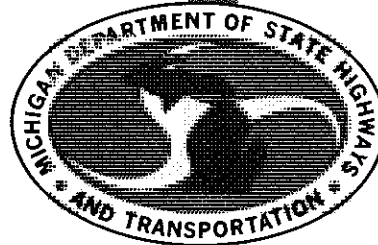


CONCRETE ARCH RIB CONDITION SURVEY, M 99
(LOGAN ST) OVER THE GTW RR AND THE
GRAND RIVER, CITY OF LANSING (X01 of 33011)



**TESTING AND RESEARCH DIVISION
RESEARCH LABORATORY SECTION**

CONCRETE ARCH RIB CONDITION SURVEY, M 99
(LOGAN ST) OVER THE GTW RR AND THE
GRAND RIVER, CITY OF LANSING (X01 of 33011)

H. L. Patterson

Research Laboratory Section
Testing and Research Division
Research Project 76 TI-359
Research Report No. R-1039

Michigan State Highway Commission
Peter B. Fletcher, Chairman; Carl V. Pellonpaa,
Vice-Chairman, Hannes Meyers, Jr., Weston E. Vivian
John P. Woodford, Director
Lansing, January 1977

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INTRODUCTION

A letter of July 6, 1976 from W. J. MacCreery to K. A. Allemeier initiated the exploratory work covered in this report on the concrete arch river spans of the Logan St structure. This letter stated that the current bridge was constructed about 1930, and it explained that the recommended scheme for its reconstruction called for the utilization of the existing arch ribs in the three south spans should their condition be satisfactory. The letter requested that the Testing and Research Division evaluate the condition of the arch span concrete and make an estimate of its approximate life expectancy under present loading conditions. Accompanying the letter was a half-size set of the original plans. The letter and plans were sent to the Research Laboratory Section where the requested evaluation was assigned to the Concrete and Surface Treatment Unit.

INVESTIGATION WORK

Proposed Investigation Procedure

After contacting A. L. Jones and his Bridge Design Squad, Research Laboratory personnel decided to employ basically the same investigative procedures as were used on the M 95 Menominee River bridge¹. This consisted of a Swiss Hammer survey that was equated to compressive strength values through correlation with 4 by 8-in. field cores. A basic difference between the two bridges existed in that the four arch ribs in each span of this bridge are much more massive than the twin ribs and spandrel wall that supported the M 95 bridge. For this reason, it was felt the surface hardness as measured by the Swiss Hammer might not be as representative of the actual interior strength of these arch ribs. Design personnel pointed out that the arch ribs were probably overdesigned and for that reason it would be permissible to drill cores from the rib itself.

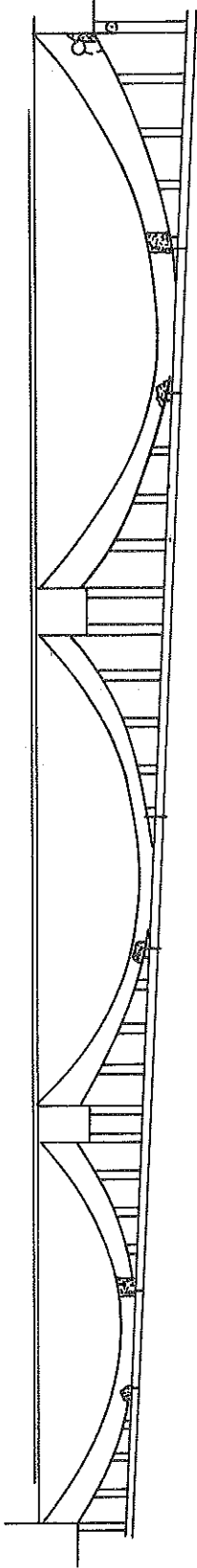
Arrangements were made with the Maintenance Division for the use of their 'Snooper' type machine that was designed for this type of work. Since a limited time was available for this project due to the demands of four other active projects, we hoped to accomplish the field work in seven to ten days, beginning in late August. The proposed work schedule was coordinated with the Lansing Public Service Department in a letter from G. J. McCarthy to Robert Backus dated August 20, 1976.

¹ "Determination of Concrete Strength in the Arch Ribs of Bridge B01 of 22011," MDSHT Research Report R-759, February 1971.

AREAS HEAVILY DAMAGED BY
FREEZE-THAW DISINTEGRATION

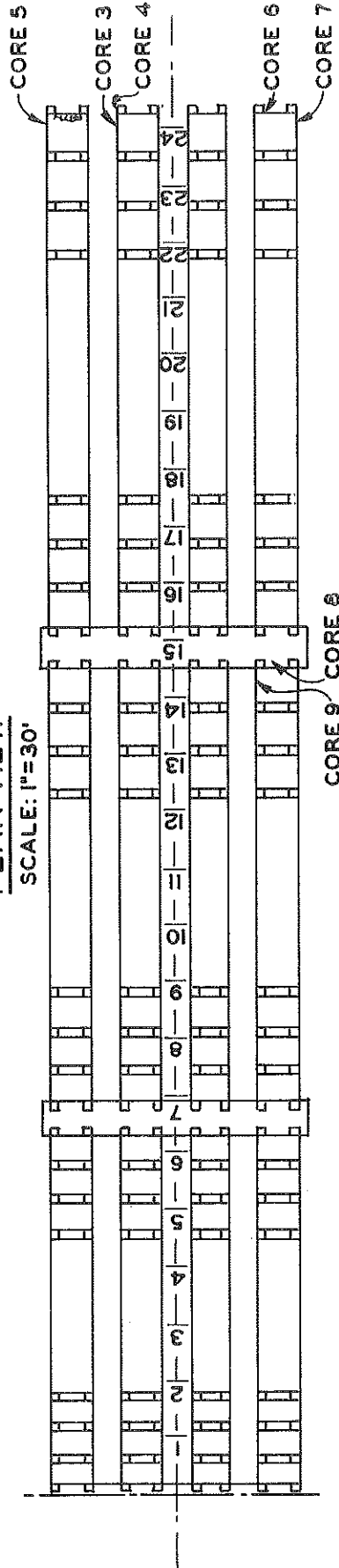


WEST ELEVATION



PLAN VIEW

SCALE: 1"=30'



CONST. JOINT
LOCATIONS (TYR)

EAST ELEVATION

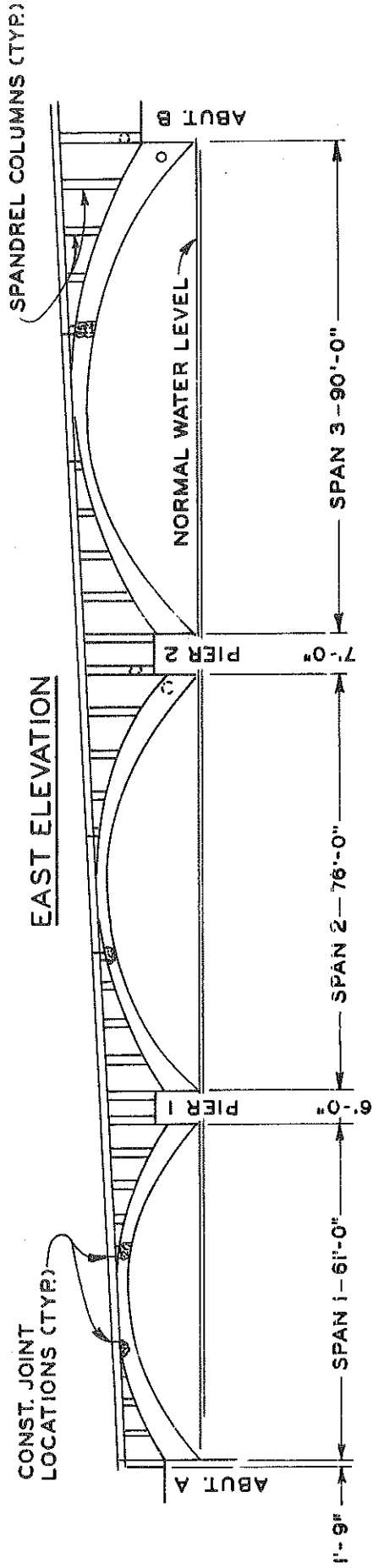


Figure 1. Diagram of arch ribs X01 of 33011, M 99 (Logan St over GIWRR and the Grand River).

Equipment Problems

Numerous mechanical problems developed with the Snooper equipment to delay field operations. Three different days, when arrangements had been made to work from the Snooper's platform, control failure impeded progress on two of the days, and a mechanical failure prevented work on the third day. On two other occasions work was impeded by arrow-bar trailer problems. With a tight workschedule these problems were especially frustrating since they severely curtailed the scope of the investigation before the onset of bad weather.

Extent of Work Accomplished

Snooper Platform Work - On September 1, investigative work progressed on the west side of Span 3. Swiss Hammer readings to measure concrete hardness were taken at various locations; deterioration features were closely examined and photographed before and after hammer probes; and coring locations were laid-out to avoid steel reinforcing bars. The sides and bottom of both the fascia and interior ribs were examined. On September 10, work resumed on the east side. The crown area of Span 3 and part of Span 2 were completed before equipment failure again stopped work.

