DETERMINATION OF CONCRETE STRENGTH IN THE ARCH RIBS OF BRIDGE B01 of 22011
(M 95 over Menominee River)

MICHIGAN
HIGHWAY RESEARCH LABORATORY SECTION

MICHIGAN DEPARTMENT OF STATE HIGHWAYS

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IN THE ARCH RIBS OF BRIDGE B01 of 22011
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Testing and Research Division
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INTRODUCTION

In a letter to R. L. Greenman, dated May 6, 1970, M. Rothstein explained that the Design Division was planning to renovate the subject bridge by placing a wider and more adequate deck on the present arch ribs. The present structure is a three-span concrete arch design with a total length of 246 ft and a 19-ft roadway. Since the bridge was constructed in the winter of 1918-19, and the quality of the concrete in the arch ribs was unknown, they thought it best to have the concrete examined before proceeding further. After consulting with M. G. Brown of the Research Laboratory, it was decided to check the quality of the concrete by means of a Swiss Hammer survey; with readings to be taken at regular intervals along the arch ribs and compared with similar readings taken at core locations on the spandrel walls, pier pilasters, and abutment wing-walls. The cores obtained would hopefully establish a correlation between the concrete compressive strength and the Swiss Hammer readings. Thus, the concrete compressive strength of the arch ribs could be indirectly determined without actually coring them. The mechanics of the Swiss Hammer are discussed in the Appendix.

PRELIMINARY FIELD WORK

In May, an initial trip was made to the bridge site to generally assess the problem and obtain some initial cores and hammer readings. It was noted that water was entirely under all three spans with a swift current under the south span (Fig. 1). This limited the possible arch-rib working platforms to either a floating scaffold or one suspended from the bridge itself. Upon request, District 1 furnished chain, block and tackle, and a 20-ft painters scaffold. After manually lowering the scaffold over the side of the bridge and suspending it with block and tackle from the chains mounted to the parapet rail, it was discovered that the scaffold was unstable, dangerous to stand on, and too far removed from the arch rib to do any work. It was evident that more elaborate and specialized equipment would be required.

In spite of the unsuitable scaffolding, Swiss Hammer readings and 4-in. cores were taken from six locations along the abutment wing-walls that could be reached from the earth embankment. Before leaving the area, various techniques were discussed and basic measurements were taken for designing suitable scaffolding.

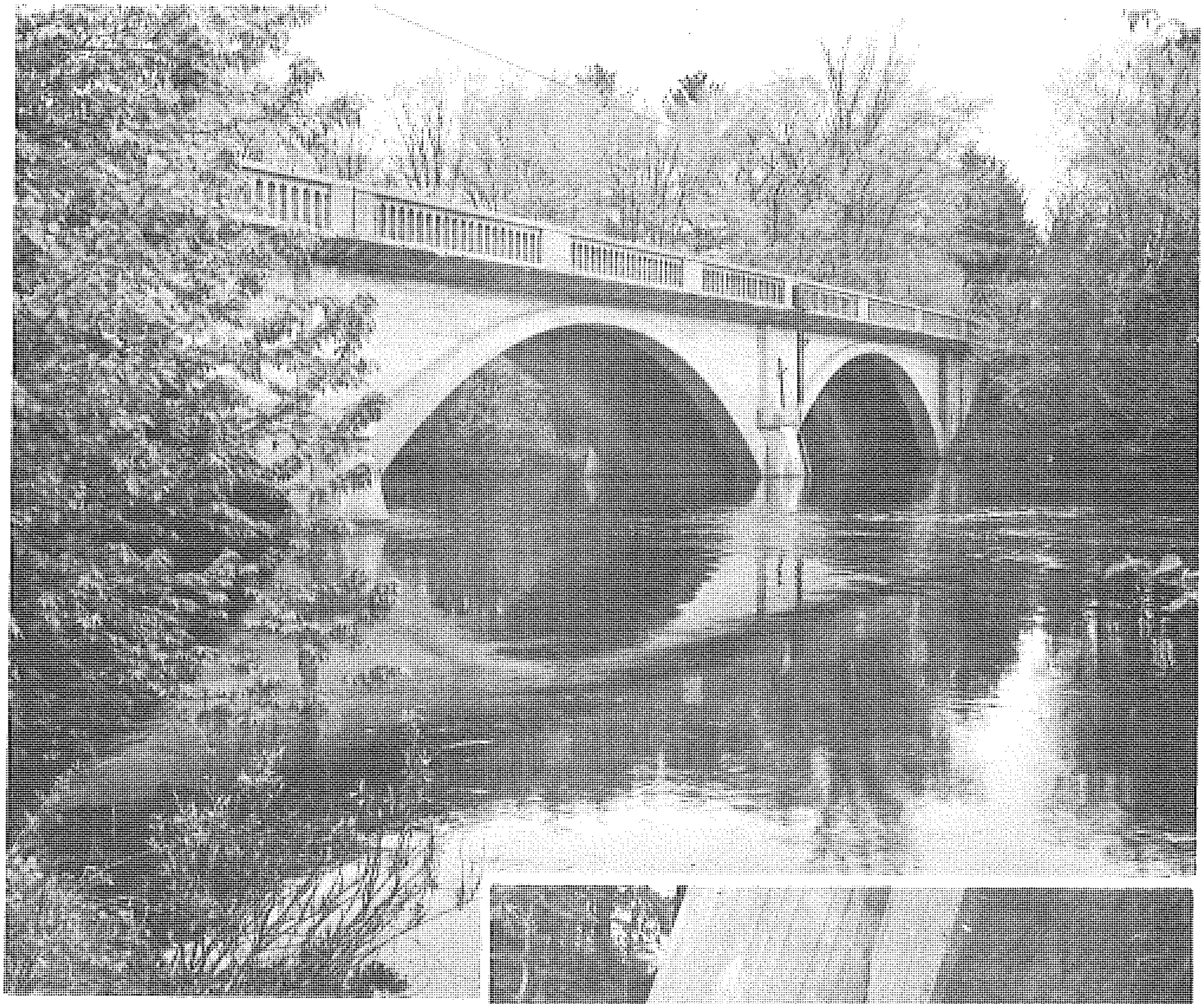


Figure 1. View of east side of bridge from the south (Wisconsin) bank.

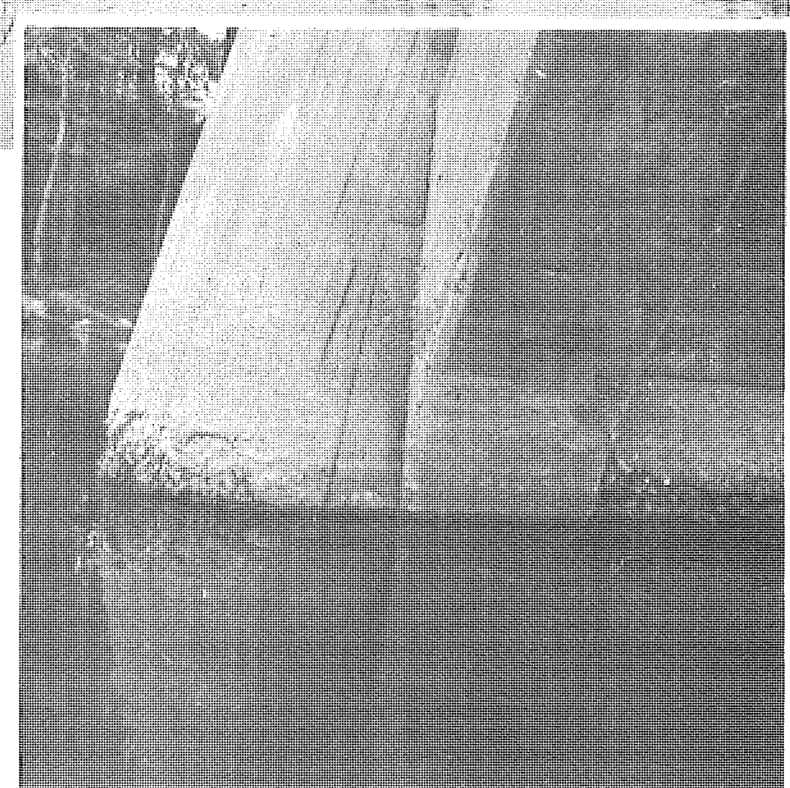


Figure 2. Water line heavy scale and spalling on the west nosing (south side of the north pier).

