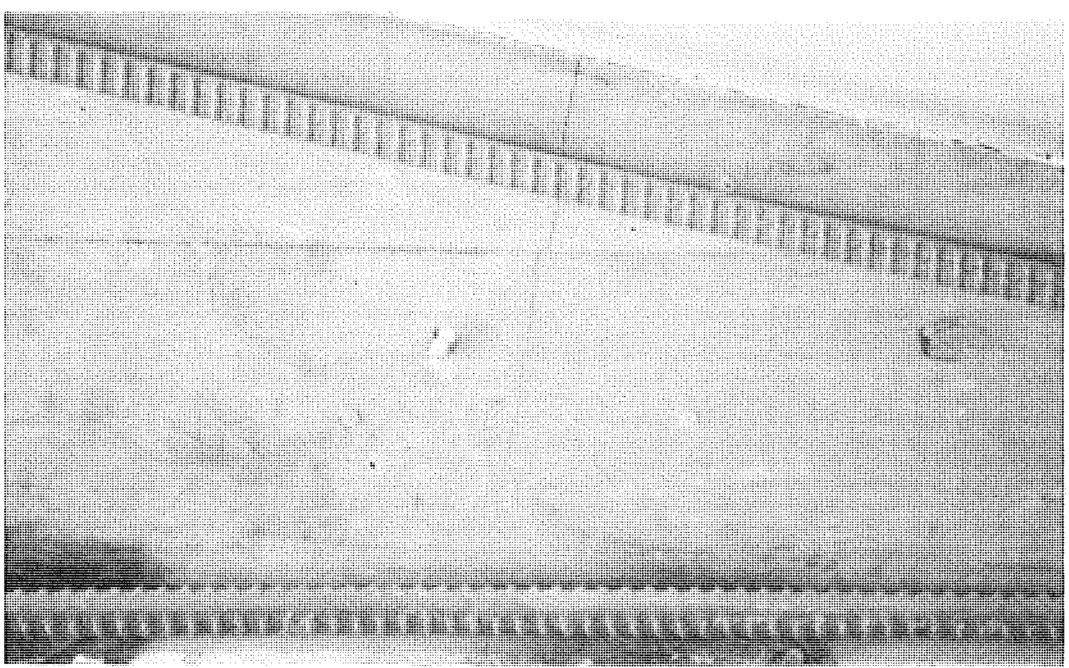


Figure 28. Cracked bearing block in northeast corner.

set at eight locations and a caliper used to monitor any movement which might take place at the cracks (Fig. 29).

Figure 29. Calipers are used to monitor bearing block cracks.

Three sets of readings were taken on January 31, 1984. Two locations, both situated at the top of the block, showed the crack to be opening. A frame was subsequently constructed to hold the block together. Construction personnel monitored the cracks after this and found no further movement had occurred.

Rotation

Introduction - The two new bearings were placed on the top of the columns, the superstructure lowered back onto them, and the most critical step of the repair procedure began. This involved rotating the superstructure back into place.

The process involved a five step procedure. First, the jacks on the north side of the falsework would be lowered 1-1/4 in. The hydraulic rams at the damaged expansion joint were then stroked 7-1/2 in., pulling down the expansion joint end of the superstructure and raising the south end. The jacks on the south side of the falsework were then raised to meet the new superstructure position. The vertical tie-down tendons were then anchored. The final step involved a survey and review of the data. This process was scheduled to be repeated six times and is illustrated in Figure 30.

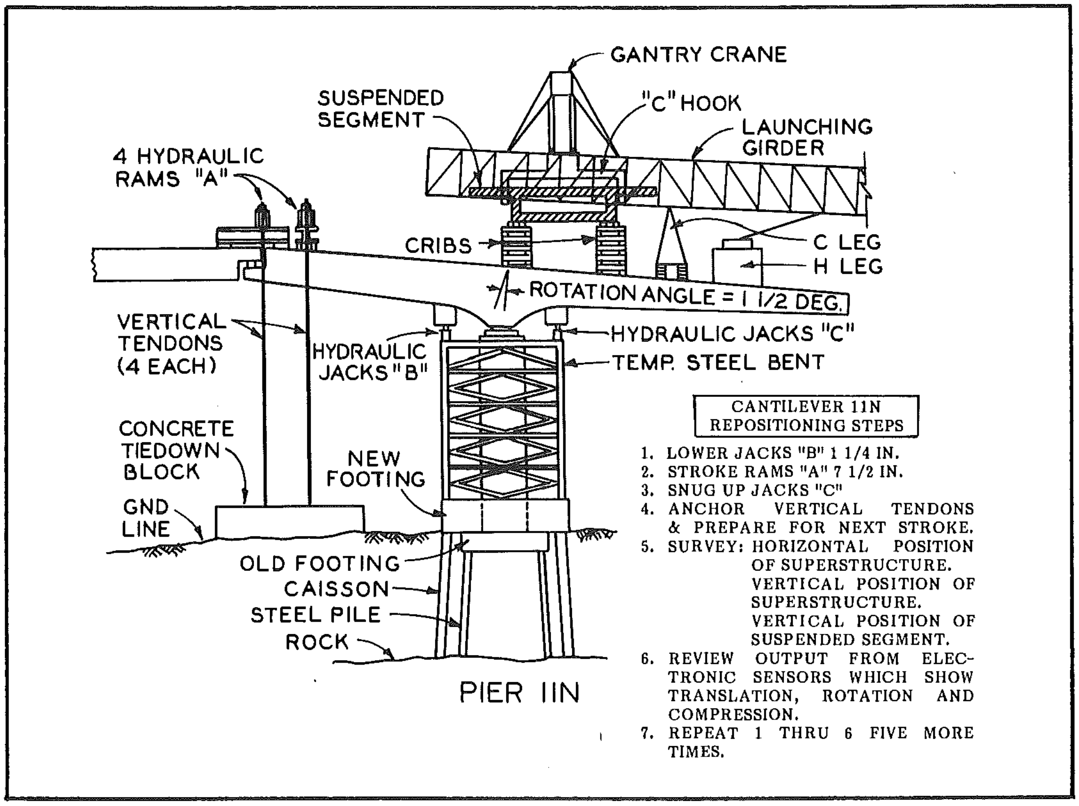


Figure 30. Diagram showing realignment procedure (after Ref. 1).

Preparation for Rotation - During the rotation procedure, tilt sensors, the top target on the east column, and the superstructure between piers 13N and 14N were monitored by the Structural Research Unit. Strain gages were placed on the additional spacer (rotation) blocks placed in the damaged expansion joint and were monitored by the Research Laboratory's Instrumentation Unit. Construction personnel monitored movement of the top of the west column, transverse movement of pier 11N, deck elevations, tie-down jack stroke and pressure, and elevation of the hanging segment. Structural Research personnel took elevation measurements on the four corners of each column, caliper readings of the segment cracks, measurements inside of the functioning expansion joint north of the river, and measurements on the top of the damaged expansion joint and the two additional expansion joints immediately north, before and after each rotation. A complete set of column target readings were also taken before and after each rotation step.

The three tilt sensors previously used during jacking were left in place. These included the one on the superstructure, one on the bottom of the column, and the one on the top of the column. A fourth sensor was added at the top of the west column to measure transverse movement. This sensor was located 88 ft above the new footing (Fig. 31).

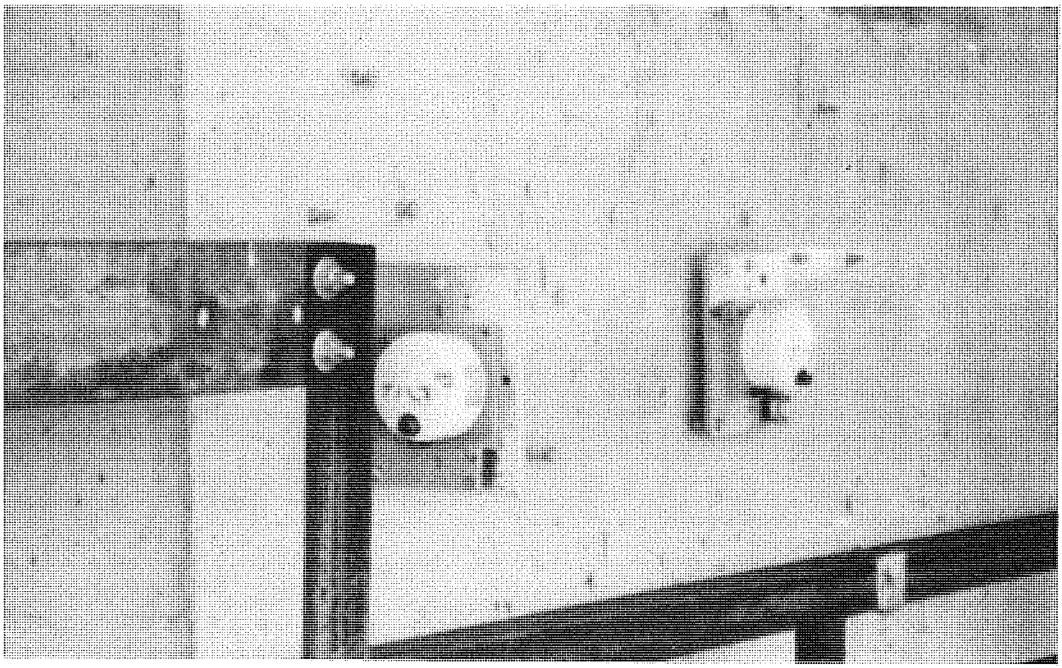


Figure 31. Two tilt sensors at top of column.

Initial Rotations - The first rotation increment took place on the afternoon of March 7, 1984. It appeared that the bearing was not sliding as it should and, therefore, could be a potential problem.

The tilt sensors showed that the south end of the cantilever rose a total of 10.54 in. and the expansion joint end dropped a total of 8.05 in. This was verified later by the Construction Division's survey crew. Both the column tilt sensors measuring longitudinal movement showed the west column moved away from the river. The bottom sensor moved a total of 1.85 arc-minutes. The top sensor moved a total of 4.20 arc-minutes. The total displacement at the top of the column was calculated to be 0.92 in. away from the river. The top target, 76.2 ft above the new footing on the east column, showed a total movement of 0.80 in. away from the river when read with the theodolite. The sensor measuring transverse movement of the west column showed a movement of 0.13 arc-minute (0.03 in. at the top of the column) to the east.

Column elevations showed virtually no change with an average for the eight readings showing a drop of 0.005 in. The caliper readings across the segment cracks showed an average closure of 0.010 in. The superstructure between piers 13N and 14N showed a total movement of 0.48 in. to the north.