Boat Work - Access by boat to Piers 1 and 2, and shoreline access to Abutments A and B, permitted work on the top surfaces of the lower portion of the arches. The boat also permitted a fairly good view of the crown areas and a close inspection of the waterline areas of both abutments and piers. The depth of these latter inspections was limited to about one foot below the water level.

Coring Work - Since neither the Snooper nor traffic control arrangements were required for work at the north abutment, five cores were cut from this easily accessible location. Two other cores were cut from the east side of Pier 2 which was reached by boat.

INVESTIGATION RESULTS

Locations of Distress

Mid-Span Construction Joints - As shown on the original design, each span of this bridge had transverse construction joints through the deck that were located above either end of the arch's crown section. The locations of these joints are shown in the elevation views of the diagram in Figure 1.



Figure 2 (above). View of header wall of west fascia rib at north end of Span 3. Note weep holes through header and salt stains down inside face of rib caused by drainage from deck construction joint.

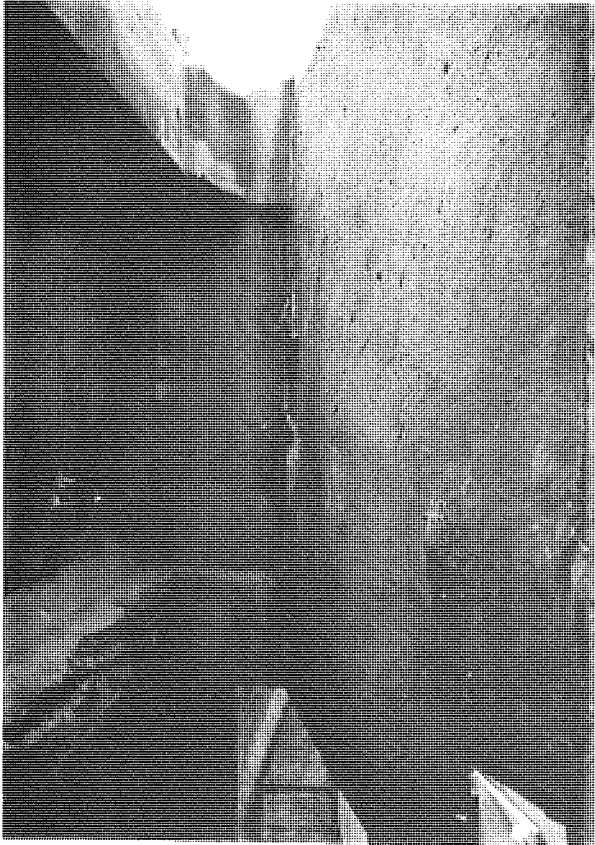


Figure 3 (right). General area and close-up views of header wall of west fascia rib in north half of Span 3. Note salt discharge residue that plugged weep holes and left residue line along entire length of construction joint between rib surface and header wall.

These joints were located immediately inside of a header wall that formed a bulkhead between the lower surface of the deck and the upper surface of the arch rib (Fig. 2). Between these headers, the deck was supported directly on the top surface of the arch rib. Outside the headers the deck was supported by regularly spaced spandrel columns which rose from the arch rib. To prevent a build-up of construction joint seepage water from forming behind the concrete header, weep holes were provided that consisted of galvanized iron pipes cast into the concrete. As could be anticipated, the most severe damage to the arch ribs occurred at the construction joint locations that permitted the heaviest seepage of chloride-laden surface water from the deck above. These locations were beneath the curb line having the longest distance of surface water collection. Figure 3 shows general area and close-up views of these concrete header weep holes. They carried so much salt water leakage over the years that they eventually plugged-up with salt. In addition to flowing out the weep holes, the trapped water behind the header also escaped through the edges of the construction joint where it ran over and down the side of the arch. The exposure of this non-air-entrained concrete to salt water and many years of repeated freeze-thaw cycles produced serious damage to the arch rib. Figure 4 shows the condition before and after removal of the distressed concrete with a rock hammer. The removal work revealed that the expansion that accompanies the disintegration of concrete produced horizontal stratification planes throughout the distressed area. This rarely observed phenomenon existed because the compressive forces in the arch prevented this characteristic expansion from occurring in all directions. Hammer blows at the limits of concrete removal shown in Figure 4 produced hollow sounds that suggested the limits of disintegration went much deeper. In addition to escaping from the weep holes and the side of the construction joint, it was also noted that some of the trapped water had seeped through the construction joint between the arch rib and the header and left a line of salt residue which can be seen in Figure 3. Figure 2 shows the arch rib below the deck construction joint on the inside of the rib. The view shows that trapped water has also run out and down this surface, but it does not appear to have produced the distress such as that observed on the side exposed to the weather. Collectively, all of this evidence would suggest that the full width of this arch rib beneath the deck construction joint would be distressed to some varying, but unknown depth.

All of the photographs showing this problem (Figs. 2 through 4) are of the west fascia rib at the north half of Span 3. They are also typical of the fascia ribs at several deck construction joint locations.

Figure 5 shows weep holes over the west interior rib at the north half of Span 3. Their excellent condition and the absence of salt deposits indi-

Figure 4 (left). Views of distressed concrete before and after removal to check depth of disintegration. Location is at west fascia rib at north header wall in Span 3. Note salt deposit build-up in "before" picture. Depth of removal in "after" picture still failed to reach solid concrete.

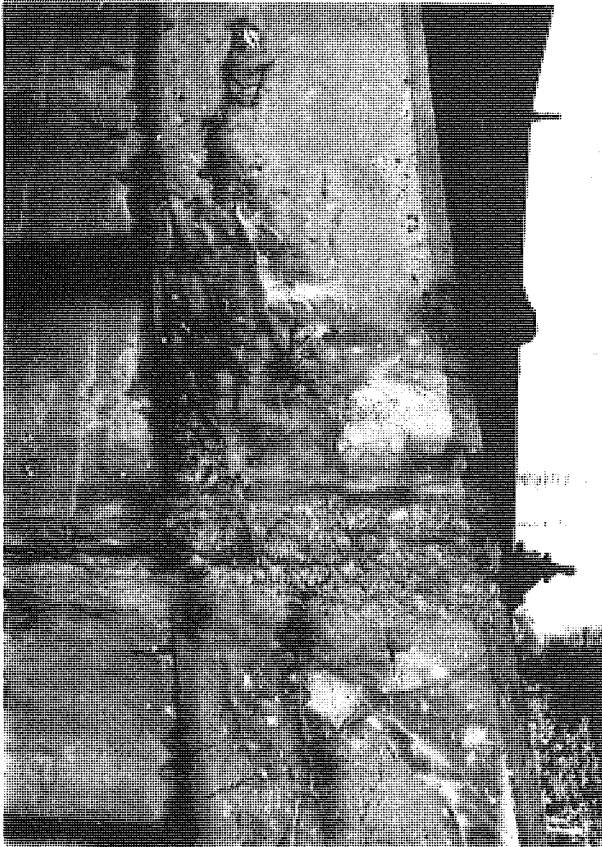
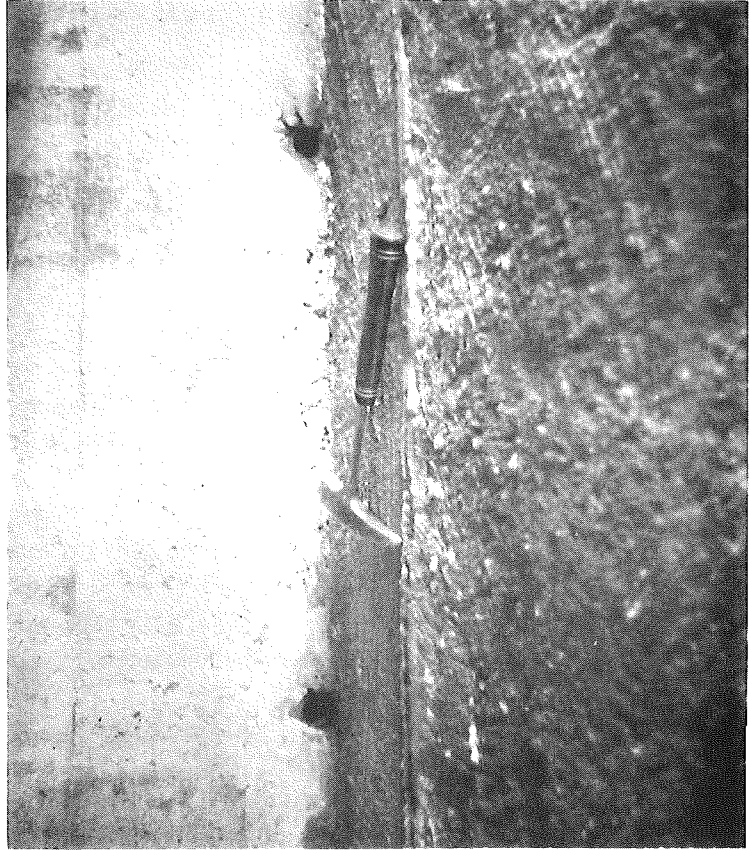
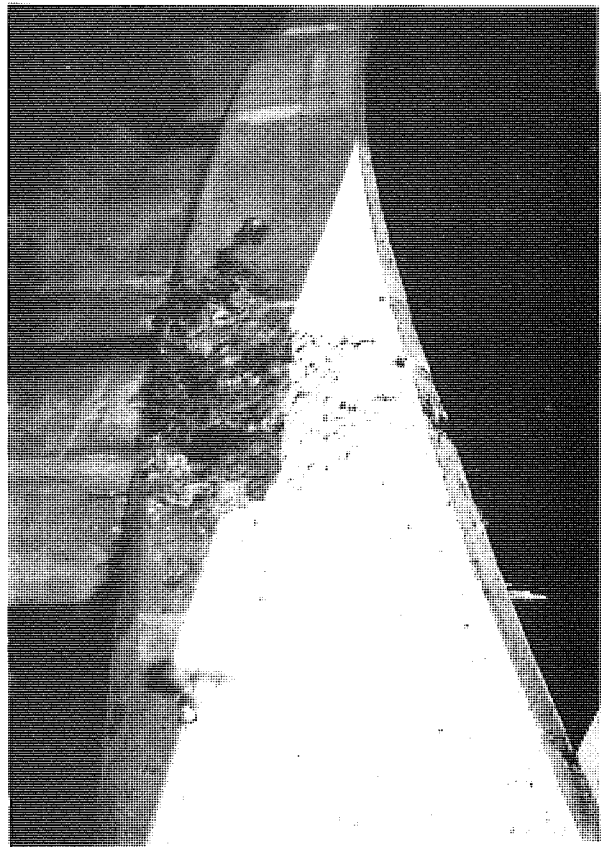