FINAL INSPECTION

After returning to Lansing, a workable scaffold was devised that would utilize a float to correctly position it beneath the bridge. The scaffold was to be manually raised off the float by block and tackle to a working position beneath the deck. When all the work at that position was completed, the scaffold would again be lowered to the float which would be moved to a new position and the procedure repeated. Besides being cumbersome, this arrangement would be excessively time consuming, since half of the time would be spent in repositioning the scaffold.

In an effort to secure alternative methods, S. M. Cardone of the Maintenance Division was consulted. He stated that an ideal machine for this work had been developed, but that the State of Michigan did not own one. He recently had observed a demonstration of Wisconsin's new machine known as the "Snooper." It is a truck-mounted hydraulic crane-type machine with an articulated and telescoping boom which can place its working platform immediately beneath the deck which supports it. From this railing-encircled platform, men have an inter-communication system with the operator and can perform underside inspection or repairs in complete safety.

Mr. Cardone offered to rent the "Snooper" from Wisconsin, but since it was an interstate bridge they graciously donated the services of the machine and its operators in spite of the fact that the bridge connected only a Wisconsin county road to Michigan's M 95. We had the services of the machine for the first two days of the week that it was scheduled to return to Madison for the correction of a minor malfunction.

At this point we wish to acknowledge the fine cooperation extended to us by the two machine operators from the Wisconsin Department of Highways and also to the Sheriff of Dickinson County who furnished us with a boat and operator for the waterline pier inspection.

Pier Inspection

The piers were inspected from a boat at and below the waterline. Heavy surface scale was typically visible at the water level, especially on the west nosing of the north pier (Fig. 2). Probing by hand below the water level revealed an irregular groove approximately 6 in. wide and 1 to 2 in. deep running horizontally around the piers at approximately 1-ft below the surface; this elevation was interpreted as being the water level during winter. This groove was checked at regular intervals around the piers and, although one place was found to be spalled to a 4 in. depth, no reinforcing bars were exposed.

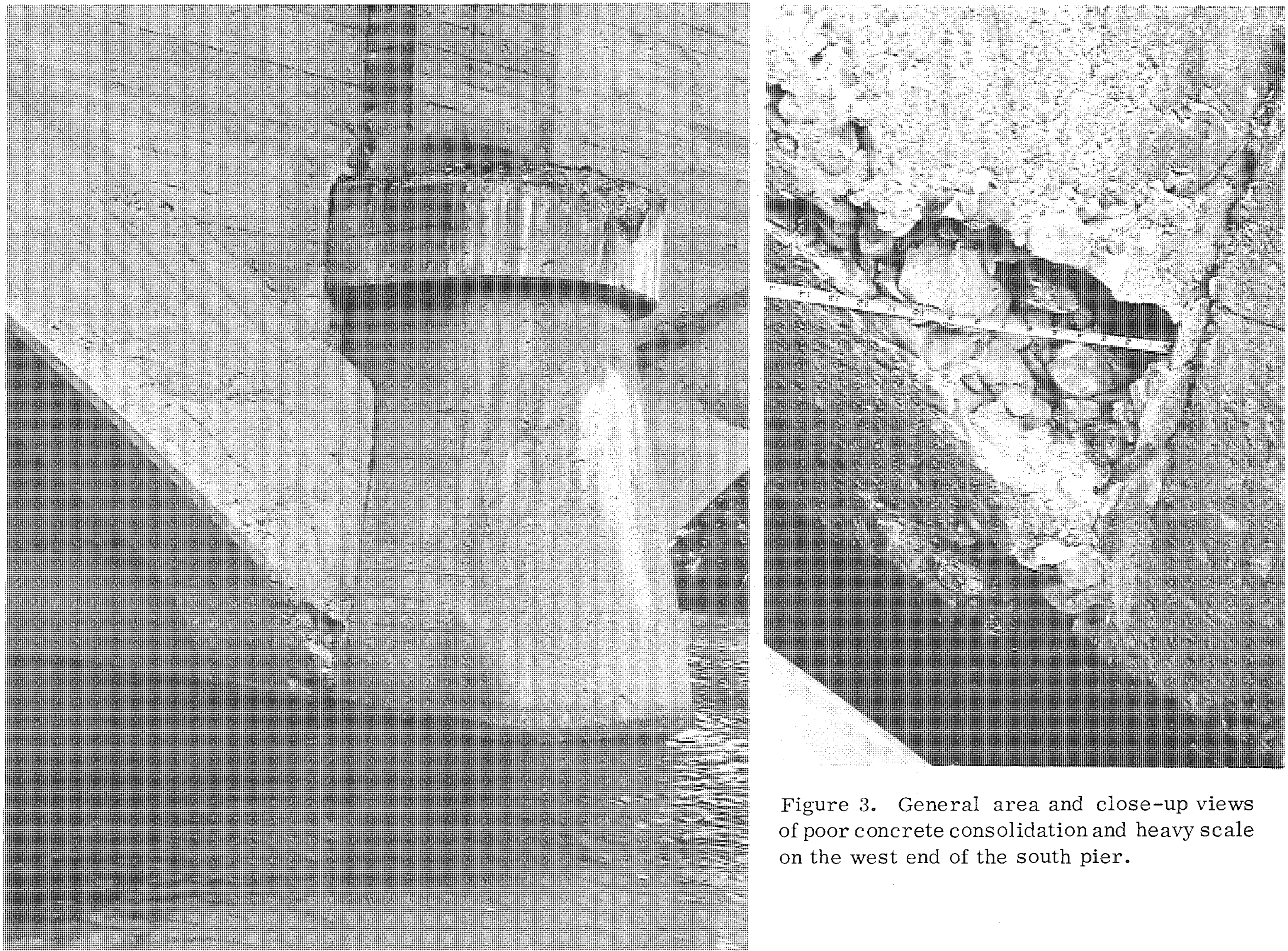


Figure 3. General area and close-up views of poor concrete consolidation and heavy scale on the west end of the south pier.

At the base of the intersection between the south pier and the west arches, some large voids were visible. After a close inspection it was concluded that these were the result of poor consolidation at the time of the original construction. Figure 3 shows the location and relative size of these voids.

Arch Rib Investigation

In effecting the Swiss Hammer survey, it was not only necessary to get the readings, but also to know the exact location on the bridge where the reading was taken, and to have an effective means of recording the values. To accomplish this both the east and west parapet rails were marked off at 10-ft intervals, beginning at the face of the north abutment pilaster and running south to the south abutment. From the north end, each of these marks were then successively numbered with paint from 0 to 26. To supplement the physical marking of the bridge, drawings were made on graph paper of all three spans; including a plan view of the deck and elevation views on either side. Both elevation views were marked and numbered from north to south in a manner corresponding to the physical numbering on the parapet rails.

It was planned that the hammer reading at any one position would not be a single reading but an average of seven individual readings taken at the six angles and center of a 3-in. hexagon. The pattern was to be marked on the concrete with a lumber crayon and each point successively tested while the results were recorded on a separate set of tables. These testing positions were purposely selected so that all the hammer points could be concentrated within a 4-in. circle where a core could be drilled. It was presumed that these seven readings would give a realistic average value to correlate with core strength.

The center half of each arch was checked on both the outside and inside vertical faces. Particular emphasis was given to the crown area where test positions were spaced at 5-ft intervals. Moreover, in each span, two test positions were checked on the spandrel wall where cores were later drilled for correlation purposes.