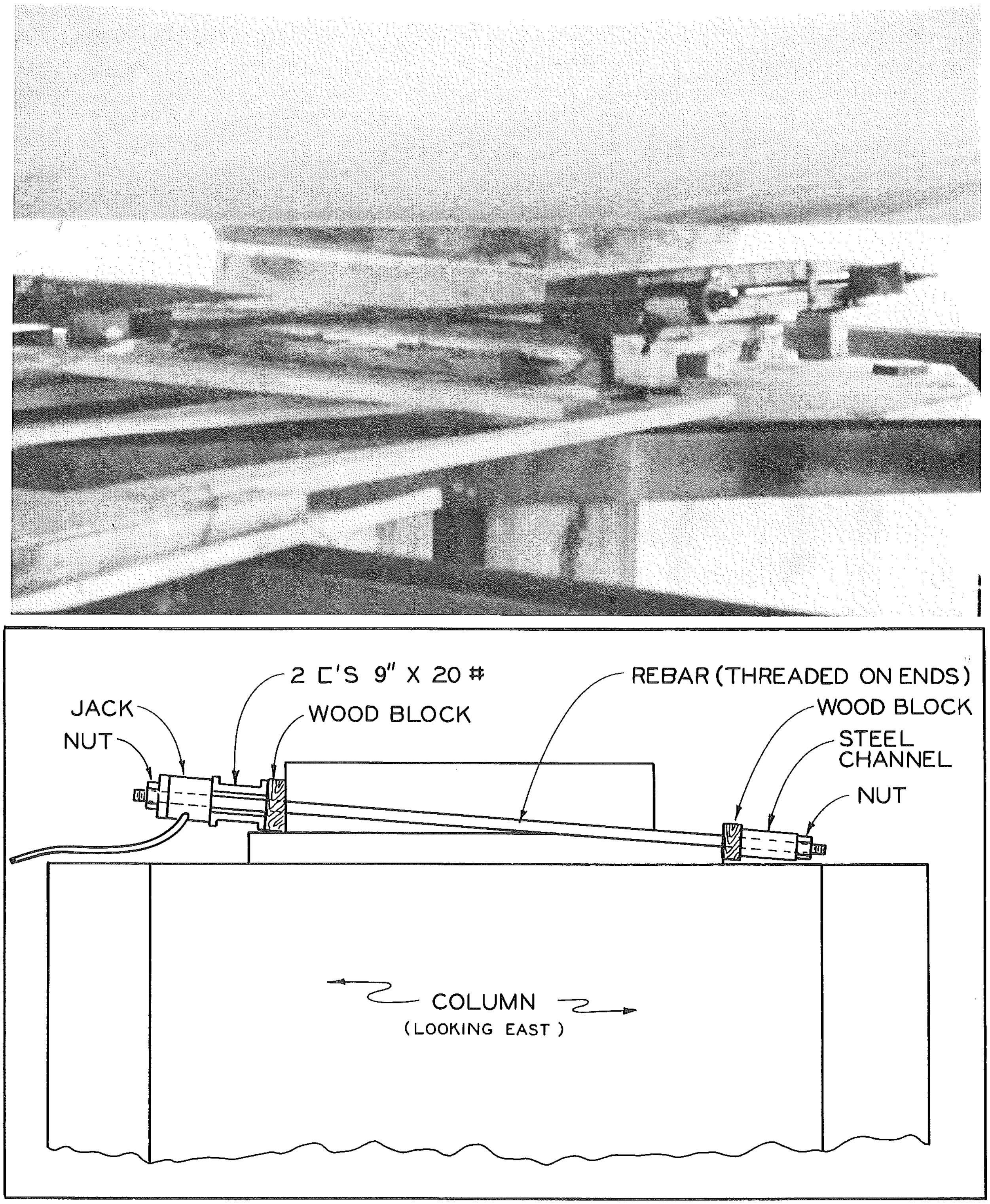


Figure 32. Bearing jacking assembly.

Second Rotation - The second rotation took place in the mid-morning of March 8, 1984. The overnight temperature drop had caused some additional movement of the bridge. The tilt sensor monitoring the superstructure showed the south end had risen 0.40 in. while the expansion joint end had dropped 0.30 in. More importantly, the column moved 1.36 arc-minutes (corresponding to 0.29 in. at the top of the column) back toward the river. The top target showed a movement 0.15 in. in the same direction.

The superstructure went up 6.80 in. at the south end and down 5.20 in. at the expansion joint end during this rotation. The column's longitudinal movement was 1.41 arc-minutes at the bottom and 3.32 arc-minutes at the top, both away from the river. The calculated deflection corresponded to 0.72 in. at the top of the column. The top target showed a 0.650 in. deflection away from the river. The transverse movement was 0.53 arc-minutes (corresponding to 0.12 in. at the top of the column) toward the west.

Column elevations fell an average of 0.017 in. The segment cracks closed an average of an additional 0.003 in. The superstructure between piers 13N and 14N showed no movement.

Displacement measurements at the top of the damaged expansion joint showed an average opening of 0.031 in. and the first expansion joint north of the river closed an average of 0.812 in. The two sets of pins in the bottom of the damaged expansion joint moved virtually the same amount, showing an average opening of 0.544 in.

During this rotation step, the bearings at the top of the pier still did not slide. It was decided to build a jacking system around these bearings, to induce the sliding motion that had to occur.

Bearing Jacking - Because of the time taken to assemble the bearing jacking apparatus (Fig. 32), this operation was not started until the late afternoon of March 9, 1984. The column had moved 0.62 of an arc-minute toward the river during the night, but moved back away from the river 1.53 arc-minutes during the day. This meant that the total displacement of the column since the previous rotation was an additional 0.20 in. to the south at the top of the column. The top target also showed a 0.200 in. movement. This brought the total column displacement to approximately 1-1/2 in. at the top since the beginning of the initial rotation.

Overnight, the south end of the superstructure had risen 0.45 in. and the expansion joint end dropped 0.34 in. During the day, the south end fell 0.09 in. and the expansion joint rose 0.07 in. Total displacement of the superstructure between the March 8 rotation and the March 9 bearing jacking was a 0.36 in. rise at the south end and 0.27 in. drop at the expansion joint end.

The bearing jacking was done in two steps. After completion of this operation, the sensor in the superstructure showed the south end came up a total of 0.64 in. and the expansion joint dropped in elevation a total of 0.49 in.

During the first step, the sensor measuring transverse movement recorded a rotation of 0.16 arc-minutes (0.04 in.) to the west at the top of the column. This trend appeared to be caused by one bearing moving more than the other. The second step of this rotation showed very small transverse movement; 0.02 of an arc-minute. It was assumed that both bearings were now moving freely.

The sensor at the bottom of the column showed a movement away from the river 0.97 of an arc-minute. The sensor at the top showed a movement of 2.46 arc-minutes in the same direction. This corresponds to a calculated displacement of 0.54 in. away from the river at the top of the column. The column targets showed a 0.500 in. movement in the same direction.

The superstructure movement between piers 13N and 14N showed a 0.36 in. movement to the north. All other measurements were taken before jacking; however, by the time the rotation step was completed, it was too dark to get an 'after' set of measurements, and no comparison could be made.

Third and Fourth Rotations - The bearings appeared to be moving as designed, so on March 10, 1984, two more rotations were attempted. Overnight changes showed little movement; the top of the column moved back to the river only 0.16 in., the south end of the cantilever rose 0.12 in., and the expansion joint end dropped 0.09 in.

The superstructure movement was uniform for both rotations. The third rotation showed a displacement of 11.68 arc-minutes, bringing the south end up 6.90 in. and the expansion joint end down 5.30 in. The fourth rotation showed a 13.1 arc-minute change, bringing the south end up an additional 7.80 in. and the expansion joint down 5.90 in. Total superstructure movement for the day showed the south end rose in elevation an additional 14.70 in. and the expansion joint dropped 11.20 in.

The longitudinal column sensor showed some movements which had not been seen before. During the third rotation, the bottom sensor moved steadily 0.88 of an arc-minute away from the river and then 0.15 of an arc-minute back. The top of the column moved 2.11 arc-minutes away from the river and then 0.70 of an arc-minute back. This meant the top of the column moved 0.46 in. south and back 0.15 in. for a net displacement of 0.31 in. to the south.

During the 56-minute shutdown time between rotation steps, the bottom of the column moved back away from the river 0.12 of an arc-minute and the top 0.23 of an arc-minute in the same direction. This was an additional 0.05 in. movement to the south at the top of the column.

The column showed much of the same types of movement during the fourth rotation. The bottom sensor showed a movement of 0.18 of an arc-minute away from the river and 0.30 of an arc-minute back. The top sensor showed a movement of 0.68 of an arc-minute away from the river and 0.92 of an arc-minute back. This meant that the total displacement for the fourth rotation was 0.24 of an arc-minute toward the river, or 0.05 in. at the top of the column. Since static friction always exceeds dynamic friction, this type of 'stick-slip' movement is to be expected.

The calculated total movement of the top of the column was 0.31 in. away from the river for both rotations. The target showed a total movement of 0.400 in. in the same direction.

The superstructure movement between piers 13N and 14N showed a 0.12 in. movement to the north for both rotations. An attempt was made to take a set of elevation readings after jacking, but windy conditions made the rod virtually impossible to read. The same windy conditions made the trammel point readings on top of the deck rather dangerous since the points were close to the edge and, therefore, were not taken. The segment crack readings showed an average closure of 0.011 in. and the bottom expansion joint pins in the inside showed an average opening of 1.191 in.

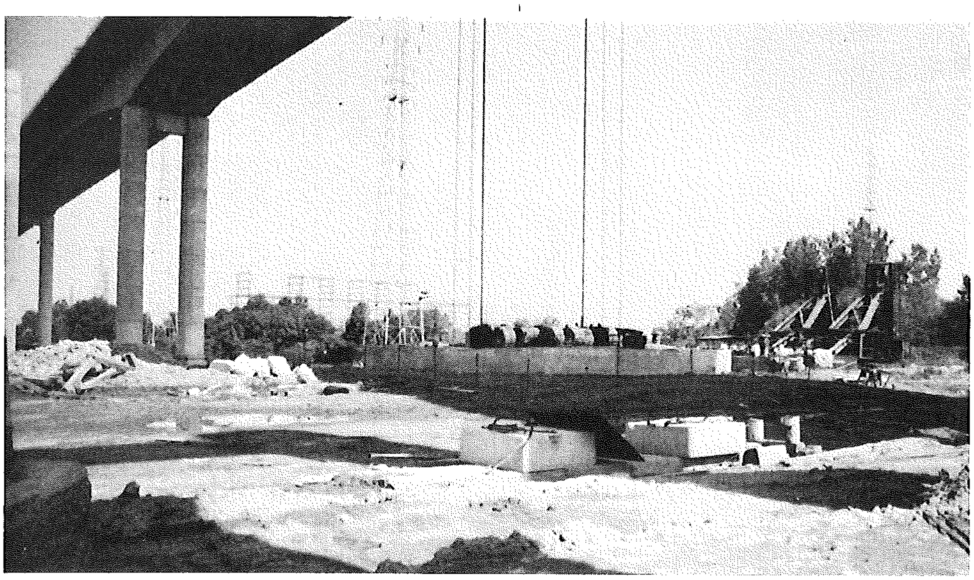


Figure 33. Weighted block with extra tie-down cable.

Final Rotations - Two extra cables were added between the tie-down block and the expansion joint and two extra hydraulic rams added. Additional weight was also added to the tie-down block (Fig. 33). This took an additional two weeks and on March 23, 1984 the final rotation step was ready to take place.

The superstructure moved a total of 27.67 arc-minutes during the two final rotations. This brought the south end up an additional 16.40 in. and the expansion joint end down 12.60 in. The superstructure was now back to its approximate original alignment (Fig. 34).

During these rotations, the column showed much less movement than during the previous rotations. The bottom column sensor measuring longitudinal movement showed a movement of 0.11 of an arc-minute away from the river, then 0.26 of an arc-minute toward the river and then 0.16 of an arc-minute away for a total displacement of 0.01 of an arc-minute toward the river. The top of the column moved in much the same manner; 0.21 of an arc-minute away, 0.69 of an arc-minute toward and 0.38 of an arc-minute away for a total displacement of 0.10 of an arc-minute toward the river. This overall displacement at the top was 0.02 in. toward the river. The column target readings confirmed this back and forth or 'stick-slip' movement.

Figure 34. Zilwaukee bridge after completion of the realignment procecure.

The transverse movement was basically stable moving a maximum of 0.02 of an arc-minute. There was no movement of the superstructure between piers 13N and 14N. There was virtually no change in the column elevation readings.

Segment crack readings were not monitored immediately after jacking but were taken a week later and showed an average closure of 0.003 in. Measurements at the bottom of the expansion joint showed an average opening of 1.280 in.

Summary of Rotation Measurements - The tilt sensors showed the final displacements of the column in the longitudinal direction to be 3.74 arc-minutes away from the river at the bottom and 8.78 arc-minutes away from the river at the top. This corresponds to a calculated total displacement of 1.90 in. at the top of the column. The sensor measuring transverse movement showed a total displacement of 0.12 in. to the west. The superstructure showed a total of 86.4 arc-minutes, or 4.30 ft up at the south end and 3.30 ft down at the expansion joint. All of these total movements include expansion and contraction changes which took place between rotations and all the final displacements will continue to change slightly as temperature changes.

The superstructure showed a total movement between piers 13N and 14N of 0.96 in. to the north during all the rotation steps; however, temperature changes between rotations were not taken into account and probably would change the final measurement. The 18 segment crack measurements showed an average closure of 0.020 in. ranging from no change to a 0.047 in. closure. A permanent change in the expansion joint was not calculated because these measurements vary so much with temperature change. There was virtually no movement in the elevation of the columns.

CONCLUSIONS

The major conclusion of the study was that although the damaged structure moved with changes in temperature, there was no significant long-term net movement or instability. This finding allowed the repair contract to be planned, designed and executed.

The Zilwaukee bridge monitoring proved that structural movement can be monitored with a high degree of accuracy. With the specialized equipment used by the Research Laboratory, it was demonstrated that monitoring could be done to tolerances of a few thousandths of an inch, which is considerably more precise than required for normal construction work. Such accuracies are required to determine trends in movements prior to major structural shifts that might occur under the uncertain circumstances that prevailed.