Figure 5 (below). Close-up view of weep holes through header wall at west interior rib in north half of Span 3. The lack of a salt deposit build-up indicates that very little surface water seeped through the construction joint at this location.



cate that little water seeped through at their location. It would appear that all six interior ribs are free from the serious type of damage sustained by the six fascia ribs in the three arch rib spans.

Consolidation Voids - Concrete consolidation, which was accomplished without benefit of modern probe vibrators, apparently was a problem when the bridge was constructed. Mortar patching and honeycombed voids were very numerous in the arch ribs, but not in the substructure units; hence, their frequency was directly related to reinforcing bar concentration.

Arch Ribs - As would be expected, the lower corners of the rib cross-sections were the most common location of mortar patches over consolidation voids. This was especially true at those locations having reinforcing bar laps. Typically, these mortar patches failed to provide adequate protection for the reinforcing steel. The resulting rust expansion loosened the patch and accelerated disintegration. Figure 6 shows a view of one of these areas before and after the patch was removed to determine the limit of distressed concrete.

Some of this edge spalling was observed beneath the column pedestal which is immediately down the slope of the top arch surface from the weep holes. Here discharged surface water and salt brine ran down the side of the arch. Figure 7 shows edge spalling which started at a patch over a splice and started the beginning of general delamination at the lower bar level.

Figure 8 shows a large void area at the bottom of the west fascia of Span 3. The attempt to patch this void was apparently unsuccessful.

Substructure Units - The bridge is located in close proximity to the Eckert Power Plant, which uses river water to condense exhaust steam returning from the turbines. Hence, freeze-thaw deterioration would be minimal at the waterline of the substructure units. A close inspection to a foot below water level revealed only one blemish; a honeycombed void at the southeast corner of Pier 2, that was produced by poor consolidation (Fig. 9). The only freeze-thaw deterioration observed was to a decorative panel on the east end of Pier 1. Deck water, leaking through an expansion joint, dripped onto this concrete and caused moderate spalling.

Except for these two superficial blemishes, the condition of the substructure units, other than the arch ribs, was found to be excellent.

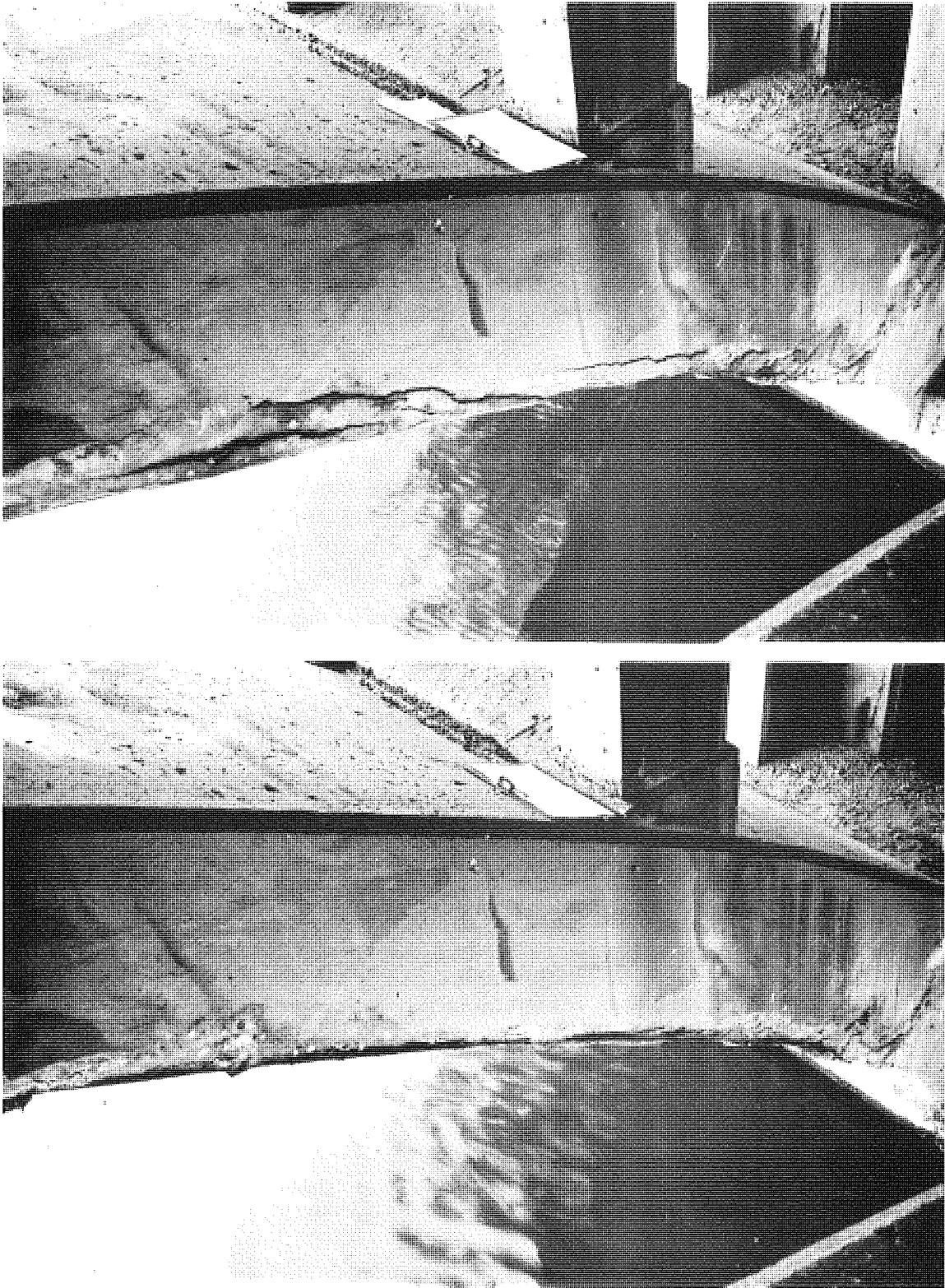


Figure 6. Before and after removal views of distressed concrete at lower edge of rib. View shows east side of west fascia rib in north half of Span 3. Some patching was placed where reinforcing bars made original placement and consolidation difficult.

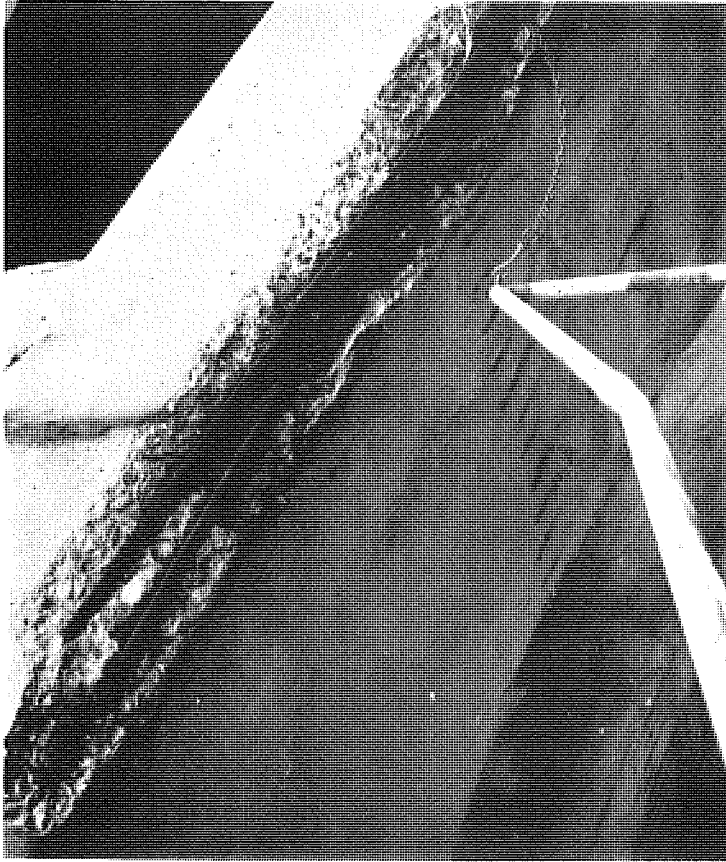


Figure 7. Before and after removal views of distressed lower arch rib corners in vicinity of bar splice. Note weep hole discharge stains down side of rib below top pedestal. Close-up view at left shows the delamination existing along line of interior bars.