The procedure employed at each test position was as follows: An instrument called a Swiss Pachometer was used to find the location of the reinforcing bars. Their location was then marked on the face of the concrete with a lumber crayon. In cases where the texture of the concrete surface was fairly uniform, a convenient spot between the steel bars was selected as a test position. It was important that steel bars not be located beneath a test surface, as their presence could greatly increase the ap-

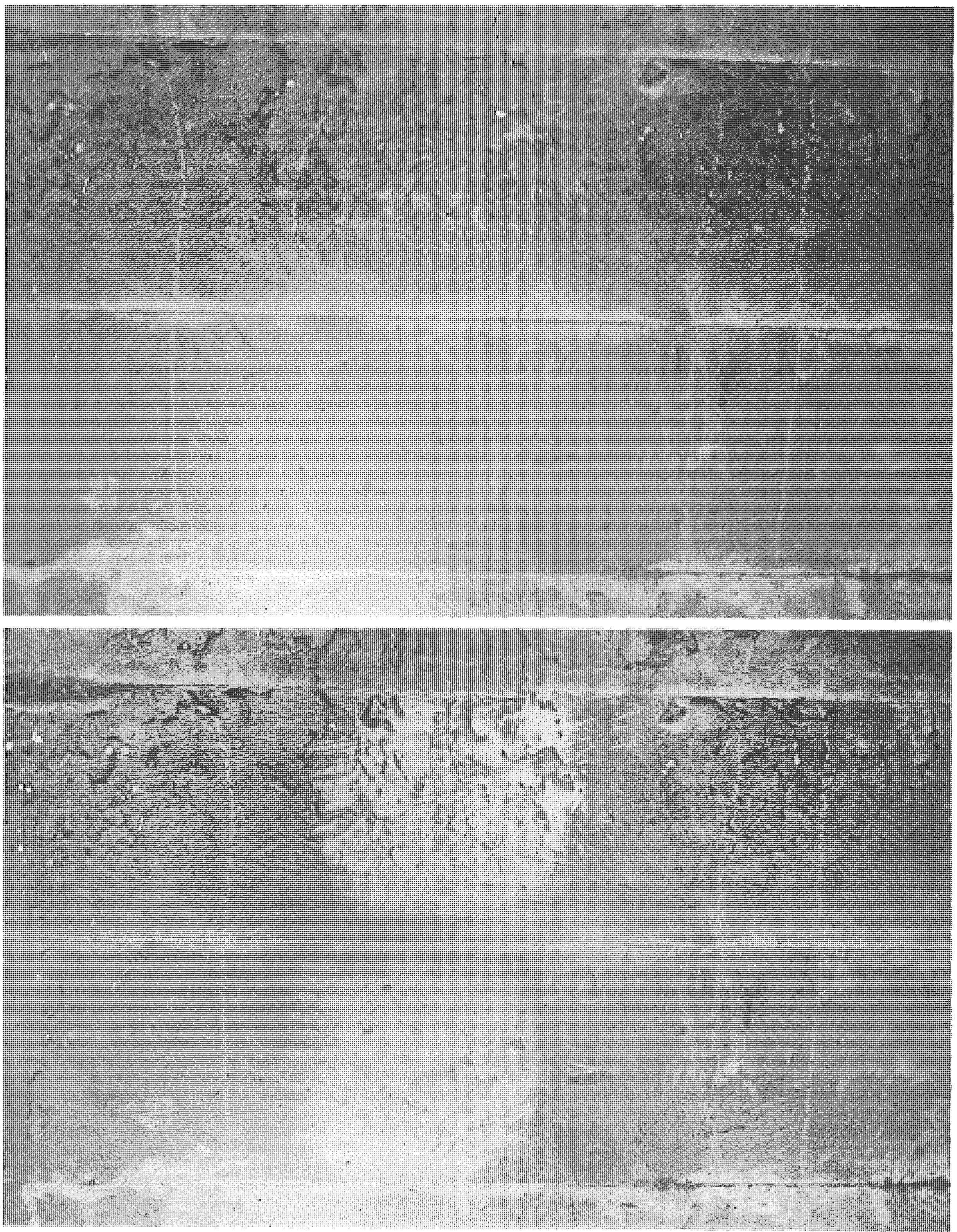
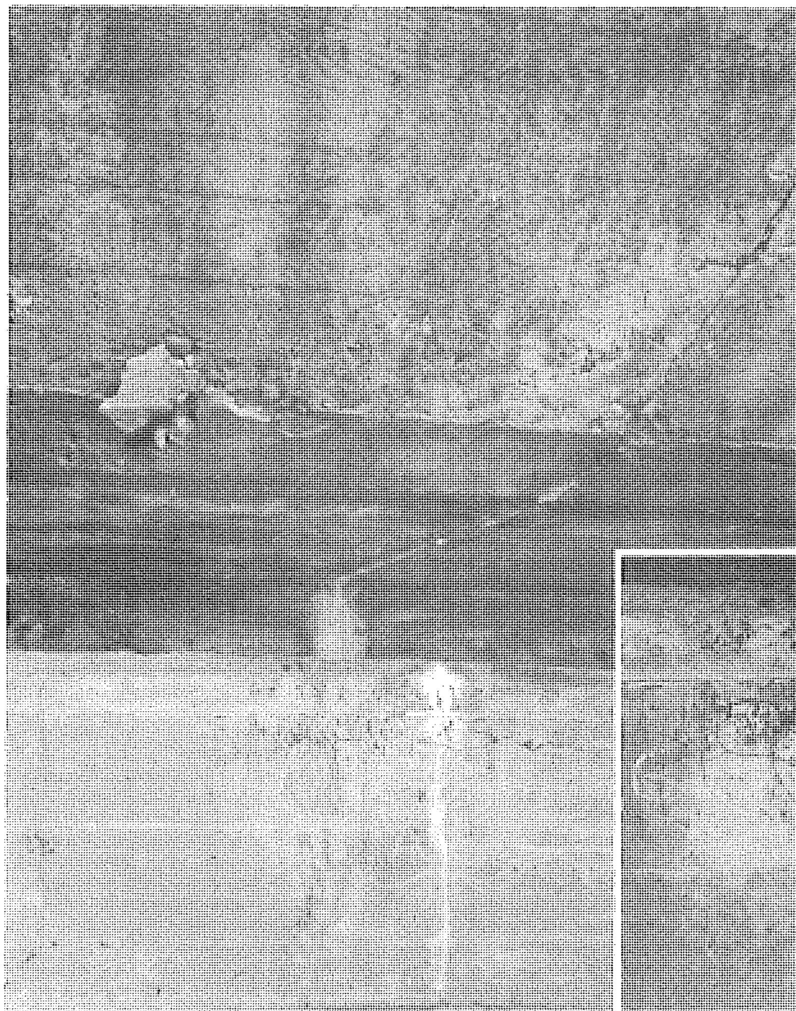


Figure 4. West arch-rib hammer reading positions 55 and 56 before and after surface preparation. Note variability of surface texture between the two positions.

parent hardness of the concrete and indicated hammer reading. In places where two significantly different surface textures existed, two test positions were selected that were each representative of their respective areas. The test positions were then rubbed with a carborundum block to remove grain striations caused by the wooden forms, and the test pattern was marked on the smooth uniform surface. Each point of the pattern was then successively tested and the results recorded. Figure 4 shows views before and after surface preparation at an area where two different surface textures exist. Average Swiss Hammer readings are tabulated in the Appendix as Tables 3, 4, and 5.

In addition to the Swiss Hammer work, all suspicious looking places on the arch ribs were sounded with a geologist's hammer to detect any weak or freeze-thaw damaged areas. The only notable blemishes found were near the middle of both the east and west arches of the center span, and on both east and west arches of the north span. The center span blemish on the west arch was directly beneath a deck construction joint at the crown of the arch; there, de-icing salts had permeated down through the joint and caused a large incipient spall area to develop on the outside vertical face. The limits of the hollow-sounding area were found by tapping with a hammer and outlined with a lumber crayon. Since it was necessary to ascertain the extent of the damage, the distressed concrete was removed and the affliction was found to be superficial; extending no deeper than 1-1/2 in. and exposing no reinforcing steel. Figure 5 shows the afflicted area before and after the investigation. On the center span east arch a patched area was noted, but sounding proved it to be securely bonded.