The Sperry tilt-sensing system provided data that correlated quite closely with the mechanical measuring devices and provided the same high accuracy. It also had the advantages of providing a gravity reference independent of surrounding unstable soil and providing a continuous output 24 hours a day that could utilize preset thresholds to trigger a warning alarm. This system, retained by the Structural Research Unit, provides the Department with an additional means of monitoring structures in the future. The monitoring during key periods in the repair process provided the Construction Division and consultants with the information needed to make critical decisions, in an extremely difficult and complex engineering task.

This report discusses the trends found during the monitoring period. Data are usually presented for the high and low measurements, along with averages. Detailed information concerning any of the areas discussed can be obtained by writing to the Engineer of Research, Michigan Department of Transportation.

REFERENCES

1. Snyder, Gus, "Repairs Completed on the Zilwaukee Bridge," Michigan Contractor and Builder, April 21, 1984.
2. VanKampen, Adrianus, "Zilwaukee Bridge Update," Michigan Society of Professional Engineers, February 1984.
3. Zilwaukee Construction Engineering, Inc., "Cause of Construction Failure in Spans 11N-12N of the Zilwaukee Bridge," March 1, 1983.
4. Howard, Needles, Tammen, and Bergendoff, "I-75 Crossing the Saginaw River near Zilwaukee, Michigan. Investigation of Construction Failure in Spans 11 and 12," January 1983.
5. T. Y. Lin International, "Zilwaukee Bridge Failure Study Summary," March 10, 1983.
6. The United States General Accounting Office Report to the Honorable Donald W. Riegle, Jr., United States Senate, "Early Decisions and Delays on the Zilwaukee Michigan Bridge Project," August 17, 1983.
7. The United States General Accounting Office Report to the Honorable Donald W. Riegle, Jr., United States Senate, "Delays and Increased Cost Result From the Zilwaukee, Michigan, Bridge Project Mishap," June 27, 1984.

APPENDIX A

Precast Segmental Construction
of the Zilwaukee Project

From the United States General Accounting Office Report to the Hon. Donald W. Riegle, Jr., United States Senate, "Delays and Increased Cost Result from the Zilwaukee, Michigan, Bridge Project Mishap," June 27, 1984. (Ref. 7)

PRECAST SEGMENTAL CONSTRUCTION

ON THE ZILWAUKEE PROJECT

At Zilwaukee the concrete bridge segments were precast in a plant built by the contractor at the bridge site. The segments were match cast (precast against each other) to ensure proper fit. The contractor used the balanced cantilever assembly method and a launching girder to position the segments on the bridge.

The balanced cantilever method involves first placing and anchoring a segment atop a support pier. Then, additional segments are alternately placed at each end of the pier segment out to mid-span. Each pier supports projecting segments (cantilevers), like wings, on each side of it. A pier frame (temporary framework) helps support the load on the pier while the cantilevers become unbalanced and then balanced again as segments are added. Where the two cantilevers from adjacent piers meet at mid-span, a concrete segment is cast in place between them to close the gap, forming a completed span. This procedure is repeated until the structure is completed.

Each segment added is attached to the already completed portion of the structure with temporary steel bars. They are joined to each other with permanent tendons (bundles of steel strands) stretched and anchored at each end (post-tensioning) after a segment is erected at the end of each cantilever. Before the segments are aligned and tightened to the already completed portion of a cantilever, their faces are coated with epoxy. This provides additional support, but the epoxy's main purpose is to seal the joint preventing the intrusion of moisture.

At Zilwaukee a typical tendon consisted of twelve 1/2-inch prestressing steel strands. Tendons were installed by being pulled through voids in the concrete segments formed by galvanized ducts. Next, they were tensioned using jack devices. After post-tensioning, the tendons were grouted to provide corrosion protection and to develop a bond between the steel and the surrounding concrete.

Segment manipulation was accomplished with a launching girder. A girder is a special mechanism that travels along the completed deck spans and maintains the work flow at that level. The essential parts of a typical launching girder are a main truss (group of steel beams forming a framework) with a length somewhat greater than the maximum bridge span, leg frames which are attached to the main truss, and a trolley which travels along the girder and is capable of moving a concrete segment in longitudinal, transverse, and vertical directions. During construction, by assuming various positions, the girder can place segments in cantilever, can place segments over piers, and can move to the next span so that the construction process can be repeated. This is the fastest method of cantilever construction, but it is limited to large projects because of the high initial cost of the launching girder.

The girder used at Zilwaukee weighs about 1,200 tons; its 940-foot length allowed the contractor to erect segments on two spans simultaneously. A 117-ton crane on the girder lifted segments from a delivery truck, travelled along the top of the girder, and carried the segments forward to their appropriate location. The girder has four movable legs, one fixed leg, one launching device to move the girder, and one leg to aid in placing pier segments on the next pier. The legs rest on the bridge until needed and are then positioned below the girder as supports. The legs are moved from span to span by the launching girder.

APPENDIX B

Monitoring Procedures

Footing Elevations

Elevations of the footing were taken using a precise level with an optical micrometer (Fig. B1) and an invar level rod . An invar rod was chosen because of its extremely low coefficient of thermal expansion. These instruments made it possible to read elevations to the nearest 0.0001 ft.

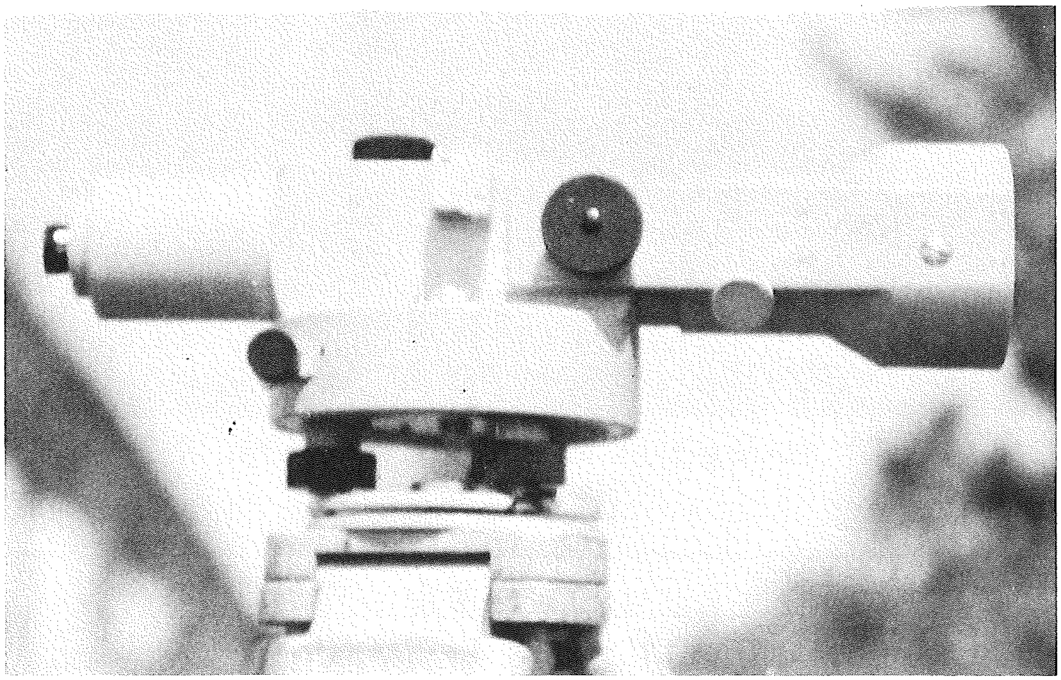


Figure B-1. Precise level with optical micrometer.

Initially a monument (No. 4) was placed 44 ft east of the damaged footing and an elevation established using an existing benchmark. During the course of Phase I readings, this monument appeared to be sinking. Another monument (No. 2) was placed approximately 200 ft northeast of the footing and its elevation established. A third monument (No. 5) was set about 60 ft west of monument No. 2. Figure B2 shows monument No. 5 and the invar rod.

Reference monuments were set about 8 ft into the ground, and an invar bar used to raise the reference level up to where the level rod could bear against it. This was done to protect the reference point from frost heave, and to maintain a precise amount of rise regardless of the temperature that varied widely from summer to winter.

Points were established by placing bolts in the four corners on top of the existing footing. These were used for monitoring throughout Phases I and II. The locations of these points are shown in Figure B3.

Figure B-2. Monument No. 5 and invar rod.

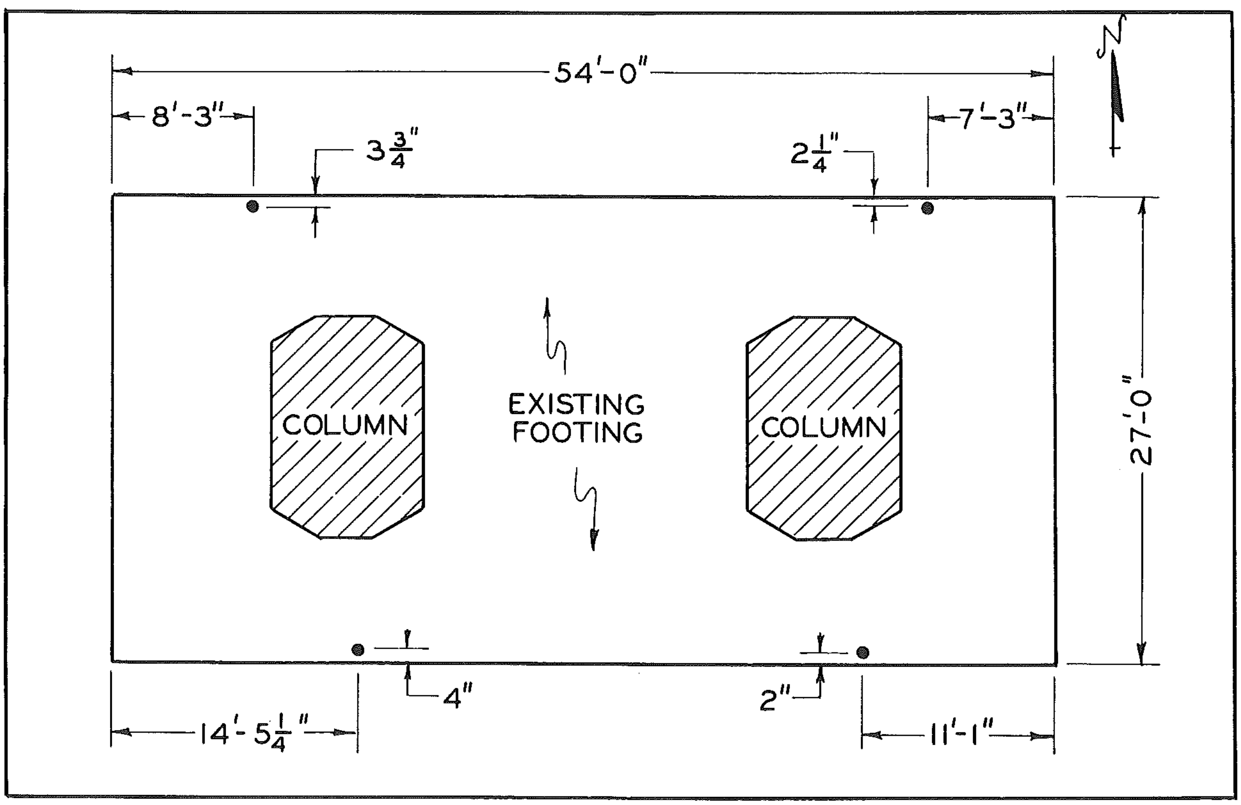


Figure B-3. Location of points for monitoring elevation of damaged footing.

After the two outside portions of the new footing were completed, two points were established on each portion using L-brackets, and used during Phase III monitoring. Also during Phase III, there was some concern as to what the existing footing was doing after the middle section of the new footing was placed on top of it. L-brackets were placed on four corners of the existing footing walls (Fig. B4).