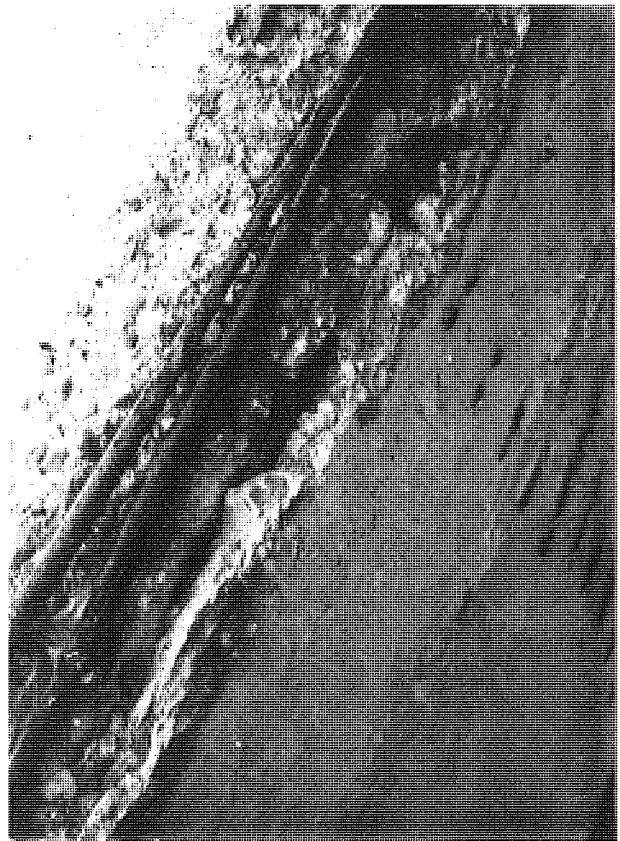


Figure 8 (left). View of an apparent honeycombed area caused by placement difficulties during construction. Located at south end of Span 3 on west fascia rib.

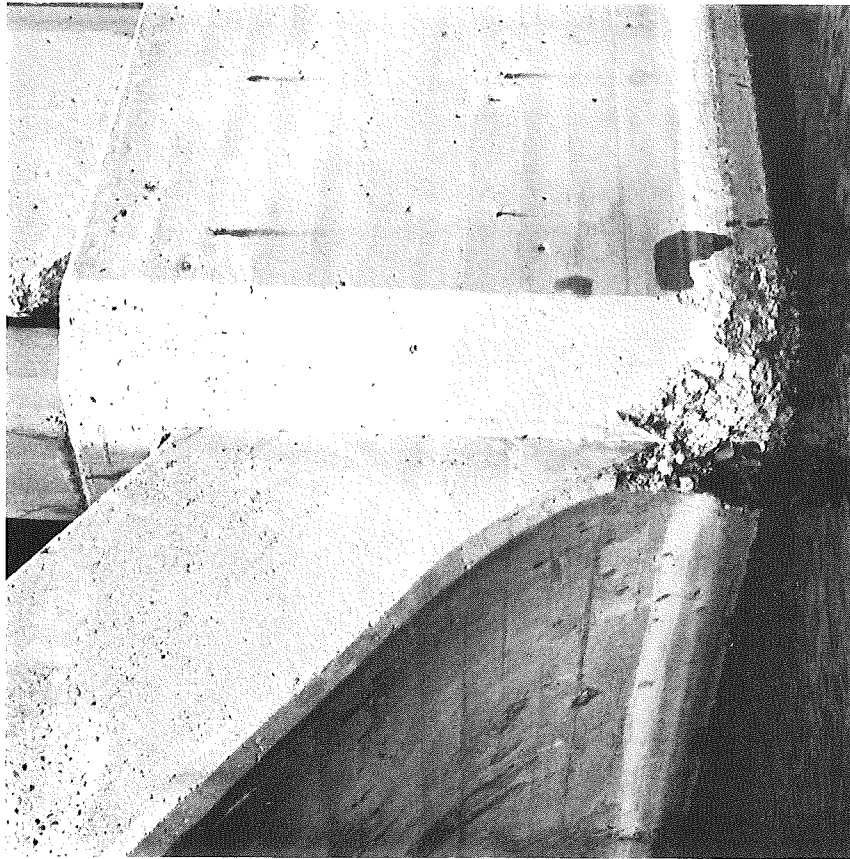
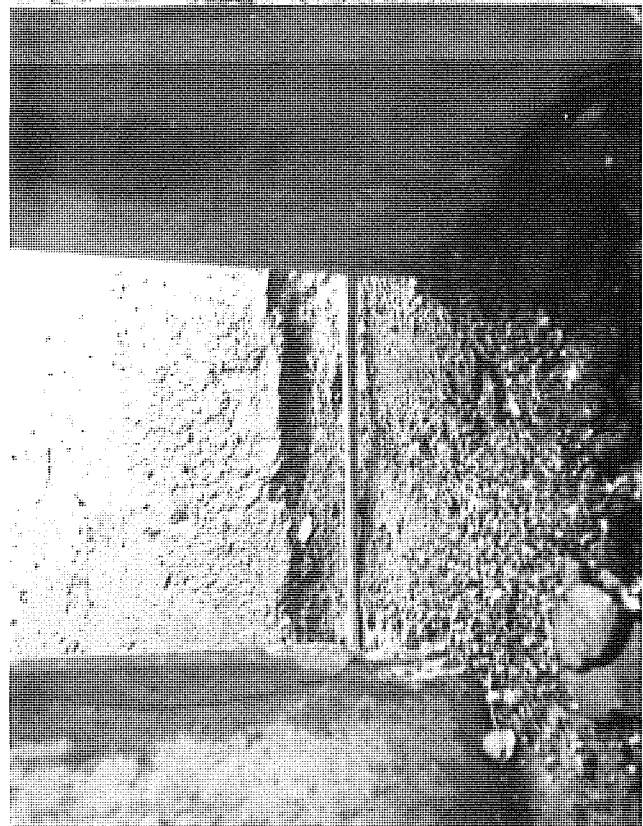
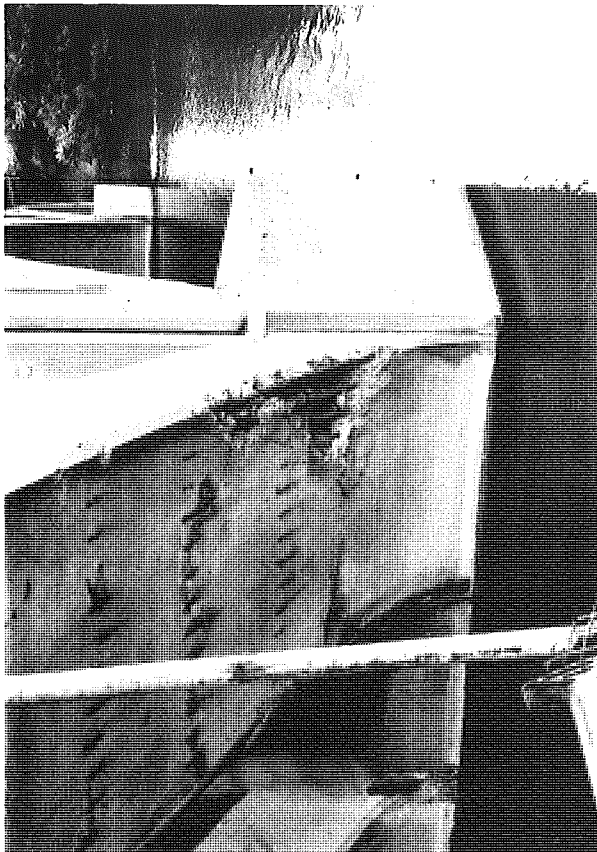


Figure 9. View of honeycombed area at the southeast corner of the north pier. This is the only blemish around the entire pier.

Figure 10 (left). View of deteriorated area at north abutment and west fascia rib prior to removal of dirt and rubble.



Rib Spalling at Abutment B - Serious deterioration was noted on the west fascia rib at the north abutment (Pier 3) but the general appearance was somewhat deceptive. Dirt accumulations, which are common in inverted slope areas of old bridges, covered the area where the top surface of the rib met the abutment (Fig. 10). The interior side of the rib displayed a large patch filling an apparent honeycombed void, and the area around the patch showed evidence of freeze-thaw deterioration. The fascia side of the rib exhibited heavy spalling near the top and less advanced disintegration over the lower portion. Three feet north of this area, heavy leakage occurred through the expansion joint where the arch supported portion of the deck met the portion supported by bent girder piers. Freeze-thaw damage to one of the columns of the first pier in this area was so severe that approximately 50 percent of the cross-section was distressed. It is conceivable that with a wind in the right direction, much of the heavy surface leakage from the west curb line could blow onto the base of the west fascia arch rib. Only this situation could have produced the severe distress to the interior and fascia sides of the rib.