In the north span arch ribs a different texture was noted in the concrete at several places along the bottom vertical face. Little attention was paid to this until tapping produced a hollow sound at one of these places. The investigation which followed found this to be the location of a bar lap where the original concrete placement left a void. These voids then were apparently patched with mortar after the forms were removed. Figure 6 shows one of these voids after its loosened patch was removed. It is obvious from the picture that poor concrete consolidation was not limited to the bar lap alone since the area above it was also poorly consolidated. It would thus appear that either arch construction began in the north span, or more problems were encountered there, because in the center and south spans these bar lap voids were not present. It should be remembered, however, that this work was performed in the middle of winter without the benefit of mechanical vibration, efficient insulation, modern admixtures, or modern equipment for mixing and placing concrete.



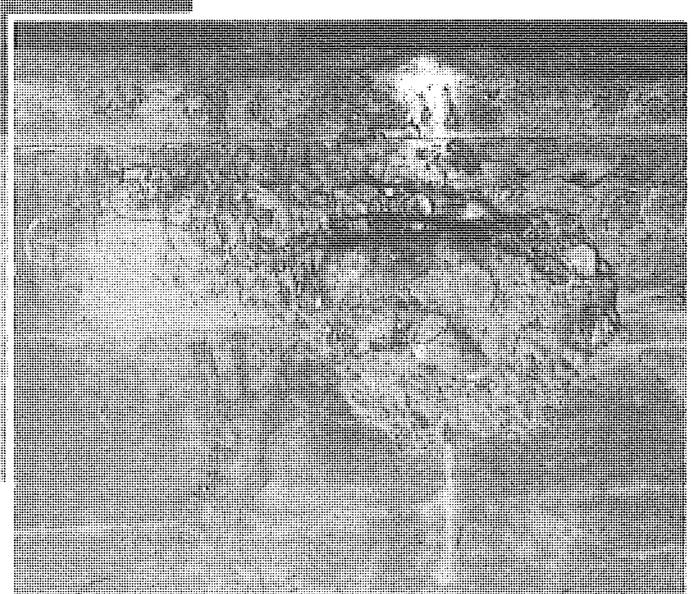


Figure 5. View of salt-afflicted area before and after investigation (mid-point of the center span on the west arch).

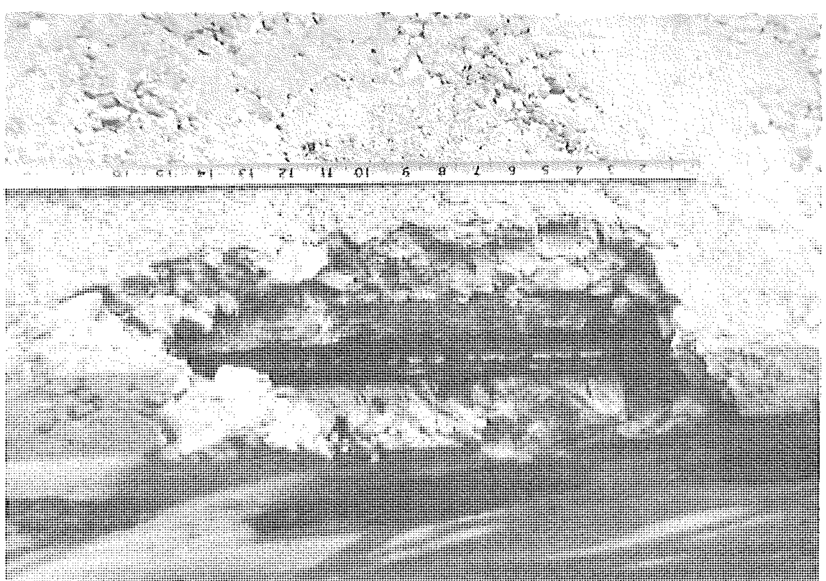


Figure 6. Void area exposed after removing loose patch to check depth and extent of distressed concrete (east arch rib 12 feet south of the mid-point in the north span). The original concrete void area was caused by inadequate consolidation around the lap of the 1-in. square bars.

EVALUATION WORK

Laboratory Work

In the Laboratory the 4-in. cores were closely inspected and were noted to have coarse aggregate whose maximum size appeared to be at least 4 in. The mortar itself was an earthy tan color which gave the impression that it was lean in cement or the fine aggregate contained a significant amount of clay or silt. Figure 7 shows the most representative sides of these cores.

The compression tests of these cores yielded highly variable results. This fact was expected when the excessive size of the coarse aggregate was considered. To yield reliable compression test values a core should have a diameter three times the size of the maximum coarse aggregate size; however, in our case, drilling a 12-in. core with a portable rig would have been impossible.

Core No. 4 failed at an abnormally low value with a distinct diagonal shearing plane adjacent to a large rectangular shaped piece of rock. To investigate this unusual occurrence, the core was crushed and the subject rock isolated and identified as Michigamme slate. This flat piece of rock had been tilted at 45° to the axis of the core and, since it comprised about 30 percent of the cross-sectional area, caused the core to fail along its surface. This occurrence prompted a more intensified study; so, after the cores were crushed, the coarse aggregate was separated from the mortar. A petrographic analysis revealed that the coarse aggregate ranged in size from 4-in. cobbles to 3/8-in. pebbles. Its composition was approximately 70 percent igneous, 20 percent metamorphic, and 10 percent sedimentary. Most of the metamorphic and some of the sedimentary rocks were laminated and included gneisses, schists, slates, and shales. All of the individual rocks identified were common to the Iron Mountain area. From the rounded edges of the coarse aggregate, it would appear that the rock came from a gravel pit of glacial origin. The rounded edges would indicate that substantial weathering and attrition had taken place. This could have also significantly lowered the shear resistance along the lamination planes of the metamorphic and sedimentary rocks.

Core-Cylinder Relationship

A recently published paper in the Journal of the American Concrete Institute[1] describes research conducted to determine the relationship be-

[1] Bloem, Delmar L., "Concrete Strength in Structures," ACI Proceedings, Vol. 65, No. 3, pp. 176-187.