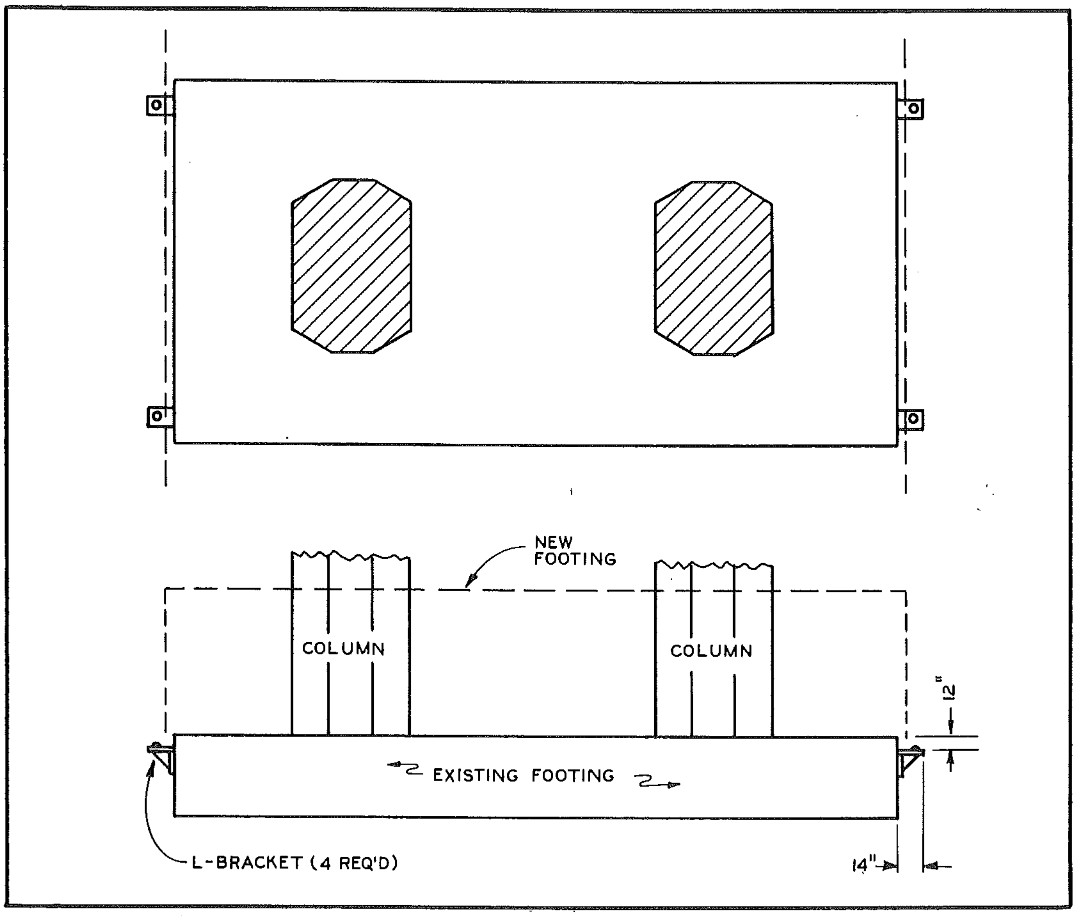


Figure B-4. Brackets for establishing elevation of old footing after new footing was in place.

Figure B5 shows the complete layout used for establishing the footing elevations. Instrument set-ups are representative of all three phases.

Column Elevations

Column elevations were measured to the nearest 0.0001 ft using the same instrumentation as that described in the footing elevation section. Four L-brackets were mounted on the four corners of each of the twin towers just above the damaged footing and read during Phase I and II

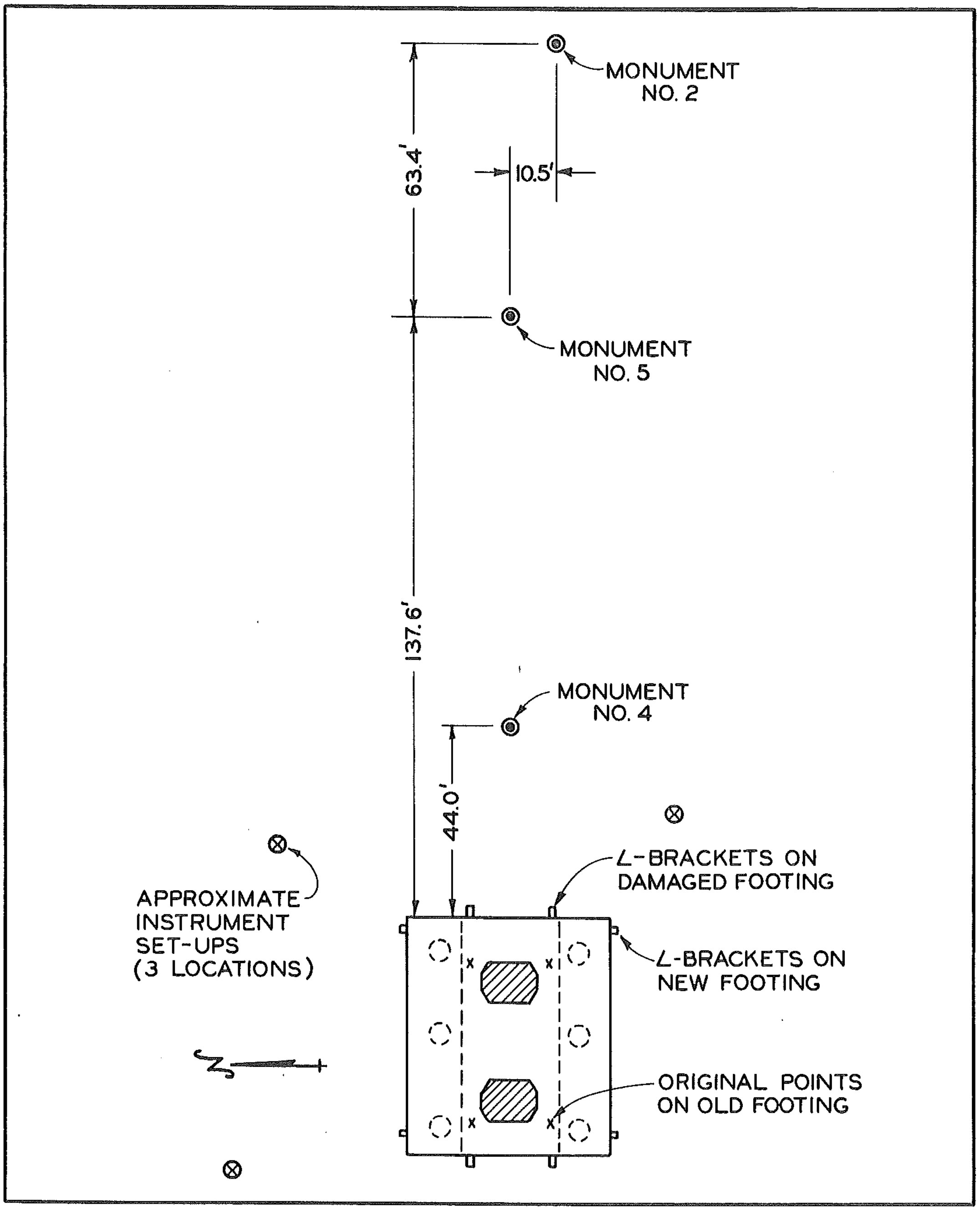


Figure B-5. Layout for establishing footing elevations.

monitoring. After the new footing was completed these brackets were re-established about 12 ft up on the columns so the elevations could be monitored during Phase III. Figure B6 shows the invar rod on one of the points and Figure B7 shows the layout.

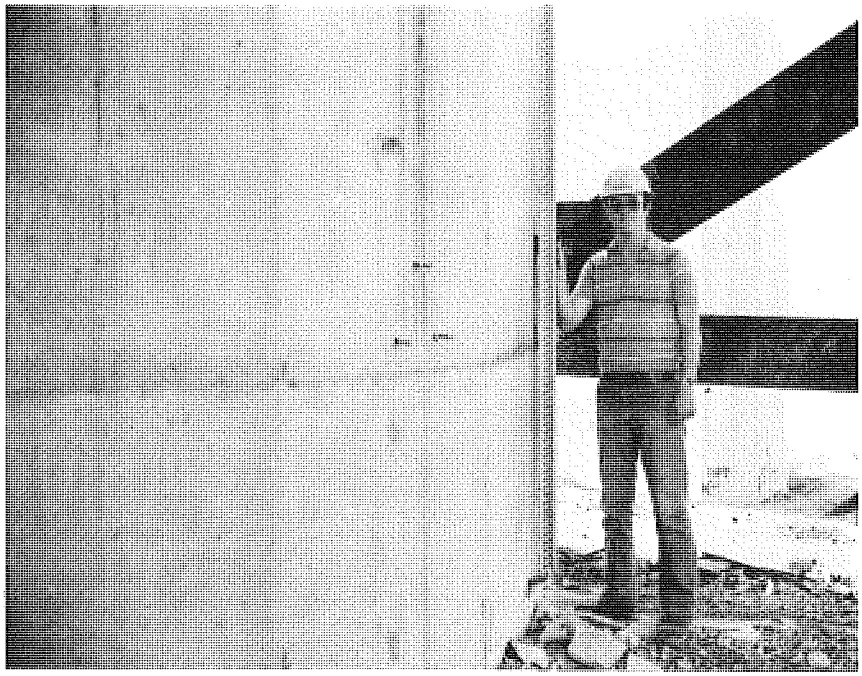


Figure B-6. Establishing elevation of the column.

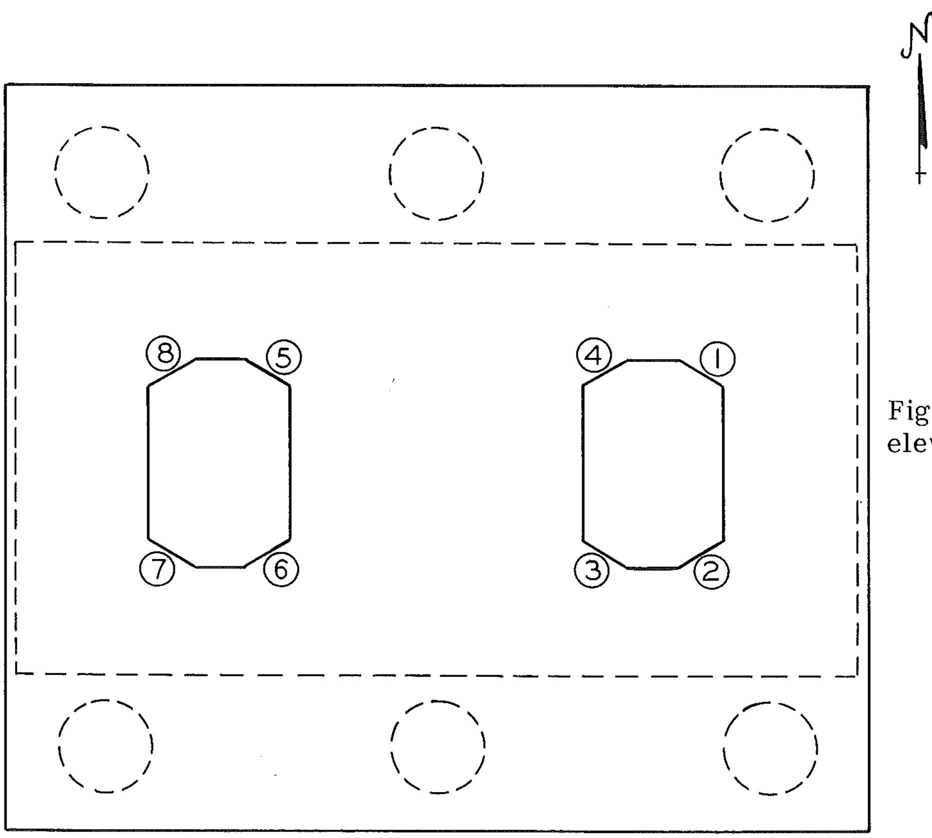


Figure B-7. Column elevation points.