Proceeding on this assumption, the dirt at the base of the rib was removed to sound concrete that was approximately 6 in. below the finished surface. All of the top reinforcing bars were exposed. The type of rubble removed appeared to be a combination of deteriorated concrete plus loose soil. Figure 11 shows a general area and close-up view of the void after the soil and distressed concrete were removed to a sound surface. Figure 12 shows a general area and close-up view of the interior side, while Figure 13 shows the fascia side before and after Core No. 5 was removed. The evidence of deterioration here suggests that the moisture trapped in the top cavity gradually permeated through the concrete at the sides of the rib and eventually debilitated it. Interior consolidation voids could also have contributed to the problem. The core indicated that the concrete was distressed to a depth of about 3 in. Figure 14 is a diagram of this area that shows the estimated limits of the distressed concrete.

Concrete Cores

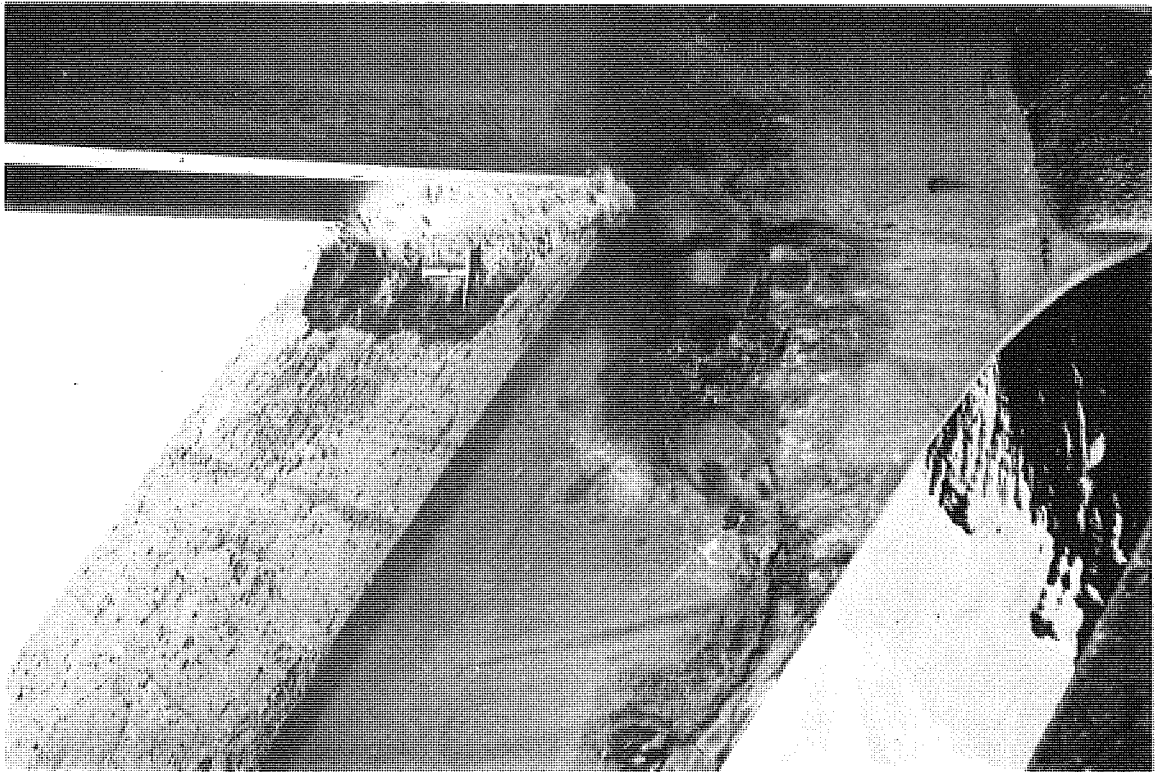
While Swiss Hammer work was being conducted from the platform of the Snooper and from the arch ribs, core locations were laid-out with a pachometer to avoid any reinforcing bars. Swiss Hammer readings were then taken at these locations to provide values for subsequent correlation work between the Hammer readings and the core strengths. A total of 12 core locations were laid out. Five could be drilled from the easily accessible vicinity of the north abutment, and seven from the more difficult pier locations. Of the 12, all five of the north abutment cores were drilled but equipment problems permitted drilling only two of the seven pier cores.



Figure 11. Overall and close-up views of spalled area at base of west fascia rib at north abutment.



Figure 12. General area and close-up views of east side of west fascia rib at north abutment. Note severity of disintegration on side of rib. Presence of patch indicates that consolidation problems were encountered.



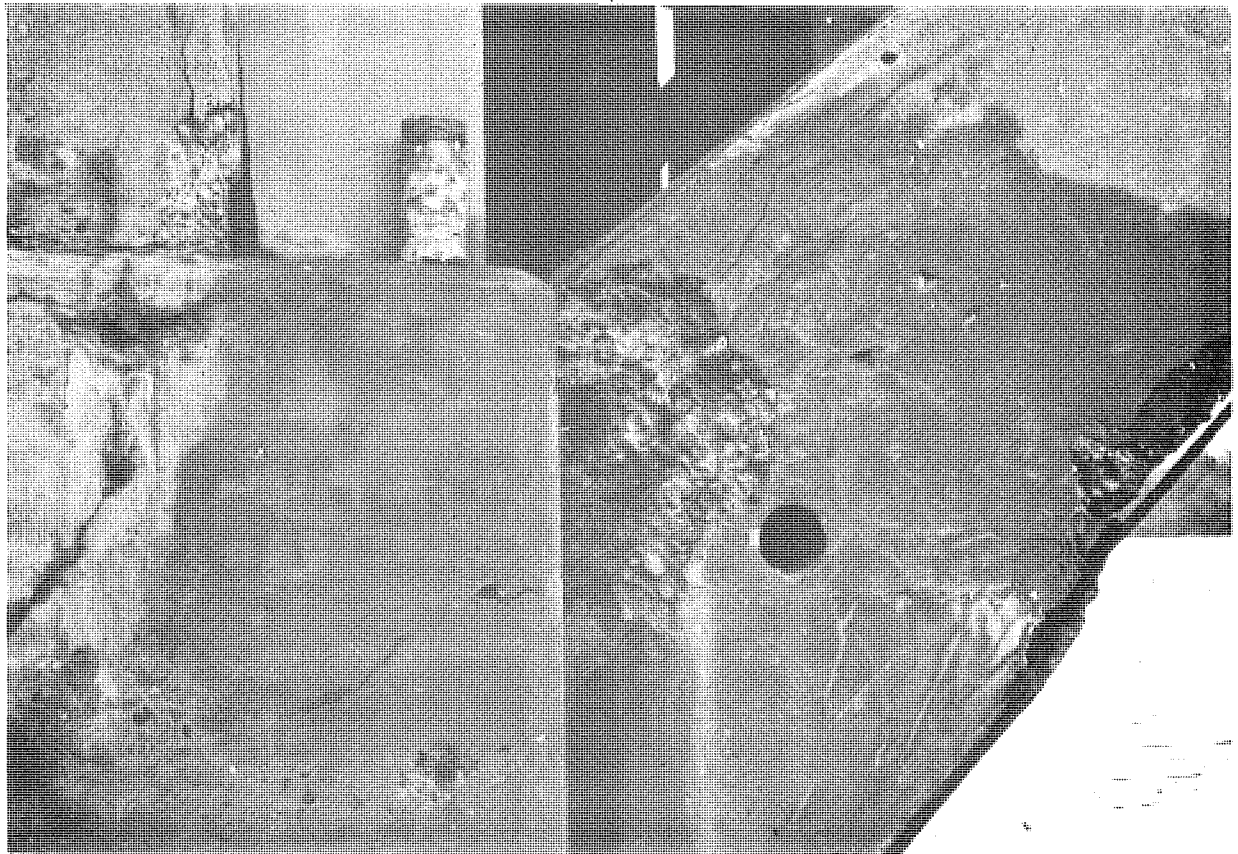
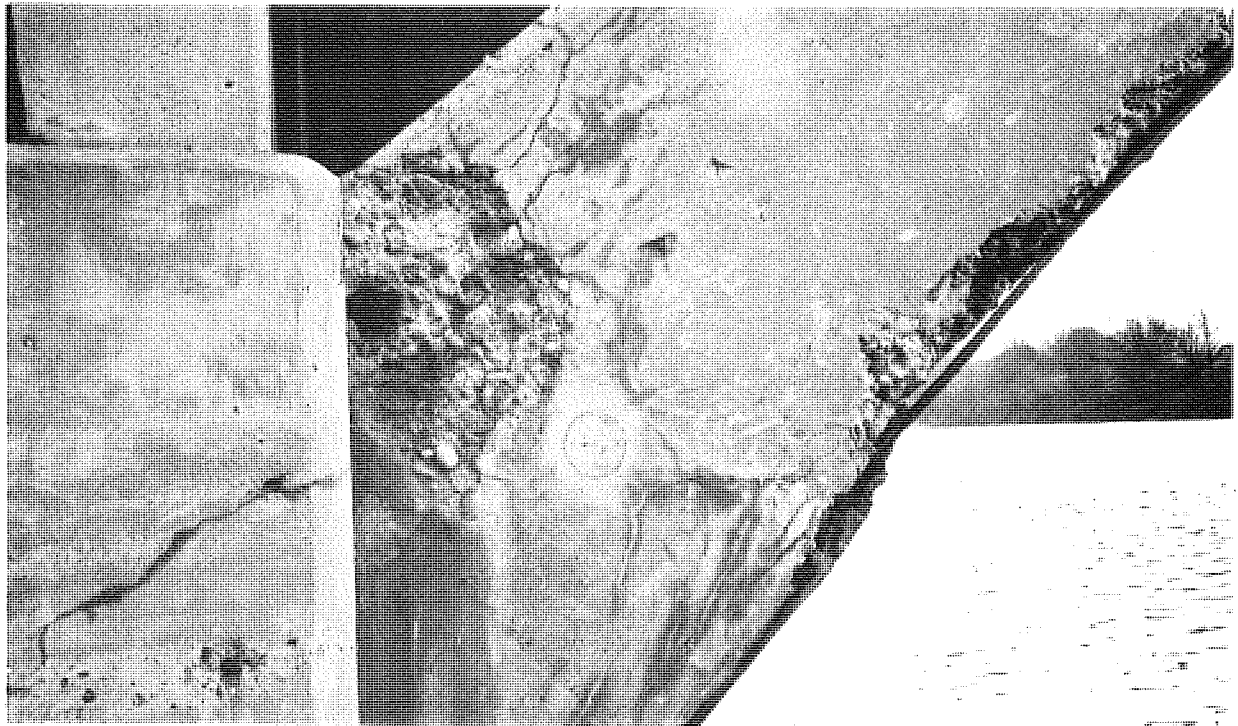