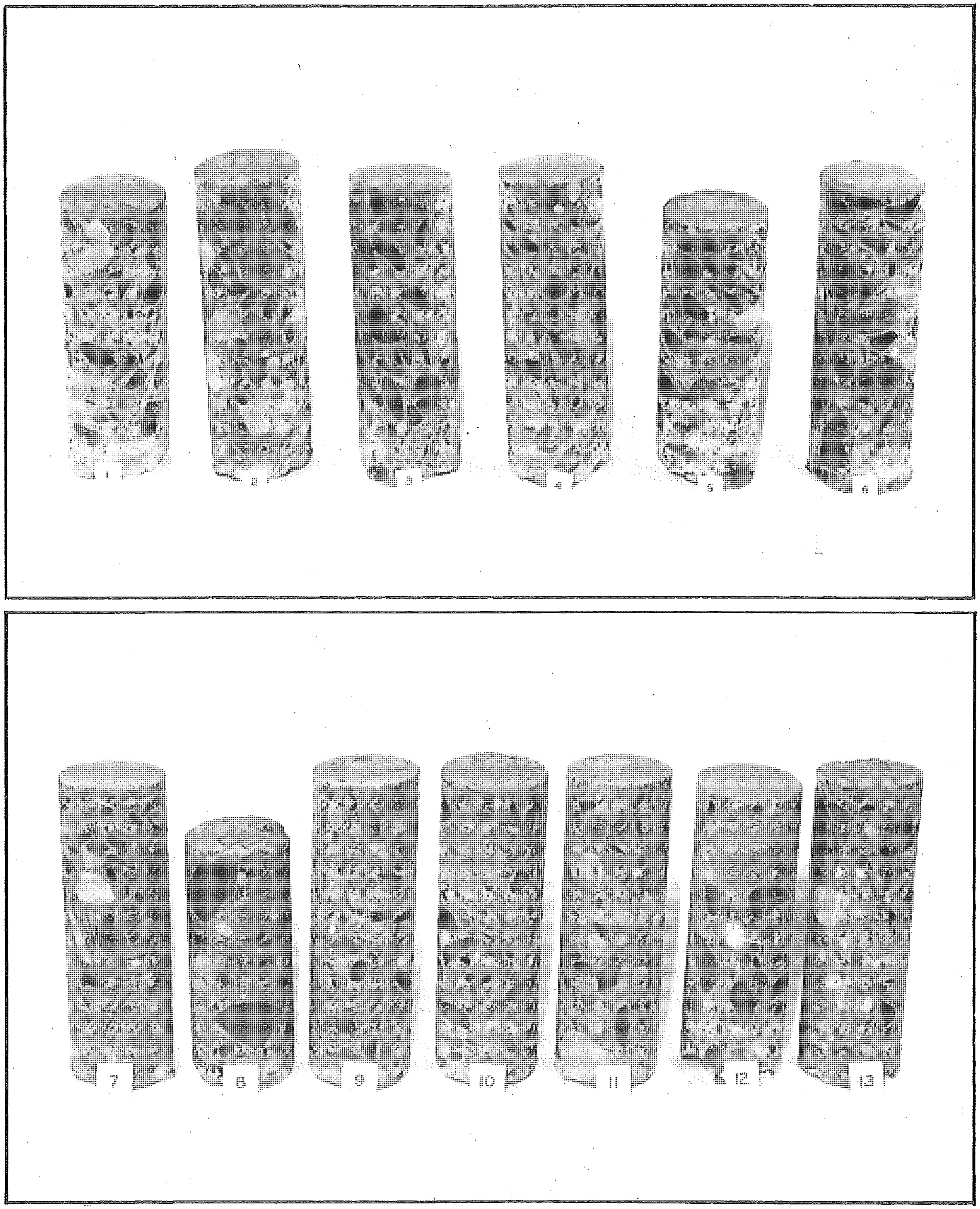


Figure 7. Cores 1 through 6 were cut from abutment wing-walls; cores 7 through 13 were cut from west spandrel wall. Note size of coarse aggregate.

tween the compressive strengths of cores and cylinders. In this work, sleeves were set in slab forms prior to the placement of concrete such that the resulting cylinders would receive curing identical to the slab. Subsequently, cores were drilled from these slabs and were tested in compression along with the "push-out" cylinders. It was consistently found that the cores developed approximately 90 percent of the push-out cylinder's compressive strength. The reason for this can be readily understood when it is noted that aggregate discontinuity results around the periphery of the core when it is cut by the core drill.

Based on the above core - cylinder strength relationship, the equivalent cylinder strengths were determined. Table 1 gives the average Swiss Hammer readings obtained at the position where the cores were drilled, the actual core compressive strengths, and the projected cylinder strengths.

TABLE 1

PROJECTED CYLINDER STRENGTH OF CORES

	Core No.	Location	Average Swiss Hammer Reading	Actual Core Strength, psi	Core-Cylinder Strength Ratio	Projected Cylinder Strength, psi
Abutment wingwall cores	1	SE wingwall	55	3,430		3,810
	2	SE wingwall	55	2,700	0.90	3,000
	3	SW wingwall	54	3,240		3,600
	4	NE wingwall	52	1,390*		1,540
	5	NE wingwall	51	2,630	0.90	2,920
	6	NW wingwall	53	1,900		2,110
West spandrel wall cores	7	S span	43	1,740		1,930
	8	S span	41	2,270	0.90	2,520
	9	center span	54	3,260		3,620
	10	center span	44	2,750		3,060
	11	N span	56	2,740	0.90	3,040
	12	N span	50	2,300		2,560
	13	N span	45	1,930	0.90	2,140

* Suspected low break from large piece of slate in cross-section.

Concrete Strength Correlation Work

Upon beginning the correlation work to establish a relationship between the average Swiss Hammer readings and the core strengths, it was discovered that so much variation existed that no distinct relationship was evident (Fig. 8). It was noted that the cores, which contained a substantial amount of large coarse aggregate (retained on a 1-1/2-in. sieve), generally tested to a lower value than the cores containing small amounts of large coarse aggregate. Thus, in order to discretely view the value of a core test result it became apparent that the percentage of coarse aggregate larger than 1-1/2 in. should be known; and the smaller the percentage, the more reliable the test result. To implement this supposition, the mix design was estimated and the theoretical amount of coarse aggregate in a 4- by 8-in. core was calculated. The mix design for the rib arches, wingwalls, and spandrel walls was indicated on the plans to be 1:2:4 by loose volume measure. This would roughly be 5 to 5-1/4 sacks of cement per cubic yard. The disintegrated cores were again examined and the coarse aggregate larger than 1-1/2 in. was isolated and weighed. The weight of the large coarse aggregate was divided by the estimated total coarse aggregate and the percentages tabulated for each core. After examining the results, an arbitrary limit of 40 percent large coarse aggregate was set;

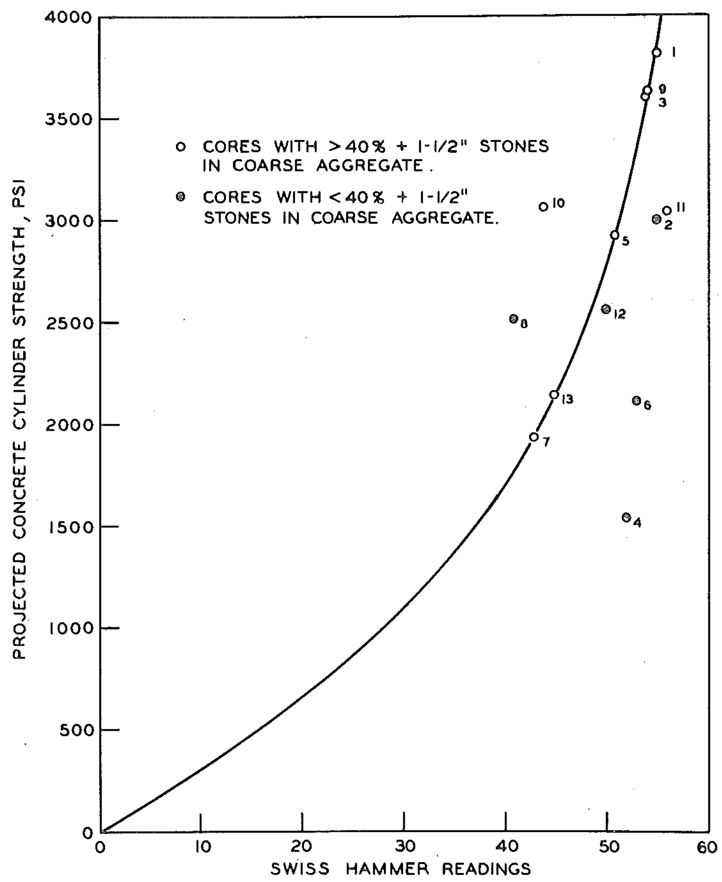


Figure 8. Average hammer reading - core strength relationship.

the compressive strength of those cores containing a greater percentage was considered of little value. Figure 8 shows all of the core's projected cylinder compression test results plotted opposite their average Swiss Hammer readings. Circled points denote cores containing less than 40 percent large coarse aggregate, and solid points denote cores containing more. The figure shows a distinct curve defined by all but two of the "reliable" cores. From this curve shown in Figure 8, compression strength values corresponding to each hammer reading were obtained and are recorded in Table 2.

Figures 9, 10, and 11 are diagrams which show plan and elevation views of each of the three spans of the bridge. On each elevation view the locations are shown where cores were drilled and Swiss Hammer readings were taken; also shown are the figures--expressed in feet-which locate distances from the north to the south abutment.