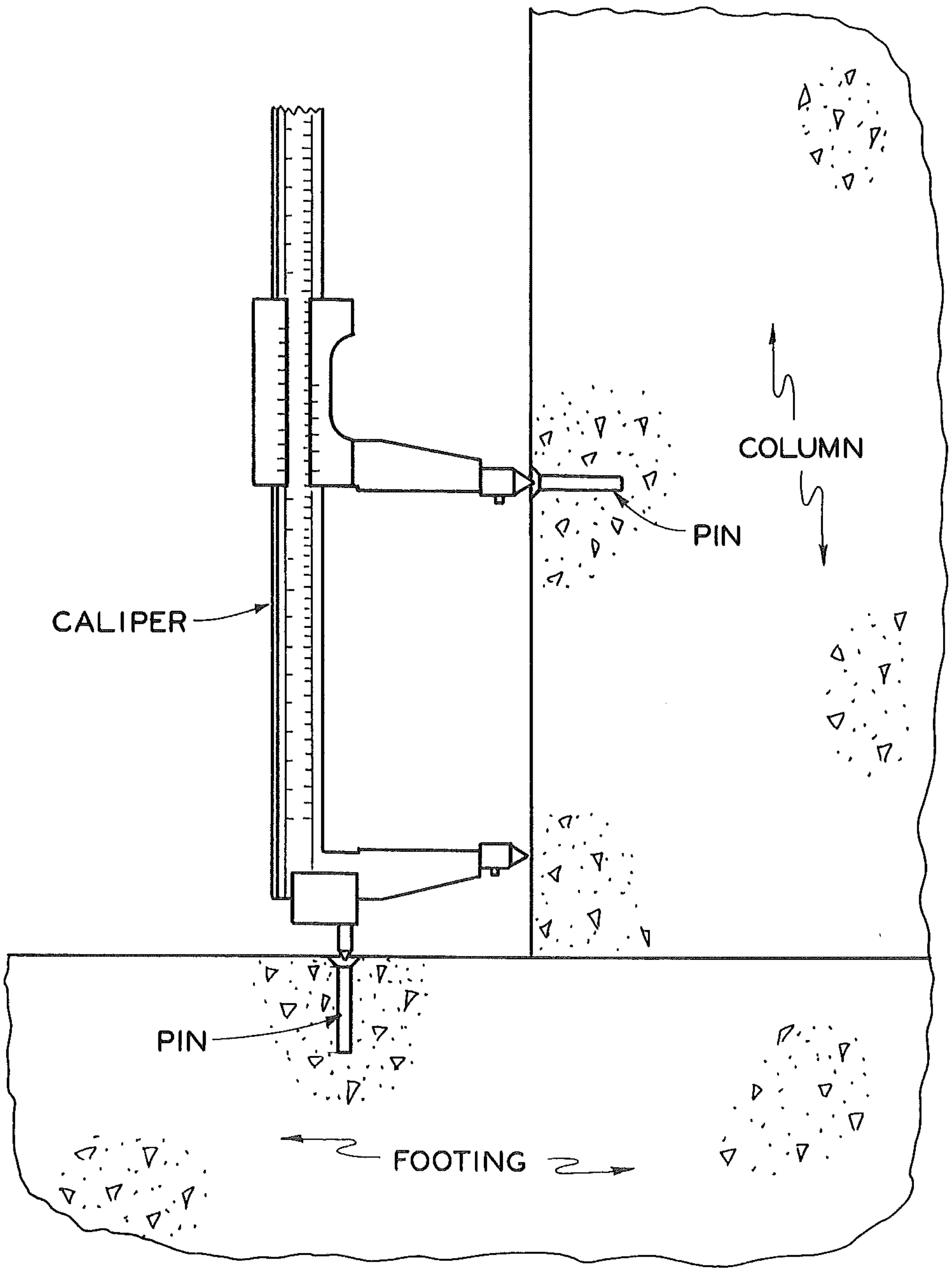


Figure B-8. Measuring column-footing relative movement.

Column-Footing Relative Movement

Pairs of pins were placed in the eight locations just below the brackets used for column elevations. One pin was placed in the footing and the other in the column and a vernier caliper used to measure the relative displacement to the nearest 0.001 in. (Fig. B8). These measurements were taken during Phases I and II.

Longitudinal Column Movement

Targets utilizing a 6-in. machinist scale (Fig. B9) were mounted on the east side of the east column. Ten of these targets were placed, starting 7 ft 2-1/2 in. above the damaged footing and every 10 ft thereafter, putting the top target (Target No. 1) 97 ft 2-1/2 in. above the top surface of the original footing. These targets were read throughout Phases I and II. After the new footing was completed and the falsework erected, only targets No. 2 through No. 8 could be read for Phase III monitoring; No. 8 being 16.2 ft above the new footing and No. 2 being 76.2 ft above the new footing.

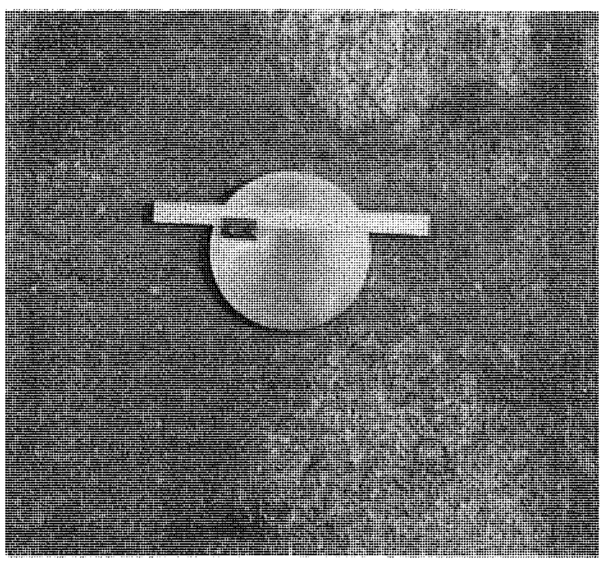


Figure B-9. A column target.

A theodolite set up over a reference monument was used to read these targets to the nearest 0.1 in., with a close estimation made to the nearest 0.025 in. A straight line was established by backsighting on two targets; one about 50 ft behind the instrument and the other 110 ft behind. (Fig. B10).

All initial target readings were zero. A more positive reading indicates a movement away from the river and a negative reading, movement toward the river.

Figure B-11 shows the location of the targets on the column and the general site layout.

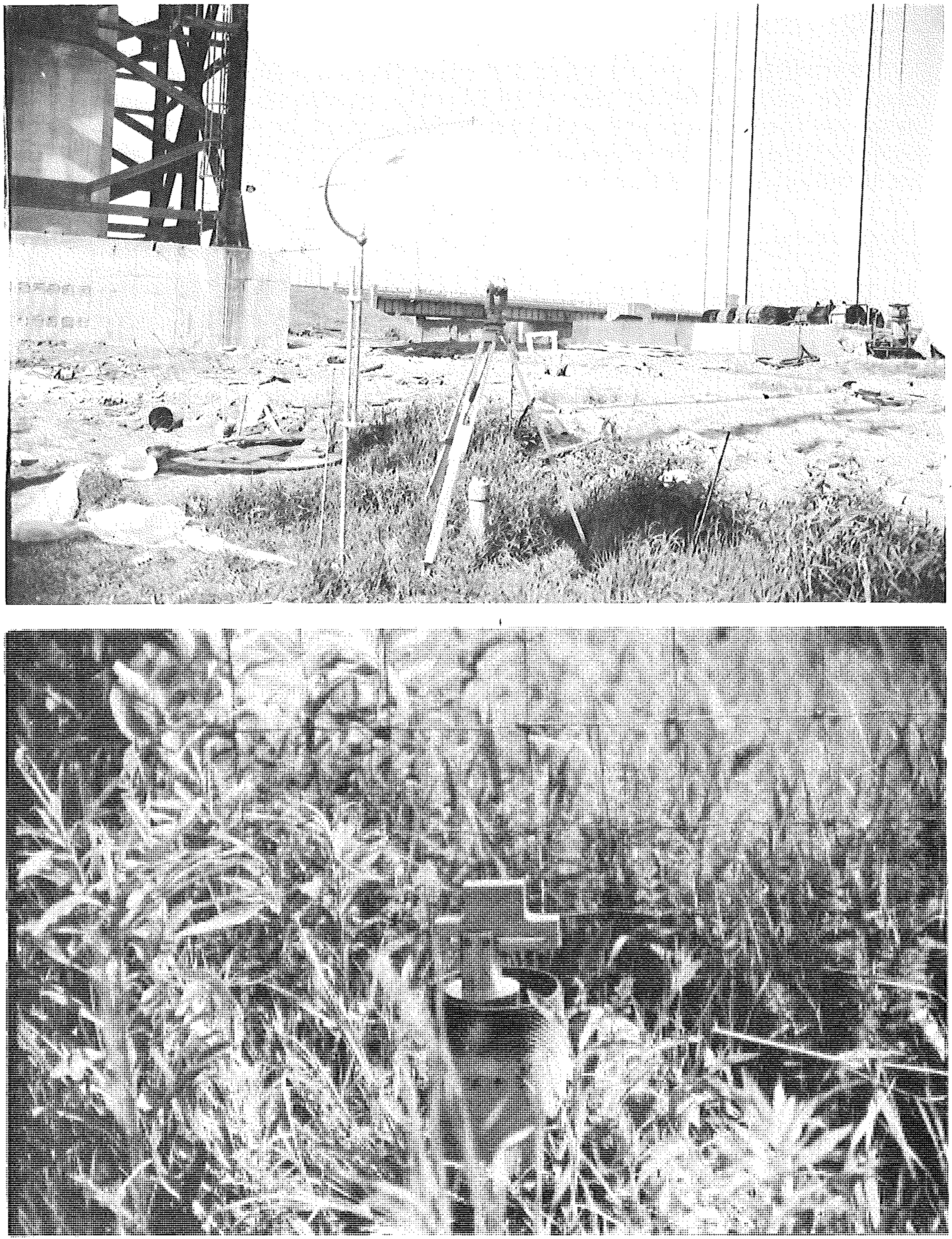
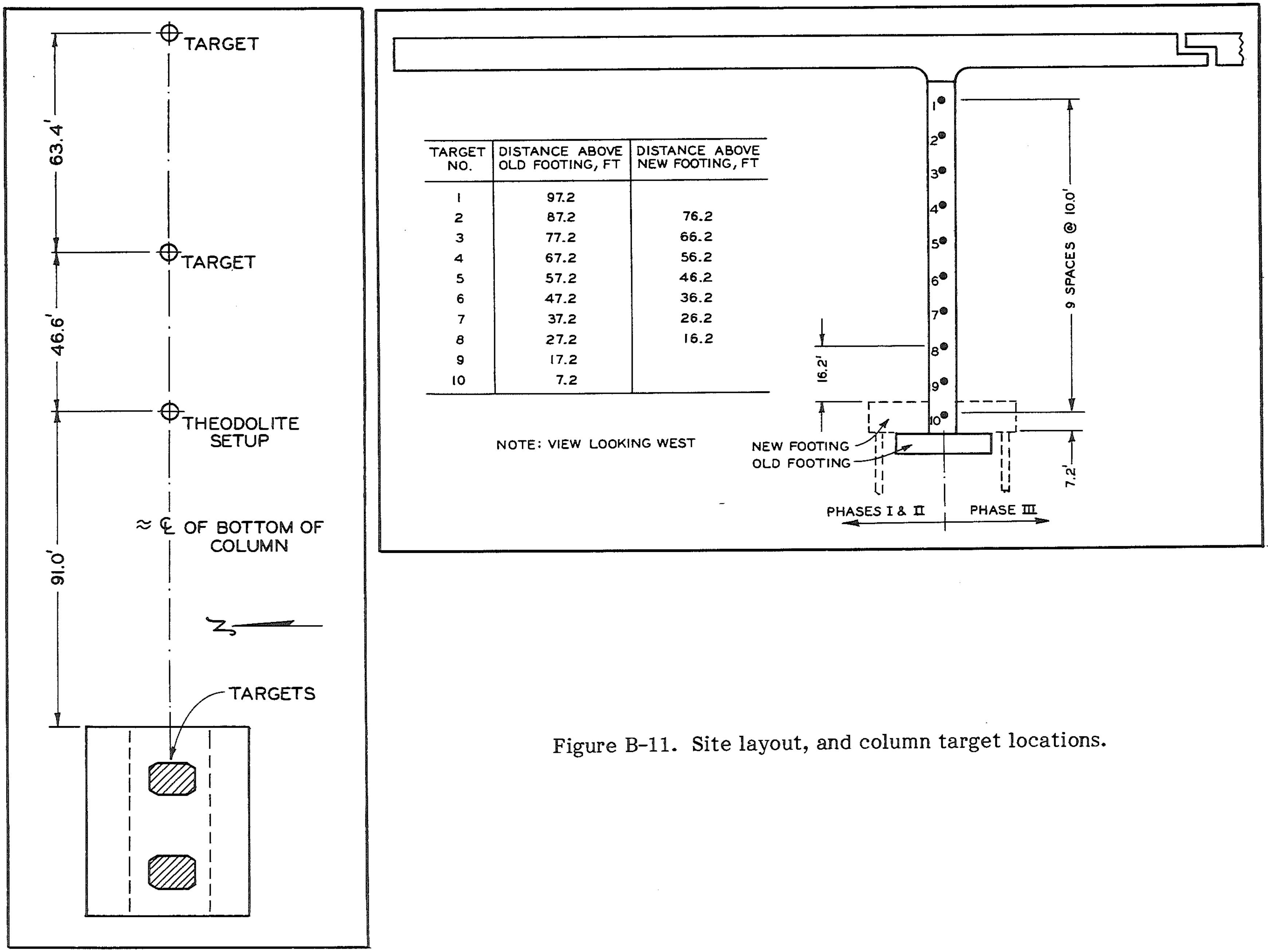


Figure B-10. Theodolite (above) for reading column targets (with umbrella to shade the instrument bubble), and backsight target (below).