Figure 13. Views before and after Core No. 5 was drilled on west side of west fascia rib at north abutment. Note severity of concrete disintegration.

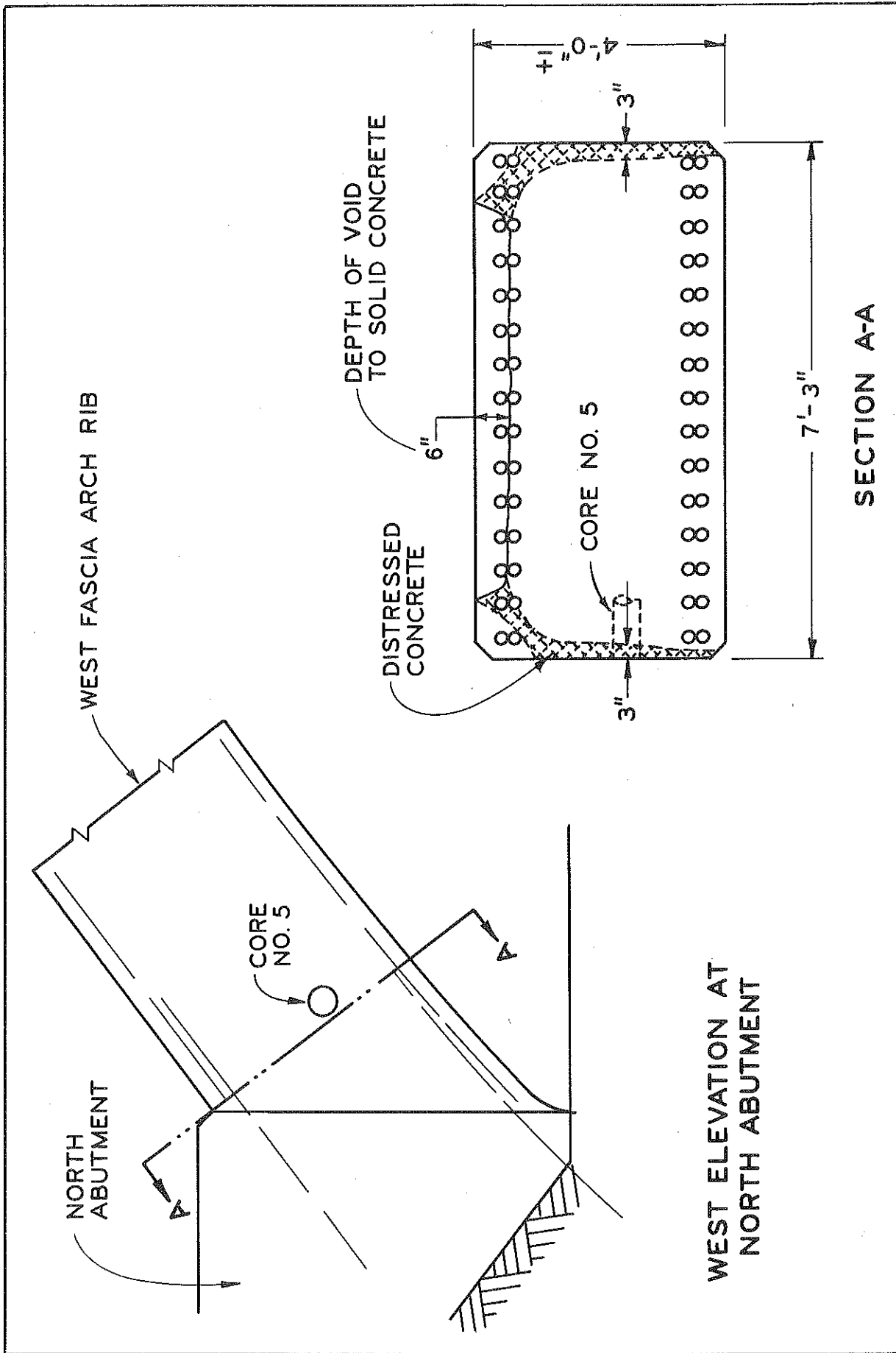
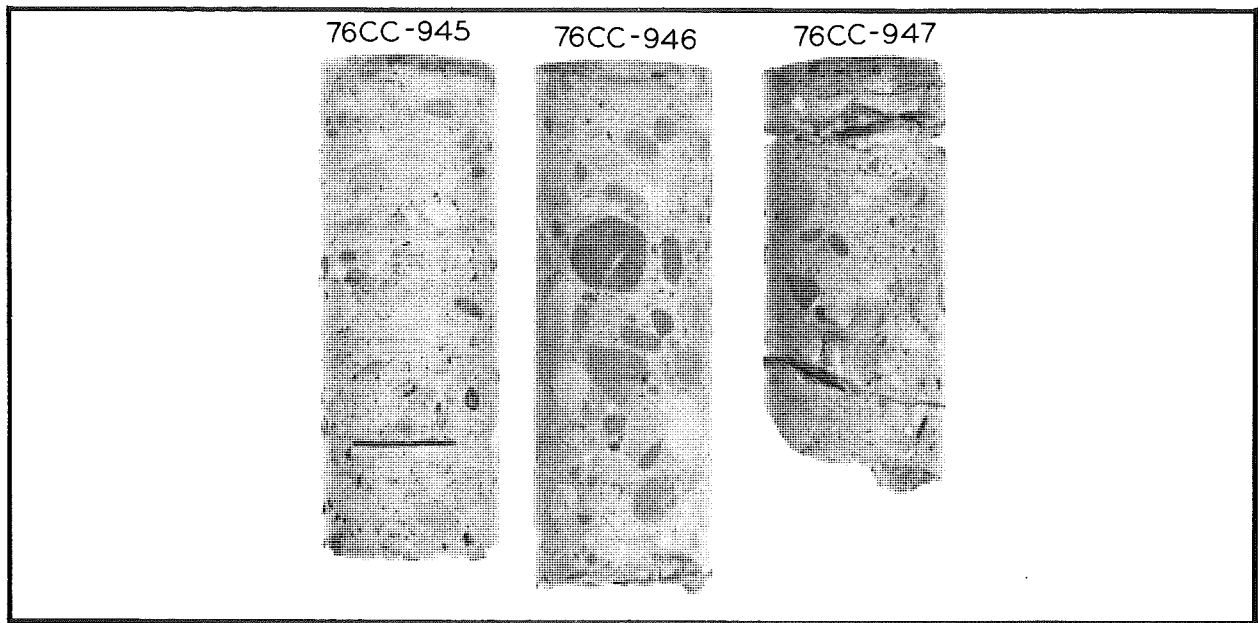
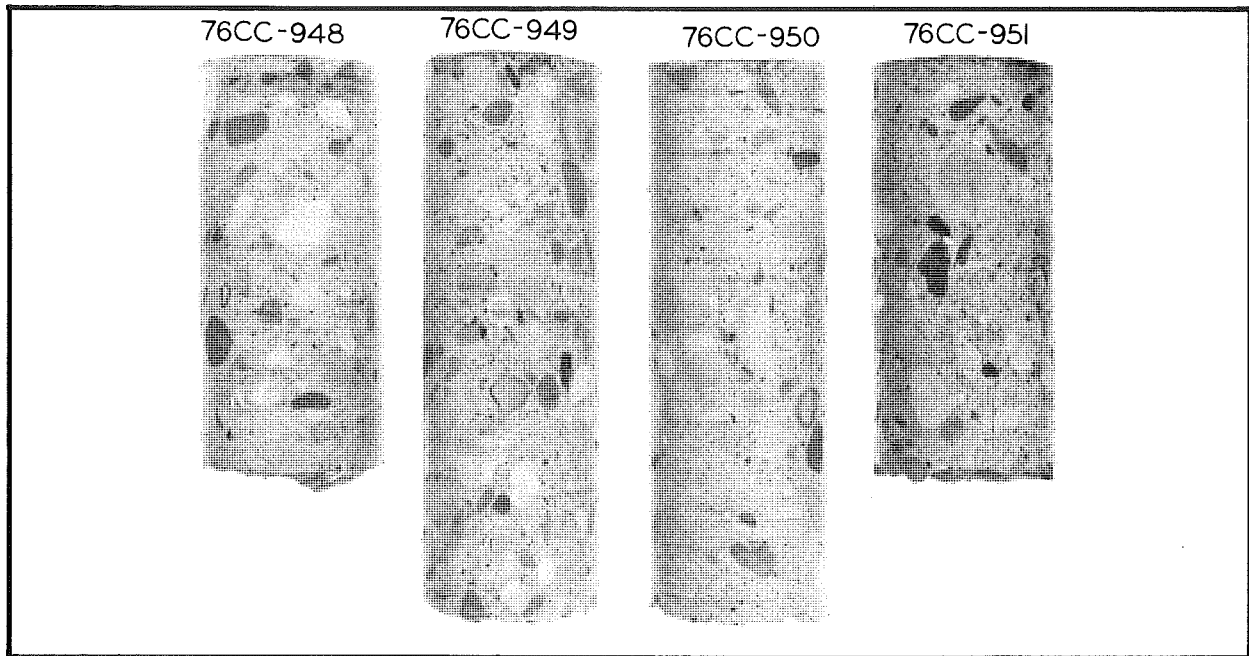