The plan view shows the average concrete strength of the arch ribs at the corresponding locations of the Swiss Hammer readings. These values are the projected cylinder strengths of the concrete, as obtained from Table 2, and represent the average of the near side and far side values at a particular location on the arch rib. The tables showing the conversion from hammer readings, and the averages of these values along the arches, are shown in the Appendix (Tables 3, 4, and 5).

CONCLUSIONS

The inspection of the piers was limited to an assessment of the depth of scaling at the water line. It established that the scaling damage was only superficial, even though it appeared unsightly. No attempt was made to evaluate the quality of the concrete within the piers.

The projected concrete cylinder strengths of the arch ribs, as is shown in Figures 9, 10, and 11, indicate the west arch of the north span contains the poorest concrete and could not be expected to develop more than 1,700 psi ultimate strength. Corresponding estimates of the minimum strength values for the center and south span arch ribs were 2,000 and 2,500 psi, respectively.

The success of this inspection project in a reasonable length of time was made possible by Wisconsin's "Snooper;" with this machine we were quickly able to get into working positions which were ideal for inspecting the arch ribs, spandrel walk, and the underside of the deck. Using conventional scaffolding methods, this job would probably have taken three times as long to complete and might also have jeopardized the inspection personnel's safety.

TABLE 2

CONVERSION FROM SWISS HAMMER READINGS
TO CYLINDER STRENGTHS

Hammer Reading	Projected Cylinder Strength (psi)	Hammer Reading	Projected Cylinder Strength (psi)
31	1180	46	2420
32	1240	47	2550
33	1300	48	2680
34	1360	49	2830
35	1420	50	2980
36	1490	51	3150
37	1560	52	3340
38	1640	53	3540
39	1720	54	3760
40	1800	55	3990
41	1880	56	4230
42	1970	57	4500
43	2060	58	4810
44	2170	59	5160
45	2290	60	5560

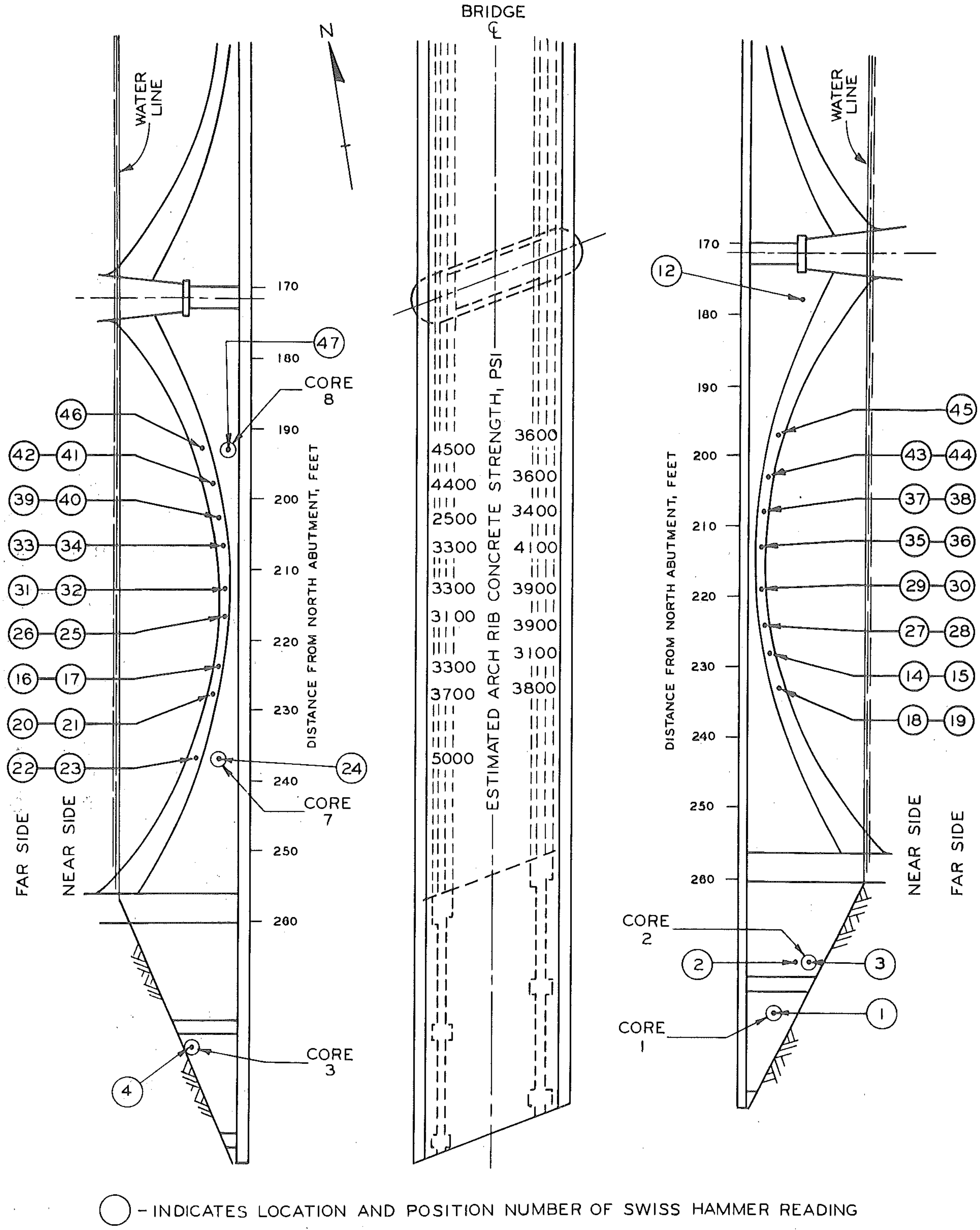


Figure 9. Core locations, Swiss Hammer reading locations, and estimated concrete strengths in the south span.