TARGET NO.	DISTANCE ABOVE OLD FOOTING, FT	DISTANCE ABOVE NEW FOOTING, FT
1	97.2	
2	87.2	76.2
3	77.2	66.2
4	67.2	56.2
5	57.2	46.2
6	47.2	36.2
7	37.2	26.2
8	27.2	16.2
9	17.2	
10	7.2	

Figure B-11. Site layout, and column target locations.

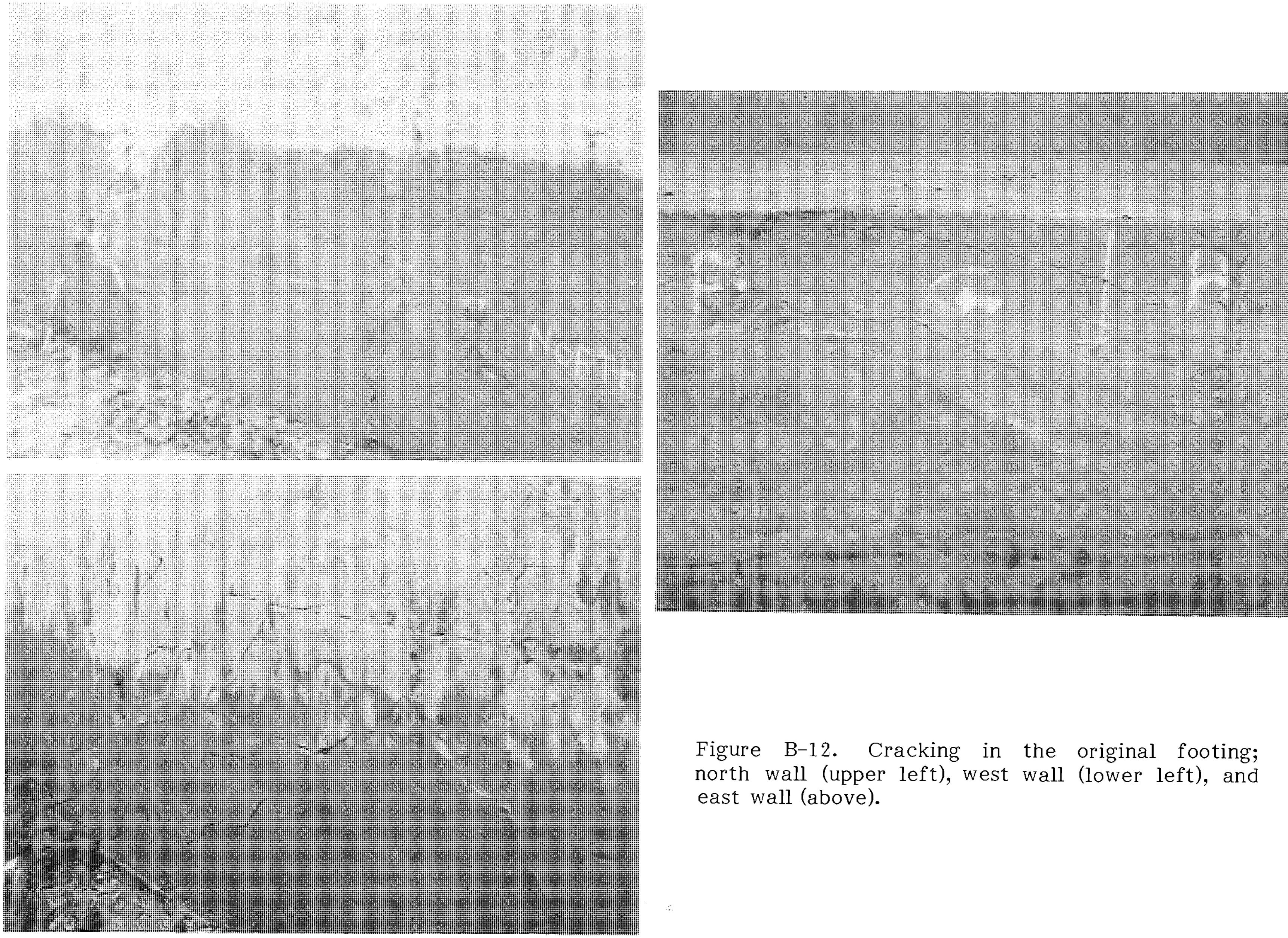


Figure B-12. Cracking in the original footing; north wall (upper left), west wall (lower left), and east wall (above).

Movement of Cracks in Footing Side

Pins were placed at various locations at the cracks found in the sides of the footing. A vernier caliper was used to measure crack movements to the nearest 0.001 in. during Phase I. Figure B12 shows the cracking in each side (No pictures are available for the south wall).

Crack on Top of Footing

Pins were set on either side of the major fracture in the top of the footing between the two columns (Fig. B13). A vernier caliper was used to measure the crack movement to the nearest 0.001 in. during Phases I and II. An inclinometer, which measures vertical movement of two reference points, was used during Phase I to measure the crack faulting to the nearest 1/64 in.

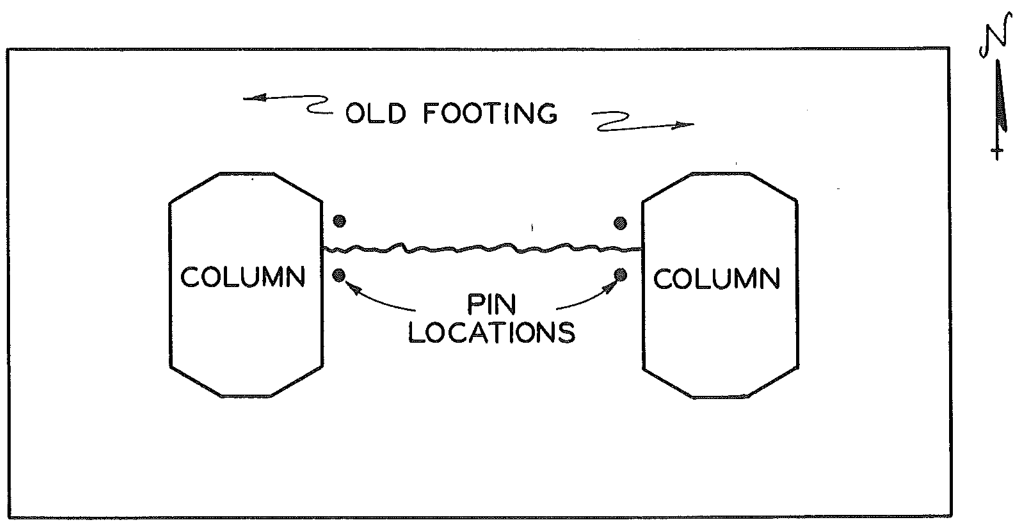


Figure B-13. Pin location for monitoring crack movement on top of footing.

Cracks on Bottom of Segments

Pins were placed along segment joints and cracks inside the superstructure as shown in Figure B14. Pins 1 through 10, 12 through 14, and 16 through 18 were placed on cracks or joints in the floor and 11 and 15 on the walls. The original surface crack width was measured to the nearest 0.001 in. with the calibrated eyepiece in the small microscope shown in Figure B15. A vernier caliper was used from then on to measure relative change in these cracks to the nearest 0.001 in. (Fig. B16).

Pin numbers 19 and 20 were used in conjunction with the unmarked pins to measure overall change in length of the cracked section of the bottom of the superstructure. An invar tape was used along with the

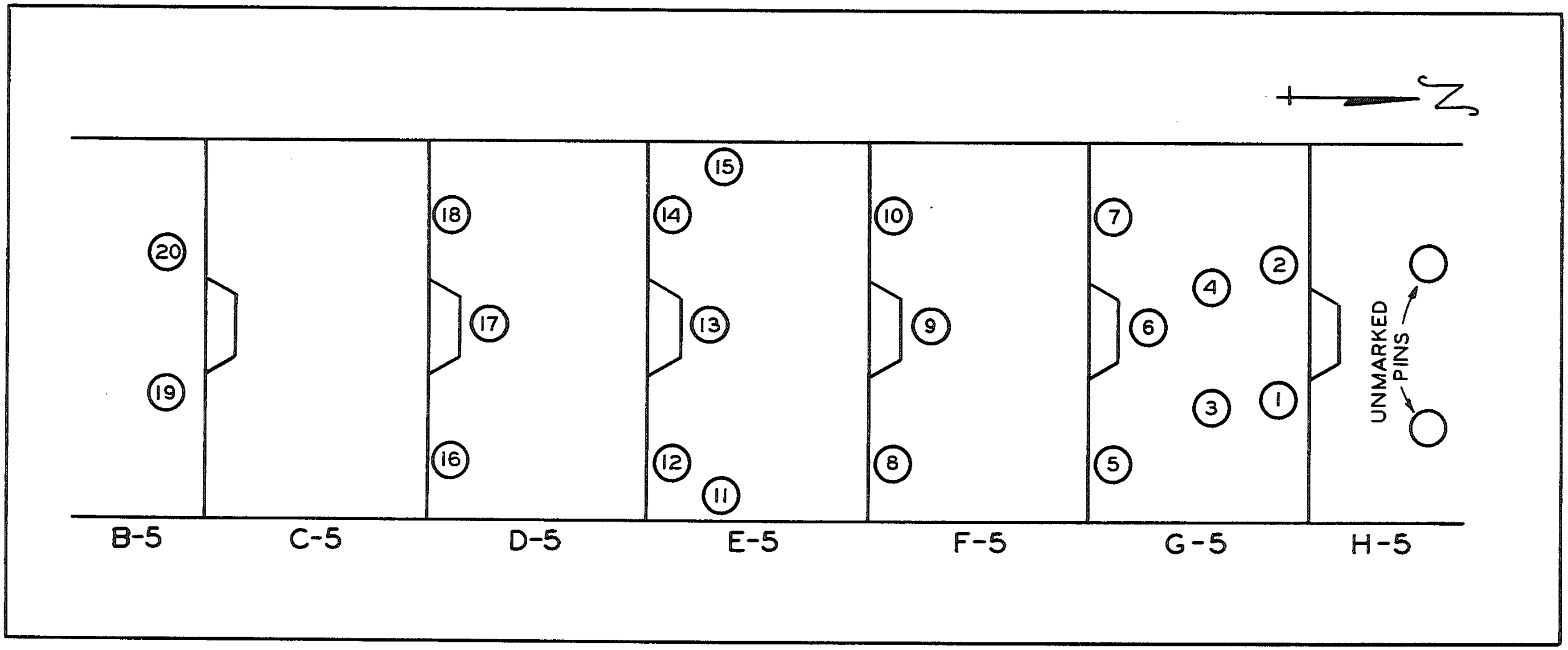


Figure B-14. Pin locations for monitoring segment cracks.