Figure 14. Estimated limits of distressed concrete on W fascia arch at N abutment.



Core No. 3 - 76 CC-945 - W vertical face; W inside rib; N end Span 3
 Core No. 4 - 76 CC-946 - W face of W column; W inside rib; N end Span 3
 Core No. 5 - 76 CC-947 - W vertical face; W fascia rib; N end Span 3



Core No. 6 - 76 CC-948 - E face of W column; E fascia rib; N end Span 3
 Core No. 7 - 76 CC-949 - E vertical face; E fascia rib; N end Span 3
 Core No. 8 - 76 CC-950 - E face of W column; E fascia rib; N end Span 2
 Core No. 9 - 76 CC-951 - W vertical face; E fascia rib; N end Span 2

Figure 15. Close-up view of cores and location where they were drilled. Cores 3 through 7 (above) taken from north end of Span 3.

Figure 15 shows photographs of the seven recovered cores. Table 1 gives the physical properties of these cores. Included in the table are the Swiss Hammer readings taken at the locations before the cores were drilled. A comparison of the Swiss Hammer readings with the core strengths shows no apparent correlation; obviously, a single series at the core location is not representative or reliable in mass concrete.

Of the recovered cores, only 76 CC-947 (Core No. 5) was cut in a distressed area. Its sound portion was too short for testing as difficulty was encountered in recovering the full length in one piece. The most interesting aspect of this core is the fact that it produced a Swiss Hammer reading of 52 with a delamination at a depth of only 1.25 in. This emphasizes the limitation of the Swiss Hammer; that it records only surface hardness and will not give any indication of the quality of the concrete beyond the limits of surface hardness influence. Hence, unless the quality of the concrete is very uniform, it is impossible to assess the interior quality of a massive concrete structure by taking Swiss Hammer readings on the outside surface.

The recovered lengths of Cores 76 CC-945, 76 CC-946, and 76 CC-948 through CC-950, were long enough to supply core ends in addition to the 8-in. compression segment. These were used for specific gravity and permeability measurements. (Due to a reduced section caused by a drill reset, only a 6-in. segment from 76 CC-948 was suitable for compression.)

Following testing, the compression segments were crushed down so that the coarse aggregate could be closely inspected. It appeared to be a good grade of gravel of about 2-in. maximum sieve size. The composition was noted to be about 50 percent igneous, 45 percent sedimentary (mostly limestone and dolomite) and 5 percent metamorphic.

Specific gravity and permeability values for those cores having sufficient length are also shown in Table 1. These values indicate that this concrete is comparable to 6-sack, low slump, air-entrained, slip-form barrier concrete in both specific gravity and permeable voids.

Figure 16 shows the locations where Cores No. 3, 4, and 9 were taken. At the location of Core No. 9, note the smooth surface prepared with a carborundum block, and also note the Swiss Hammer 7-point pattern. The average of the values measured at these points constitutes the 'reading' at that location, and shown in Table 1.

TABLE 1
CONCRETE CORE DATA
Logan Street Arch Ribs

Laboratory No.	Core No.	Measured Diameter, in.	Length, in.	Dry Bulk Specific Gravity		Permeable Voids, percent		Compressive Strength, psi	Swiss Hammer Reading (horizontal)	Maximum Void Size, in.		Core Surface Appearance			Comments
				Surface	Interior	Surface	Interior			TAV ²	BAV ²	No. of Voids > 0.05 sq in.	TAV 1/4-in. diam.	BAV 1/8 x 3/8 in.	
76 CC-945	3	3-31/32	10-1/2	--	2.27	--	13.1	7,440	50	1/4	1/4 x 1/2	1	2	Core appears to be very well consolidated.	
76 CC-946	4	3-31/32	10-3/4	2.39	2.37	11.1	11.5	6,140	50	3/16	1/8 x 1/4	0	0	Core appears to be very well consolidated.	
76 CC-947	5	3-31/32	8-1/4	2.25	2.22	15.1	15.5	---	52	3/16	1/4 x 1/2	0	1	Core broken into 3 segments, top 1-1/4 in. (delaminated) center 5-1/2 in., bottom 1-1/4 in. Measured depth of distressing F-T delaminations: 2-1/2	
76 CC-948	6	3-31/32	8-1/2	2.34	--	12.5	--	7,500	52	1/8	1/8 x 1/4	0	0	Well consolidated	
76 CC-949	7	3-31/32	11-1/2	2.28	2.24	13.6	13.8	6,280	52	1/8	3/16 x 1/2	0	1	One side of core shows about 5 small BAV, but generally core shows good consolidation.	
76 CC-950	8	3-31/32	11-3/4	2.27	2.25	14.1	13.3	6,970	49	3/16	1/8 x 1/2	0	1	Well consolidated	
76 CC-951	9	4	8-1/2	--	--	--	--	6,050	49	1/8	1/8 x 3/8	0	0	Well consolidated	

¹ Permeable voids determined by ASTM C 642.

² TAV - trapped air voids, BAV - bridging air voids.