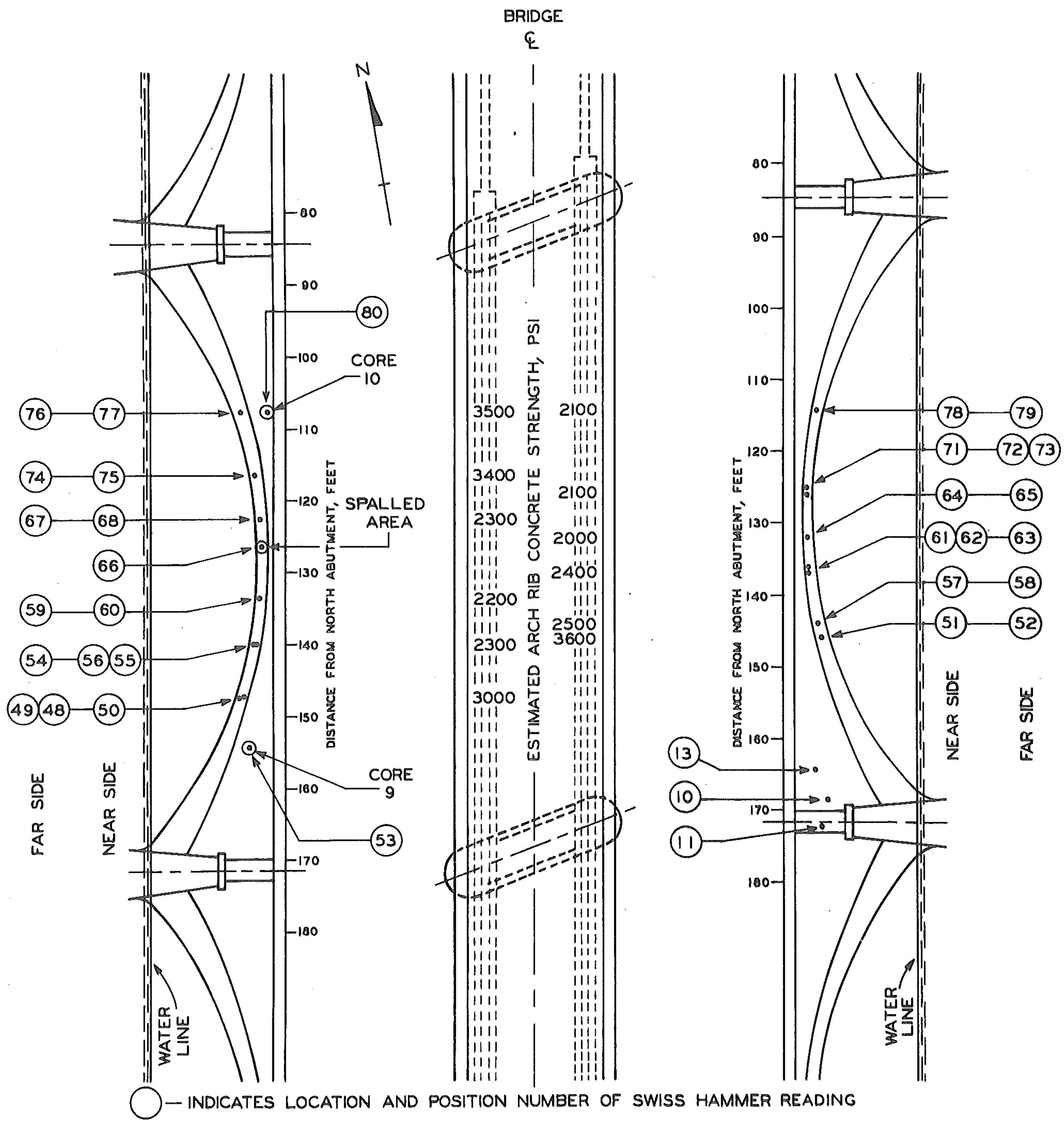


Figure 10. Core locations, Swiss Hammer reading locations, and estimated concrete strengths in the center span.

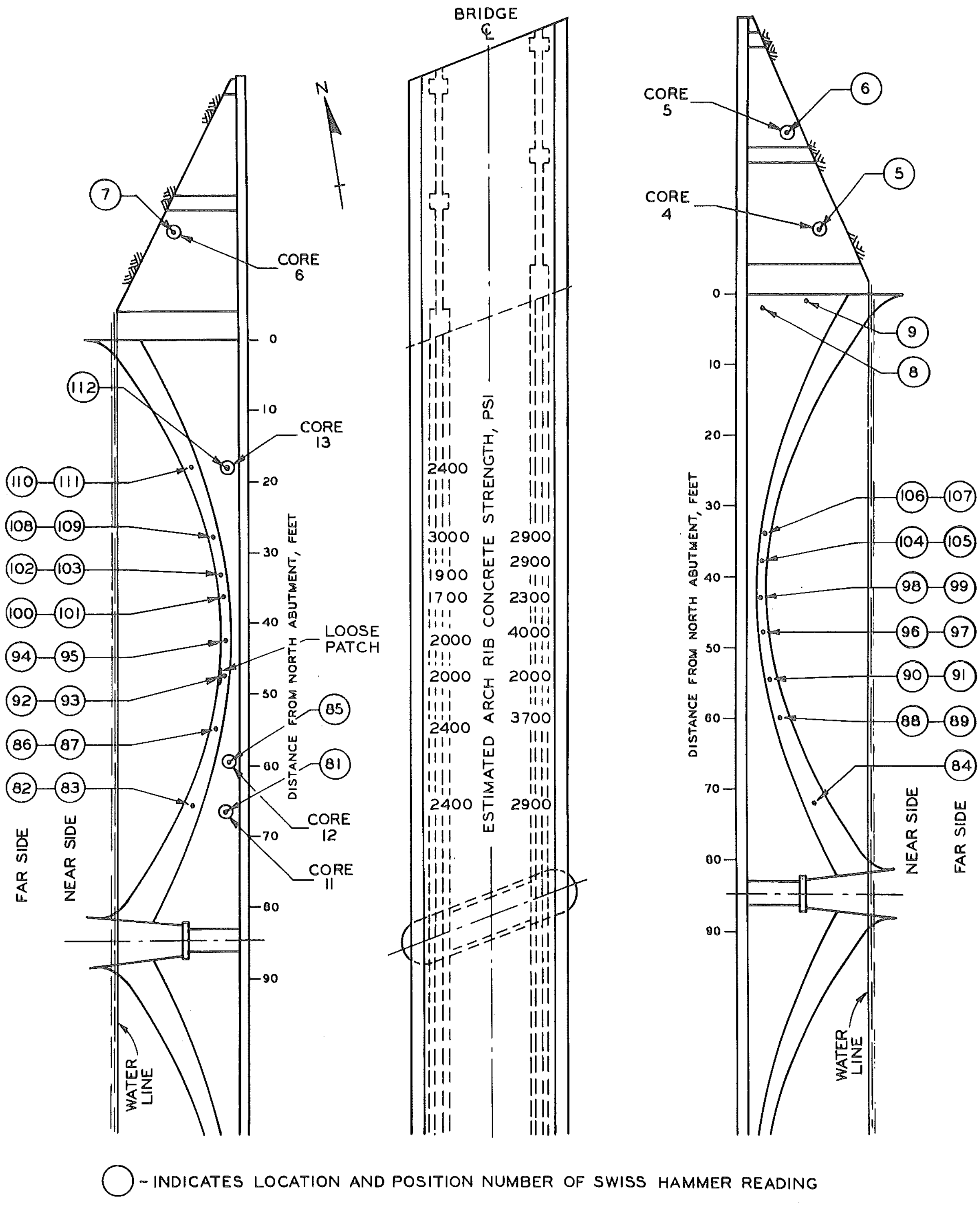


Figure 11. Core locations, Swiss Hammer reading locations, and estimated concrete strengths in the north span.

APPENDIX

MECHANICS OF THE SWISS HAMMER

A Swiss Hammer is a patented instrument used to determine the hardness of different materials. It is a cylindrical tool with a sliding shaft which protrudes from one end (Fig. 12). At the beginning of a test, the shaft--which is slightly rounded at it's exposed end--is placed in its fully extended position and is in contact and normal to the surface to be tested. As the body of the hammer is pushed toward the surface, the shaft pushes against a spring loaded sliding weight inside the body until the shaft is fully retracted and seats against the rear of the hammer. At this point the sliding weight is tripped and it's fully compressed spring drives it toward the front of the hammer where it strikes an anvil and rebounds against its spring. On the rebound, the sliding weight moves an indicator along a scale which is calibrated from 0 to 100. The amount of rebound depends on how much impact energy is absorbed by the test surface; the harder the surface, the greater the rebound. In the case of concrete, where its compressive strength is directly related to its hardness, a correlation can be established such that its compressive strength can be estimated with reasonable accuracy. Since concrete is not a homogeneous material, however, several tests must be taken at various points in the same area to secure a representative average value. Large particles of coarse aggregate, reinforcing steel, and large voids near the concrete surface will adversely affect rebound readings.