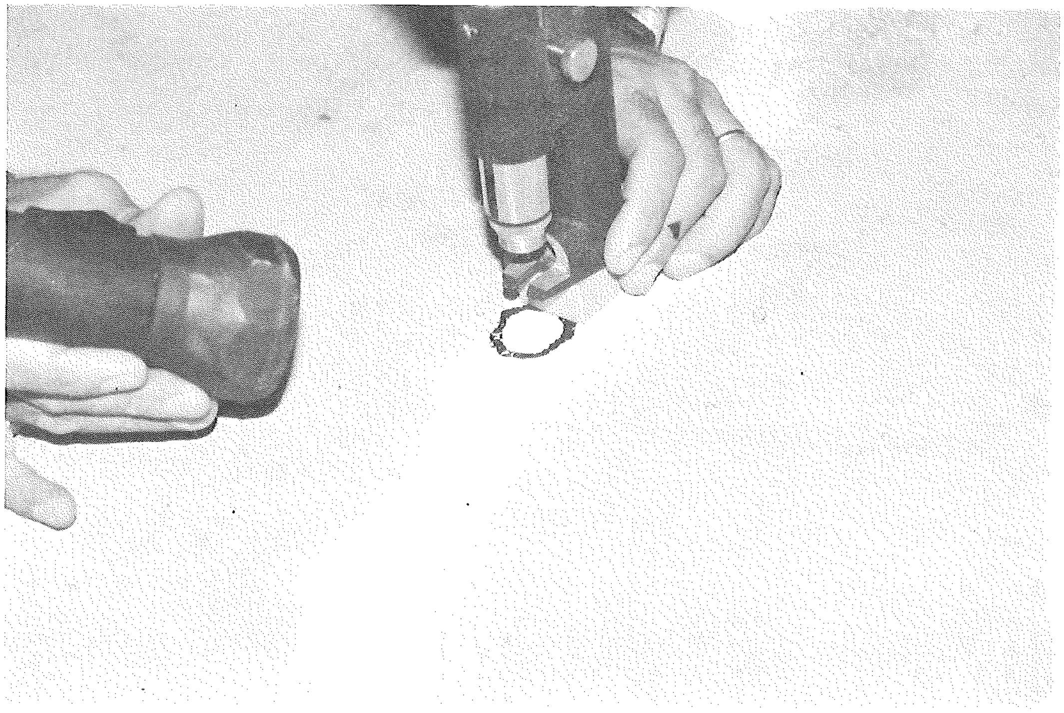


Figure B-15. Crack widths inside superstructure were measured with a calibrated eyepiece.

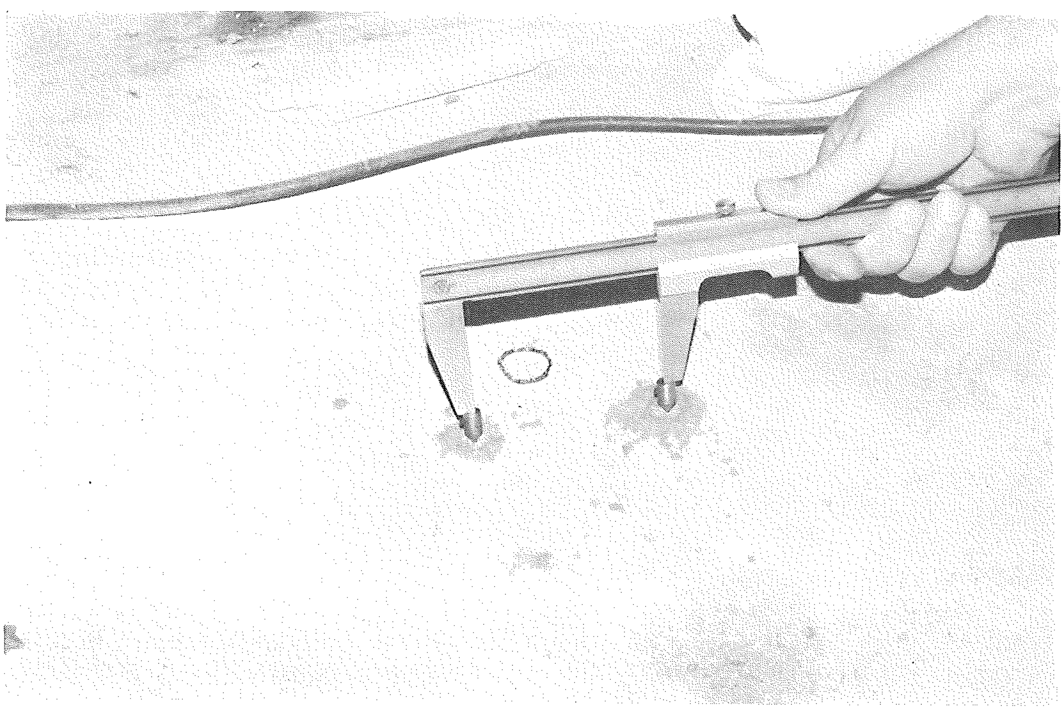


Figure B-16. Measuring segment crack widths with a caliper.

vernier calipers to measure to the nearest 0.001 in. as shown in Figure B17.

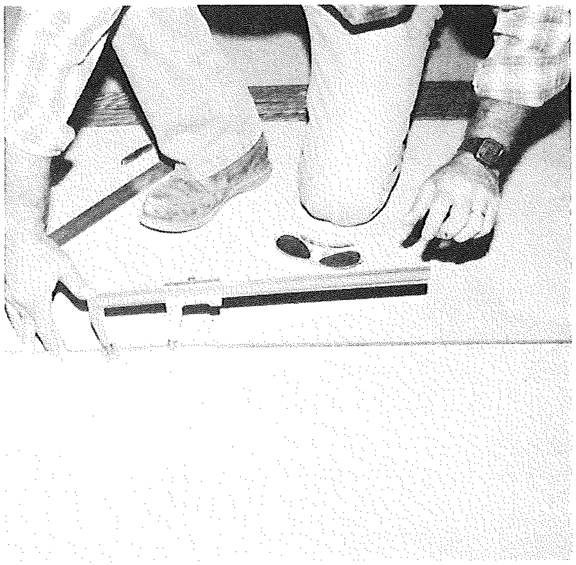


Figure B-17. Total length of cracked sections of segment bottoms were measured with a caliper and an invar tape.

Expansion Joint Movement

Pins were placed on the inside of the bottom of the damaged expansion joint and measured with a vernier caliper to the nearest 0.001 in. as shown in Figure B18.

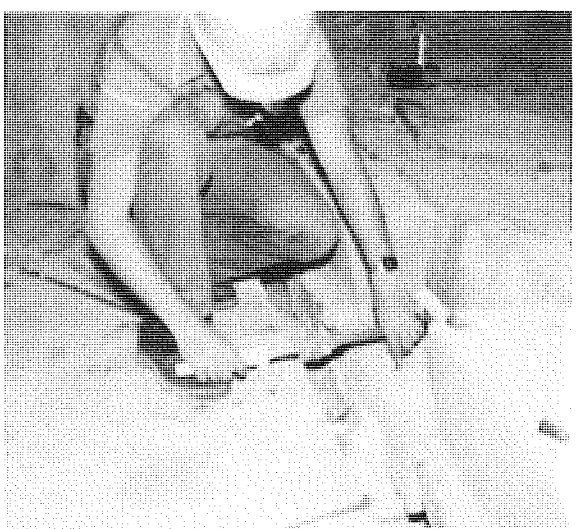


Figure B-18. Measuring opening at the bottom of the damaged joint.

Marks also were placed on the walls of the damaged expansion joint just north of pier 11N, and on the functioning expansion joint just south of pier 14N. A tape measure was used to measure change in the expansion joint to the nearest 1/32 in. Measurements on the east side and the west side of each joint were taken.

Superstructure Deck Profile

Deck profiles were taken with the precise level with the optical micrometer and the invar rod to the nearest 0.0001 ft. One initial deck profile was taken using all 40 points shown in Figure B19. Otherwise a complete deck profile included only points 15 through 40. For the most part, only partial profiles were taken starting at point 15, using the elevation obtained in the initial profile, and measuring the points of primary concern; point 28 just south of the damaged expansion joint and point 40 at the end of the cantilever (Fig. B20).

Vibrations

Vibration measurements were taken throughout most of the entire monitoring period. Seismometers were used to measure particle velocity in inches per second. This information was fed into a digital readout, with a permanent record being kept on a chart recorder. The seismometer and its signal conditioning unit are pictured in Figure B21. This type of equipment measures oscillation; not slower, longer term motion.

Tilt Sensor System

This system is based on the principle that a bubble enclosed and suspended in an electrolytic liquid in a curved tube, similar to a spirit level bubble, will always be bisected by a line straight to the center of the earth. The current carried, varies proportionally as the glass moves around the bubble. The system is capable of detecting angular movements as small as 1/6000th of a degree. This system was utilized in various locations during Phase II and III monitoring.

Extensometers

Extensometers were used only once by Research personnel during monitoring and that was when the superstructure was lifted off the column on January 30, 1984. This device measures relative movement between two points; in this case the column and superstructure. Measurements were recorded to the nearest 0.001 in. with a permanent trace kept on a chart recorder.

Readings on Top of Expansion Joints

Two sets of pins were placed on each of three expansion joints; the damaged expansion joint and the next two immediately to the north. For each joint, a set of pins was placed near the east shoulder and the other near the west shoulder. A set of trammel points was used to monitor movement to the nearest 1/64 in. immediately before and after each rotation step (Fig. B22).

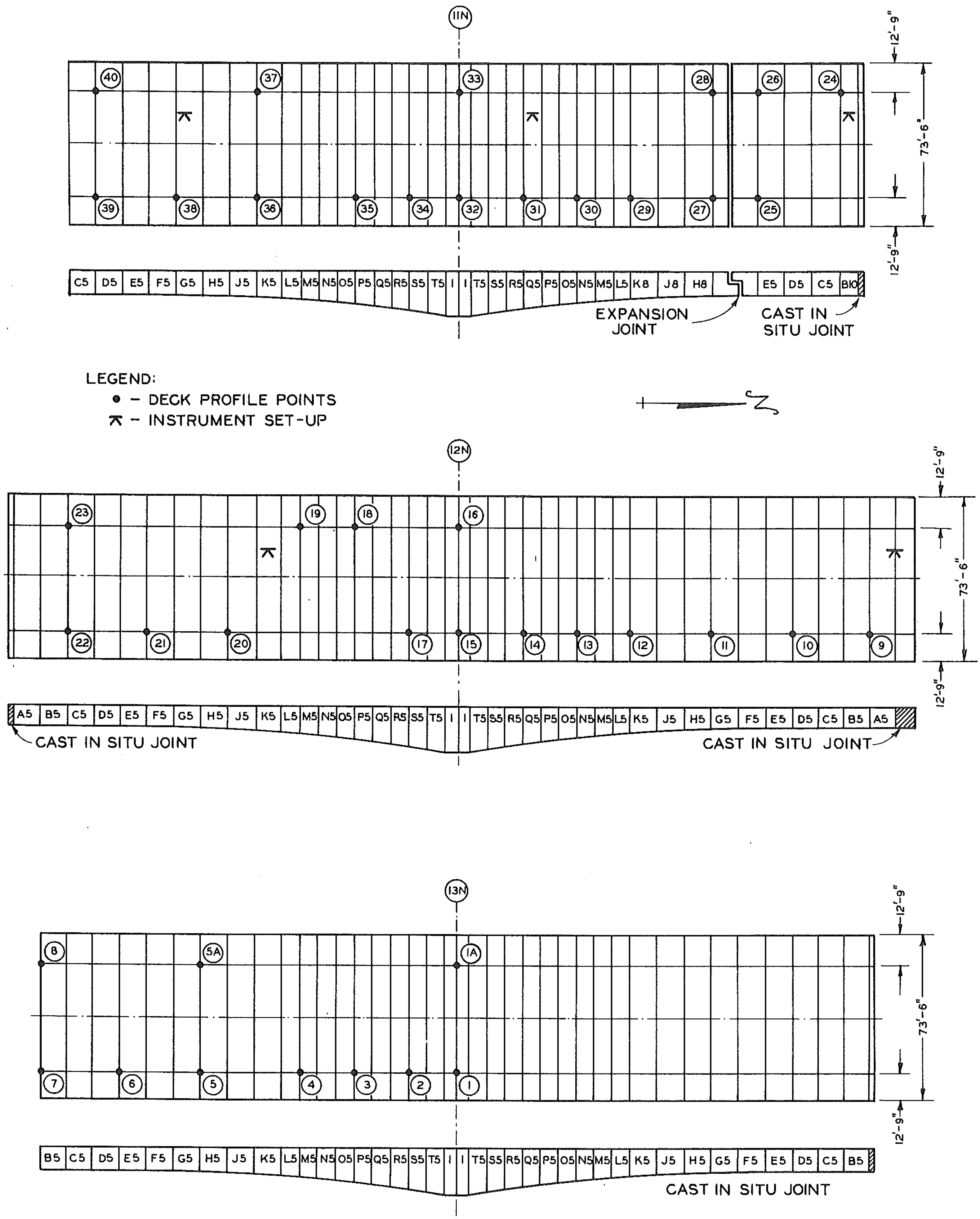
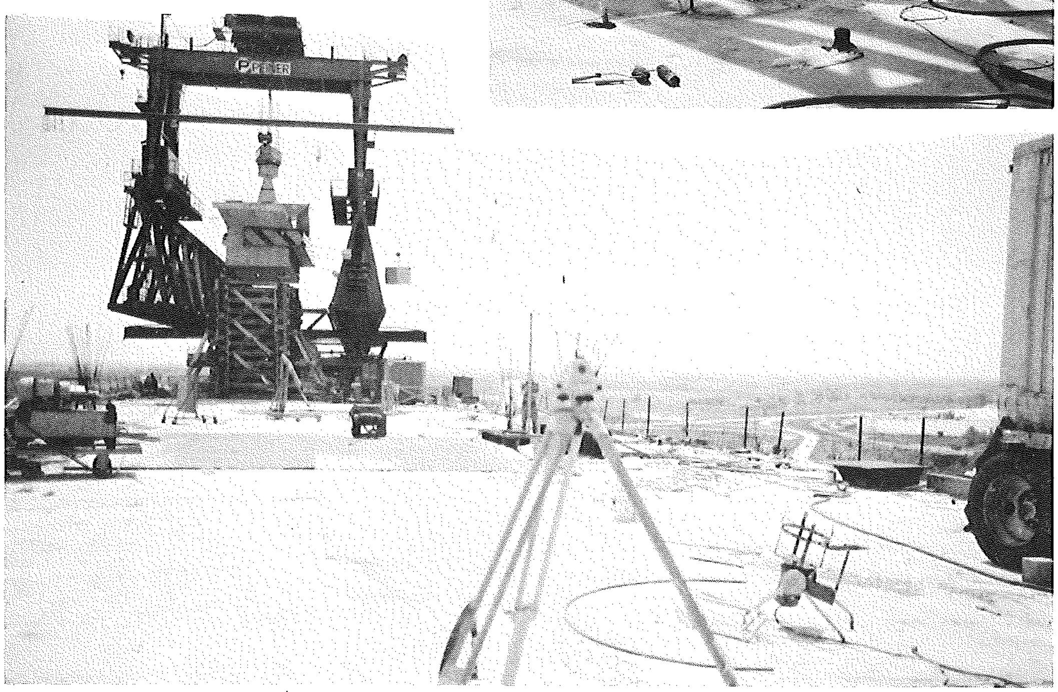