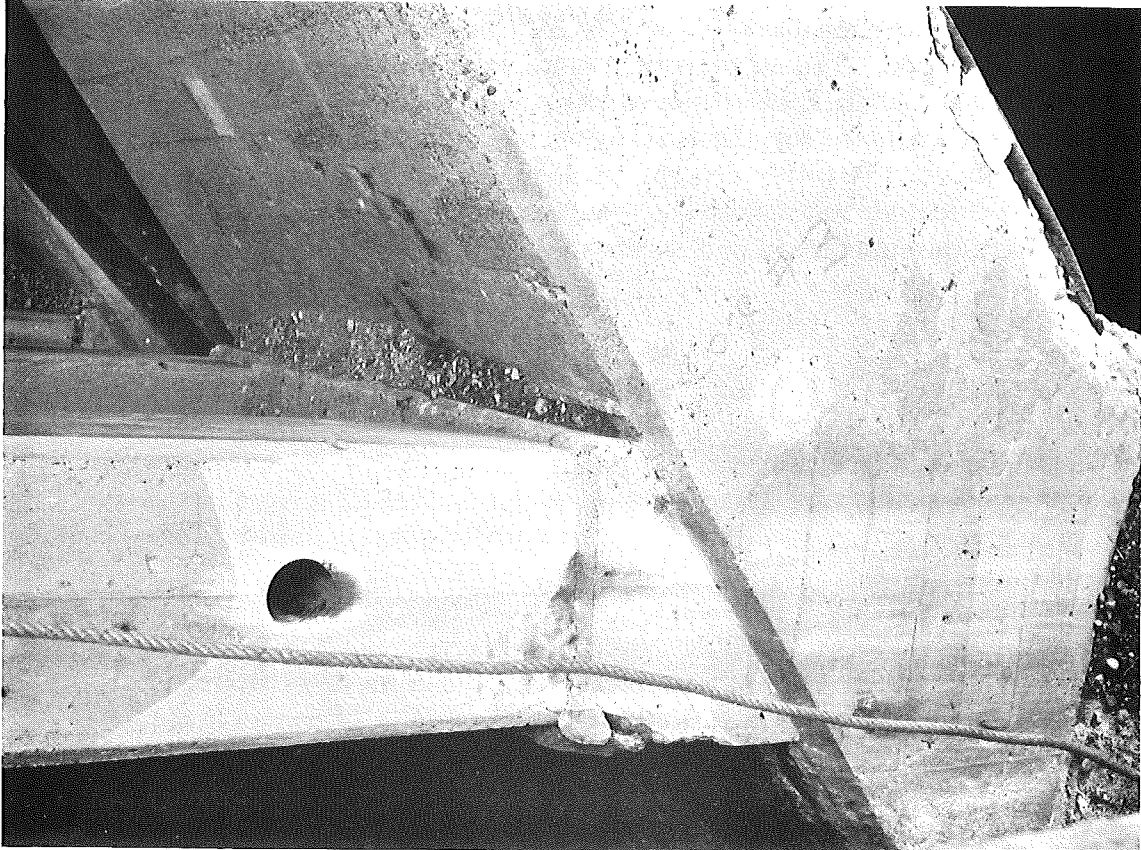
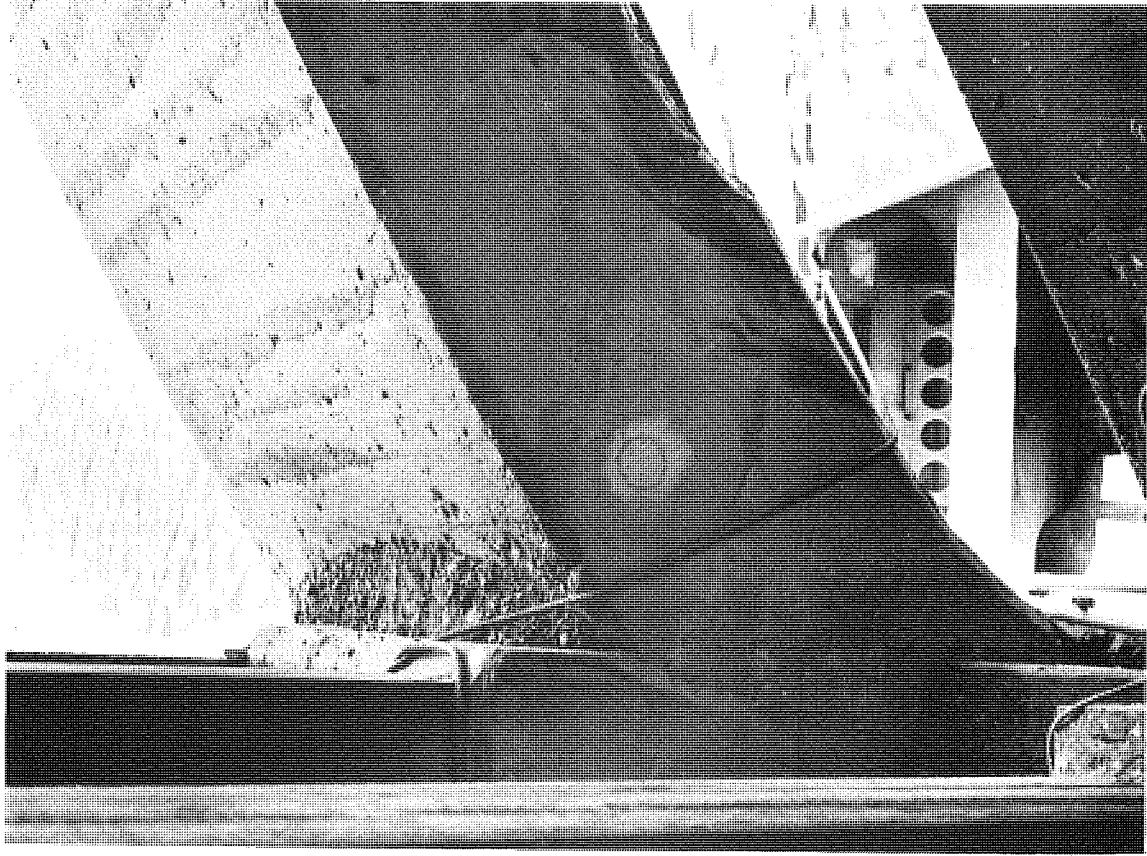


Figure 16. View of location of where cores were drilled. Left: Core No. 4 in column. Proposed Core No. 3 in rib. Right: Proposed Core No. 9.

CONCLUSIONS

Since the Swiss Hammer work was incomplete and insufficient data were available to effect a correlation, the potential usefulness of this portion of the report never developed; hence, it was omitted.

The strength of the concrete cores (Table 1) was very impressive, both from the standpoint of magnitude and of uniformity. Although these cores cannot be considered as representing the strength of the arch ribs in all three spans, they can be considered as representing the concrete mix design that was used throughout the concrete arch ribs and spandrel columns. Thus, it would seem reasonable to assume that the strength of the sound concrete throughout the arches would be typical within moderate variation of these cores. It would also seem reasonable to assume that if the distressed parts of the fascia arch ribs can be economically repaired, and if the arch ribs are sealed against further freeze-thaw deterioration, they should adequately support a new bridge deck for an indefinite length of time. From the standpoint of reduced flexural fatigue stresses, these arches would probably be superior to plate girders in their support of a new concrete deck.

Should economic considerations rule out the repair and re-use of the arch ribs, the substructure units (base of piers and abutments) can still provide an excellent foundation on which to locate pier columns for the support of a new deck. The substructure units appear to be in excellent condition and should be usable without expensive repairs. The only required modifications would be for the column anchorage and aesthetic standards of the new piers. Possibly divers, such as the Scuba diving team on the Mackinac Bridge, should make a deep-water check before plans are finalized to make sure the submerged portions of the piers and abutments are sound.

REPAIR OF DISTRESSED LOCATION RECOMMENDATIONS

Should economic considerations permit the re-use of the arch ribs, it is recommended that the distressed locations be repaired in accordance with procedures outlined in this report or equally effective alternates.

Mid-Span Construction Joints

Following removal of the concrete deck, these areas of distress are again to be carefully examined to determine the limits of concrete deterioration from the top surface. As mentioned earlier, all six of the fascia

arch ribs, or 12 construction joint areas, will require varying degrees of repair. The interior ribs appear to be in good condition in the crown area, but these also should be examined after the deck removal. A discerning operator with a larger (3/4 to 1-in.) rotary impact drill bit will be able to determine the depth of distressed concrete by drilling a pattern of holes. If the limits of deterioration exceed a uniform 3.5 in. depth across the width of the rib, or if the total distressed area comprises 20 percent or more of the cross-section, then the concrete should be removed full-depth and replaced with a shrinkage compensating patching concrete. If the distressed area is less than 20 percent of the cross-section, it should be repaired by simply replacing the damaged concrete with a shrinkage compensating patching concrete. The design of this concrete will be included in the supplemental specification if the repair option is selected.

The reason for a full-depth replacement of the concrete is that 'shrinkage compensating' concrete never completely escapes shrinkage; hence, it will never fully participate with the original concrete in uniformly carrying subsequent compressive loads. With more than 1/5 of the section gone, this might present load distribution problems.

The principle employed in most shrinkage compensating concretes is to obtain sufficient expansion, most of which occurs during the initial 18 hours, to not only counter the initial 18-hour shrinkage, but to also counter the subsequent shrinkage that occurs as the concrete dries out. This principle works best in grouting work where the concrete is totally confined. In partial confinement applications, when the multi-directional expansion reaches the limits of expansion in a confining direction, the concrete begins building compression in that direction. When the elastic limit is reached, however, the elastic shortening associated with compression in that direction ceases, and plastic expansion occurs in the direction of least resistance. The compressive strength of most green concrete of this age is quite low so very little elastic shortening becomes available to counter subsequent drying shrinkage.

If full-depth concrete removal is required in the arch rib crown areas, a tentative method of support and repair has been devised. This can be discussed with the Design Division at the appropriate time.

Rib Spalling at Abutment B

The distressed concrete at the north end of the west fascia rib as shown in Figure 14 should be removed and replaced with the same shrinkage compensating patching concrete described previously. Should removal work reveal deep interior consolidation voids, provisions should be made for epoxy injection grouting.

Other Arch Rib Spalls

All areas displaying minor distress, such as those shown in Figures 7 and 8, are to be chipped down to solid concrete and patched with a latex mortar or concrete. Another equally good alternate for deeper areas would be the preplaced aggregate concrete method, since forms will be required to hold the patch in place regardless of the patching mixture used. The preplaced aggregate method is especially suited to overhead applications such as shown in Figure 8. Included with these areas shall be the loosened patches and spalled concrete at the lower corners of the rib cross-section, and at all original construction consolidation voids. The mix design as well as repair procedures for this group shall be detailed in a supplemental specification if this option is selected.

Additional field data will be needed to obtain a better estimate of number and size of repair areas such as shown in Figures 6 through 8. Possibly a joint inspection can be made soon with Research Laboratory and Design Division personnel to obtain these measurements. The limits and depth of concrete removal will be difficult to estimate accurately, but should be close enough to arrive at a realistic repair figure. The most difficult area to estimate deterioration is in the crown portion of the arch ribs where the top of the arch is covered by the 'boxed in' sections.

Sealing Repaired Arch Ribs

After repairs have been completed on the arch ribs, they should be sealed against any further moisture penetration to prevent additional freeze-thaw deterioration. The material used for this protective coating as well as the preparation and application procedures will be detailed in a supplemental specification should this option be selected.