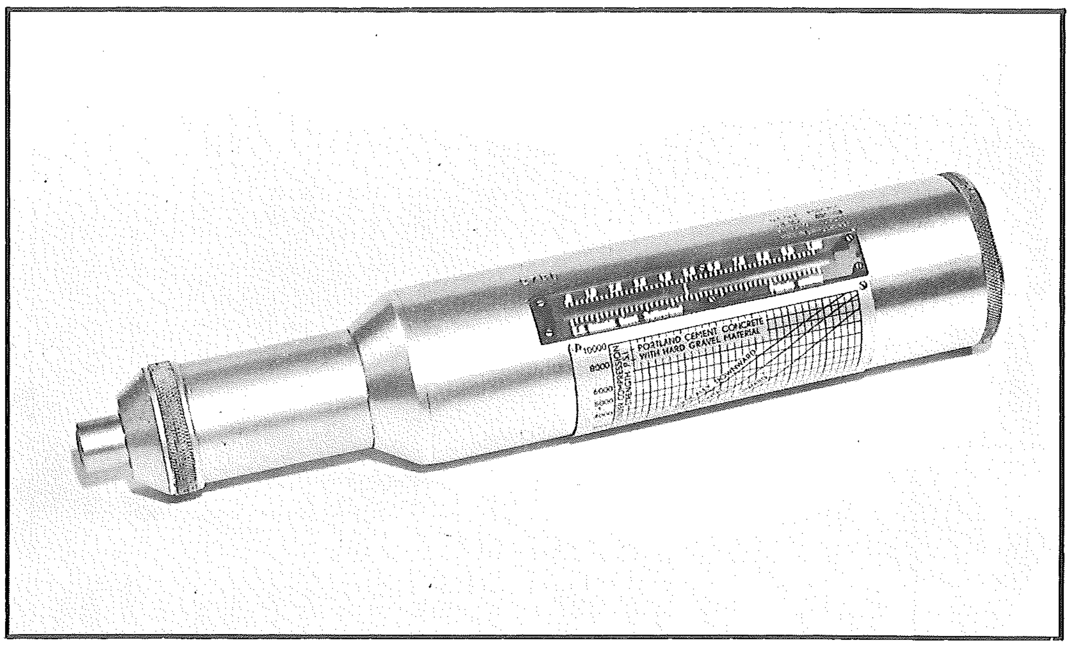


Figure 12. Model CT320 Swiss concrete test hammer. Shown in this view is the reading indicator scale and a calibration chart.

TABLE 3

SWISS HAMMER READINGS AND ESTIMATED CONCRETE STRENGTH-SOUTH SPAN
(Readings taken on the arch rib unless otherwise specified)

West Arch

Location, ft from N abutment line	Position Number	Average[1] Hammer Reading	Projected Cylinder Strength, psi	Avg. Proj. Cylinder Strength, psi
abut. wing-wall 279	4	54	3590	3600
237 outside spandrel wall	24[7]*	43	1940	1900
238 inside	22	58	4830	5000
238 outside	23	59	5150	
227 inside	20	52	3120	3700
227 outside	21	56	4180	
224 inside	16	53	3340	3300
224 outside	17	53	3340	
217 outside	25	51	2930	3100
217 inside	26	53	3340	
213 inside	31	51	2930	3300
213 outside	32	54	3590	
207 inside	33	54	3590	3300
207 outside	34	51	2930	
203 inside	39	52	3120	2500
203 outside	40	43	1940	
198 outside	41	55	3880	4400
198 inside	42	58	4830	
193 outside spandrel wall	47[8]	41	1770	1800
193 outside	46	57	4500	4500
193 inside		Could Not Reach		

East Arch

Location, ft from N abutment line	Position Number	Average[1] Hammer Reading	Projected Cylinder Strength, psi	Avg. Proj. Cylinder Strength, psi
abut. wing-wall 279	1[1]	55	3880	3900
abut. wing-wall 272 (top)	2	52	3120	3500
abut. wing-wall 272 (bot)	3[2]	55	3880	
233 outside	18	53	3340	3800
233 inside	19	56	4180	
228 outside	14	50	2770	3100
228 inside	15	53	3340	
224 outside	27	57	4500	3900
224 inside	28	53	3340	
219 outside	29	55	3880	3900
219 inside	30	55	3880	
213 outside	35	53	3340	4100
213 inside	36	58	4830	
208 outside	37	51	2930	3400
208 inside	38	55	3880	
204 inside	43	57	4500	3600
204 outside	44	50	2770	
198 inside	45	54	3590	3600
198 outside		Could Not Reach		
178 outside spandrel wall	12	51	2930	2900

[1] Each hammer reading is average of seven individual tests. Individual values are available in Research Laboratory files for reference.

* Numbers in brackets indicate that a core was taken at this location. Core number is within the brackets.

TABLE 4

SWISS HAMMER READINGS AND ESTIMATED CONCRETE STRENGTH-CENTER SPAN
(Readings taken on the arch rib unless otherwise specified)

West Arch

Location, ft from N abutment line	Position Number	Average[1] Hammer Reading	Projected Cylinder Strength, psi	Avg. Proj. Cylinder Strength, psi
155 outside spandrel wall	53	54	3590	3600
148 inside (top)	48	47	2370	
148 inside (bot)	49	57	4500	3000
148 outside	50	48	2490	
140 inside	54	49	2630	2300
140 outside (top)	55	42	1850	
140 outside (bot)	56	43	1940	
134 inside	59	45	2140	2200
134 outside	60	46	2250	
127 outside[2]	66	40	1700	1700
127 inside		Could Not Reach		
123 Inside	67	47	2370	2300
123 Outside	68	45	2140	
117 inside	74	54	3590	3400
117 outside	75	52	3120	
108 inside	76	56	4180	3500
108 outside	77	50	2770	
109 outside spandrel wall	80[10]*	44	2040	2000

East Arch

Location, ft from N abutment line	Position Number	Average[1] Hammer Reading	Projected Cylinder Strength, psi	Avg. Proj. Cylinder Strength, psi
pilaster @ south pier	11	44	2040	2000
168 outside spandrel wall	10	54	3590	3600
164 outside spandrel wall	13	58	4830	4800
154 outside	51	54	3590	3600
154 inside	52	54	3590	
145 outside	57	42	1850	2500
145 inside	58	52	3120	
138 N outside[3]	61	44	2040	
138 S outside	62	54	3590	2400
138 inside	63	43	1940	
133 outside	64	40	1700	2000
133 inside	65	47	2370	
126 outside	71	43	1940	2100
126 S inside	72	48	2490	
125 N inside	73	45	2140	
114 outside	78	44	2040	2100
114 inside	79	46	2250	

1 Each hammer reading is average of seven individual tests. Individual values are available in Research Laboratory files for reference.

2 Salt afflicted area - See Figure 10.

3 Patched concrete.

* Numbers in brackets indicate that a core was taken at this location. Core number is within the brackets.

TABLE 5

SWISS HAMMER READINGS AND ESTIMATED CONCRETE STRENGTH-NORTH SPAN

(Readings taken on the arch rib unless otherwise specified)

West Arch					East Arch				
Location, ft from N abutment line	Position Number	Average[1] Hammer Reading	Projected Cylinder Strength, psi	Avg. Proj. Cylinder Strength, psi	Location, ft from N abutment line	Position Number	Average[1] Hammer Reading	Projected Cylinder Strength, psi	Avg. Proj. Cylinder Strength, psi
66 outside spandrel wall	81[11]*	56	4180	4200	72 inside	84	51	2930	2900
					72 outside		Could Not Reach		
66 inside	82	45	2140	2400	60 outside	88	49	2630	3700
66 outside	83	49	2630		60 inside	89	58	4830	
59 outside spandrel wall	85[12]	50	2770	2800	55 outside	90	41	1770	2000
					55 inside	91	45	2140	
54 inside	86	47	2370	2400	48 outside	96	55	3880	4000
54 outside	87	47	2370		48 inside	97	56	4180	
48 inside	92	45	2140	2000	43 outside	98	47	2370	2300
48 outside	93	41	1770		43 inside	99	46	2250	
42 outside	94	45	2140	2000	38 outside	104	49	2630	2900
42 inside	95	43	1940		38 inside	105	52	3120	
36 inside	100	43	1940	1700	34 outside	106	51	2930	2900
36 outside	101	35	1360		34 inside	107	51	2930	
33 inside	102	40	1700	1900	2 outside (top) spandrel wall	8	51	2930	2900
33 outside	103	44	2040						
28 inside	108	52	3120	3000	2 outside (bot) spandrel wall	9	56	4180	4200
28 outside	109	51	2930						
18 inside	110	43	1940	2400	-9 abut. wing-wall	5[4]	52	3120	3100
18 outside	111	51	2930						
18 outside spandrel wall	112[13]	45	2140	2100	-22 abut. wing-wall	6[5]	51	2930	2900
-14 abut. wing-wall	7[6]	53	3340	3300					

1 Each hammer reading is average of seven individual tests. Individual values are available in Research Laboratory files for reference.

* Numbers in brackets indicate that a core was taken at this location. Core number is within the brackets.