Figure B-19. Location of deck profile points.

Figure B-20. Elevation readings being taken at the end of the cantilever (right) and just south of the damaged expansion joint (below).



Movement of Superstructure

During each of the six rotation steps, the movement of the superstructure north of the damaged expansion joint was a concern. The theodolite was placed over a monument set in the shoulder along I 75 in a location where the superstructure could be seen between piers 13N and 14N (Fig. B23). A nail with inscribed cross hairs in the I 75 median curb was used for a backsight target. A tape for a Philadelphia level rod was mounted on the fascia of the superstructure (Fig. B24). This allowed monitoring to the nearest 0.01 ft. A site diagram is shown in Figure B25.

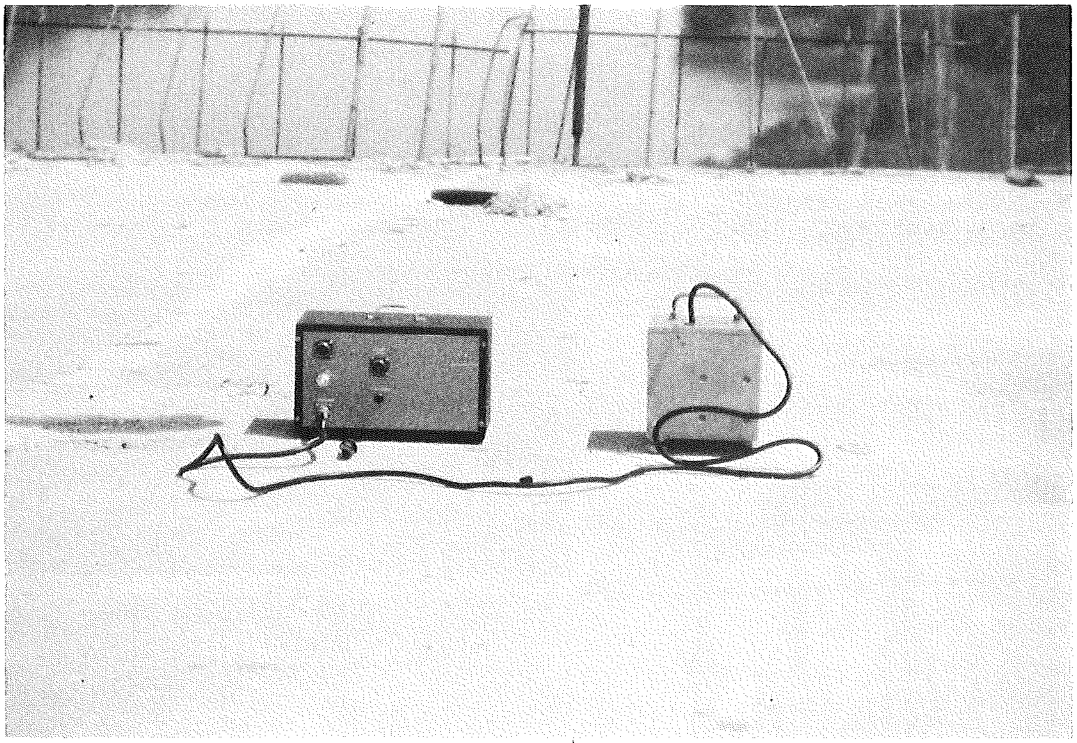


Figure B-21. Vibration seismometer and signal conditioning unit.

Figure B-22. Measuring expansion joint with trammel points.

Figure B-23. A theodolite was used to monitor superstructure movement between piers.

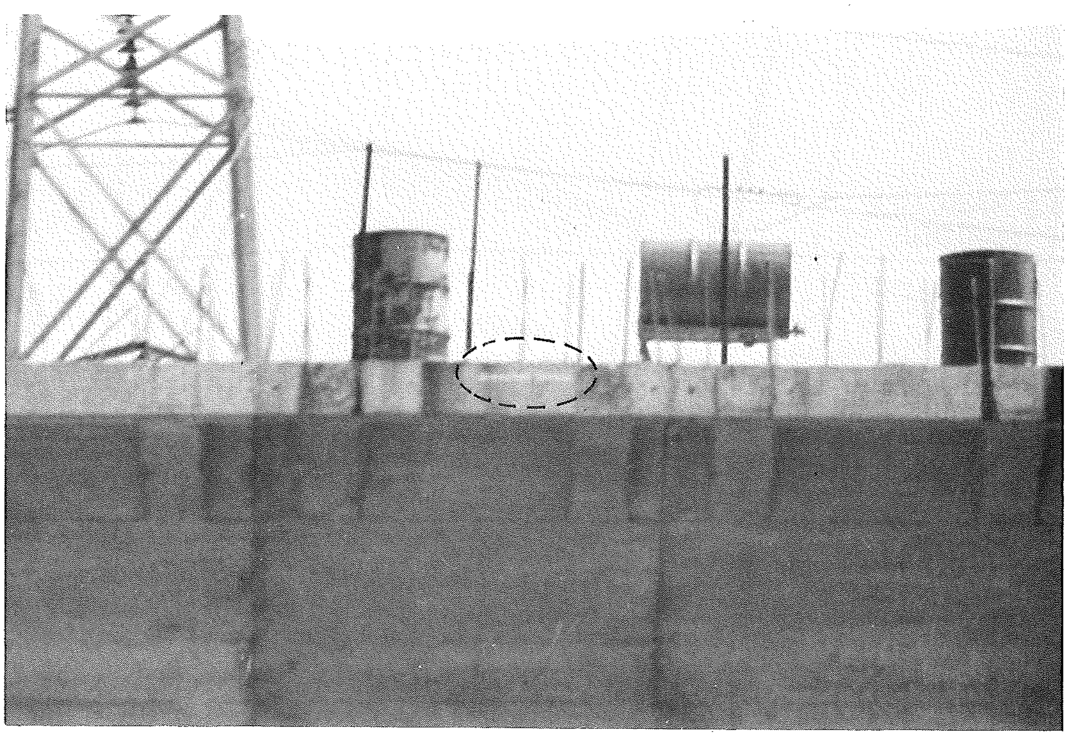


Figure B-24. Philadelphia level rod tape on side of superstructure.

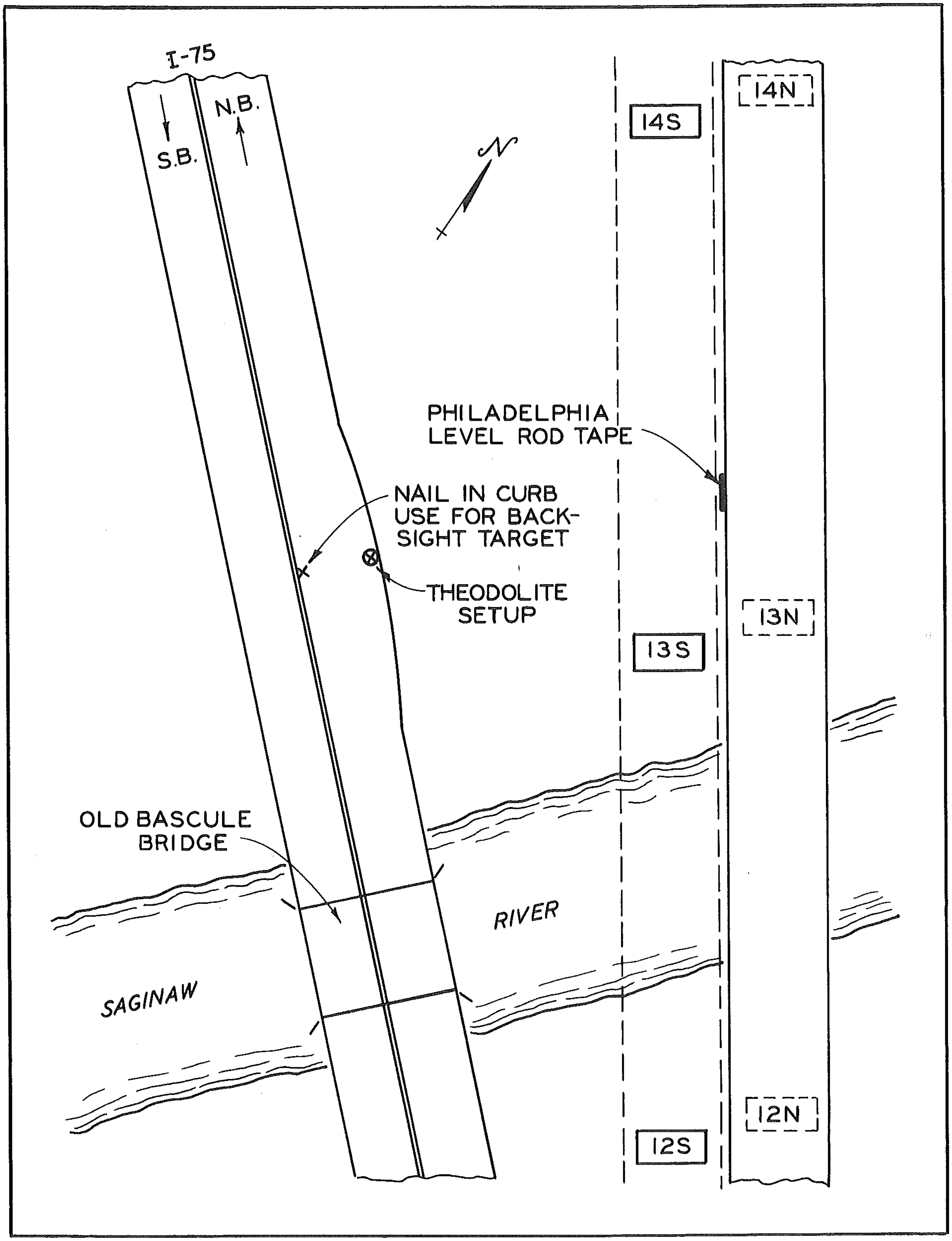


Figure B-25. Site layout and set-up for monitoring superstructure movement.