Bridge 2440

Third Avenue Bridge

Summary Engineering Report

Minneapolis, MN

March 5, 2015

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1 Introduction

MnDOT Bridge 2440 carries T.H. 65 (3rd Avenue South) over the Mississippi River in downtown Minneapolis. The bridge consists of six distinct units: the south abutment, two steel beam spans, five ribbed arch spans, two barrel arch spans, two prestressed concrete beam spans, and the north abutment. The arches are of the Melan arch construction which consists of a lattice work of steel I-beams, assembled approximating the shape of the arch, laid in a series near the undersurface of the arch. The concrete in the resulting steel-rib and concrete barrel serves a dual protective and structural purpose [1].

The bridge is located a short distance upstream of the St. Anthony Falls and spans a horseshoe dam which predates the bridge (Figure 1 in Appendix A). The bridge was built with a curved “S” shaped alignment to avoid failures of the limestone bedrock that occurred between 1869 and 1875 [2]. The bridge was constructed between 1915 and 1918 and has undergone one major concrete repair project in 1979 followed by a joint replacement project in 2004. The 1979 repair consisted of complete deck removal, new light standards, extending the spandrel columns, raising the roadway grade by five feet, replacing the approach spans at both ends of the bridge, cleaning and reinstallation of the 1939 railing, and pier repair.

2 Scope of the Report

In response to a routine underwater inspection in 2012 that identified severe concrete deterioration around the upstream third of the Pier 5 concrete footing seal, HDR was retained to perform an investigation involving a review of historical information surrounding the construction of both Bridge 2440 and nearby relevant features, and develop plans and specifications to repair the deterioration.

3 Geological Setting

The geology in the area consists of minor surface deposits containing glacial drift from the Pleistocene period, a bedrock formation consisting of Platteville Limestone, a thin layer of Glenwood shale, and the St. Peter Sandstone formation (Figures 2 & 3). At the end of the last ice age (approximately twelve thousand years ago), the falls were located near downtown St. Paul. The falls progressed upstream to the present location due to the natural erosion of the underlying sandstone formation (Figure 4). Plunging water from the falls created a deep plunge pool at the base of the falls which eroded the soft St. Peter Sandstone and undermined the overlying limestone bedrock forming the riverbed. The falls migrated upstream as limestone ledges broke off due to lack of support from below. As the falls moved upstream, the limestone layer became thinner, accelerating the migration. This migration was arrested in 1887 by the construction of the spillway [3].

The Platteville Limestone formation at the St. Anthony Falls is wedge shaped. It thins from approximately 13 feet thick at the falls to disappear entirely approximately one-third of the way through Nicollet Island. At Pier 5 the limestone bedrock is nominally 9 feet thick (see page 22 of 1968 HNTB Report [2] in Appendix B).

4 Relevant Site History

The initial development of the City of Minneapolis is due in large part to the availability of hydro-mechanical and hydro-electric power from the relatively large Mississippi flows and hydraulic head at the falls. Initially the primary operations were timber sawmills where trees cut from the forests to the north were floated downstream for processing. As the forests became depleted, operations shifted to grain milling. Minneapolis was the largest grain milling center in the world for many decades due in large part to the falls. As electrical power became more available, the grain milling operations gradually shut down. The electrical hydropower plant is the last major vestige of the area’s industrial operations.

In 1837 a treaty was established between the Dakota and U.S. government which allowed development of the east bank (north side of the river in this area). In order to direct the logs to the east bank, a rock filled timber crib structure was constructed upstream into the river in 1849. This is now the north side of the horseshoe dam. The south half of the dam was added in 1856 [3]. With the construction of the south side of the horseshoe dam, a large number of grain milling operations established near and beyond the current lock location. The 1800’s era rock filled timber dam was eventually capped with concrete and the original structure remains largely in place (Figures 5 & 6). The downstream edge of Pier 5’s concrete footing seal lies immediately adjacent the horseshoe dam (Figure 7).

Typical hydro-mechanical installations near the St. Anthony Falls consisted of a nearly horizontal tunnel mined into the easily excavated St. Peter Sandstone and exposed at the downstream bank. Near the source of water, there was a vertical drop shaft extending through the hard Platteville limestone. A large number of such old tunnels remain and are located further downstream of the 3rd Avenue Bridge and seepage cutoff with five or six tunnels under the old Main Street hydro project on the north side and a number of other tunnels beneath the old mills on the south side.

4.1 Eastman Tunnel

William W. Eastman decided to develop a tunnel in the late 1860's; the construction of which nearly resulted in the complete loss of the St. Anthony Falls. In order to develop a saw mill using water power at the site, he started the tunnel in the sandstone at the downstream end but planned to extend the tunnel all the way to Nicollet Island. He was not aware that the Platteville Limestone thickness decreased as it neared the island and that it wouldn't have sufficient strength to withstand the weight of the water above. As his crews approached Nicollet Island the tunnel collapsed and an uncontrolled flow of water resulted (Figure 8). The failure occurred rapidly and washed out a large channel, much wider than the original tunnel. Eventually both the tunnel and the collapse were filled. Initial reports indicated desperate attempts to block the washout with rock, logs etc. Later earthen dams were constructed in order to block off and repair the washed out area. The exact material used to fill in the collapse is unknown. The remains of the Eastman tunnel are located between Piers 5 and 6 (Figure 9).

Between 1874 and 1876 an underground seepage cutoff wall was constructed by the U.S. Army Corps of Engineers. The concrete "government cutoff wall" was built into the St. Peter Sandstone 35-50 feet deep and between 4 and 11 feet wide. At 1900 feet long, the wall crosses the entire width of the river (Figures 9 & 10). The purpose of this wall was to prevent further erosion of the sandstone layer due to the collapse of the Eastman tunnel.

4.2 Saint Anthony Falls Spillway Protection

The first protection of St. Anthony Falls was constructed in 1887 and consisted of a rock filled timber crib support structure covered with a wood apron. In the late 1950's and early 1960's the Northern States Power Company (NSP) replaced the wood and rock filled structure with a concrete spillway and sheet pile downstream toe (Figures 11 & 12).

In conjunction with other work efforts in the late 1980's, four soil borings were taken and piezometers were installed by NSP and located approximately along the centerline of the horseshoe dam. Two piezometers were located upstream of the "government cutoff wall" and two piezometers were located downstream. The furthest upstream is directly to the south of Pier 5 (slightly downstream and between pier 3 and pier 4). The boring operation was used to establish geologic conditions and measure the seepage and pressure conditions as a part of the Federal Energy Regulatory Commission (FERC) dam safety program. To date no excess pressure or seepage issues have been recorded.

4.3 Upper and Lower St. Anthony Falls Lock and Dam

In the late 1950's the milling operations in Minneapolis had all but ceased, resulting in an economic downturn for the area. Hoping to attract more business to the city, a decision was made to construct the Upper St. Anthony Lock (Figure 13). The construction included the locks, elimination of the milling operations water supplies and the construction of cellular sheet pile dolphins to protect against vessels going over the horseshoe dam and spillway. The cellular sheet pile is supported by the bedrock. NSP has also installed a cable between the dolphins for additional small boater safety. The upper and lower lock construction was completed in 1963.

5 Pier 5 Available Data

5.1 Construction

Originally named the St. Anthony Falls Bridge, MnDOT Bridge 2440 was built between 1915 and 1918. The final alignment was chosen to avoid constructing pier foundations on the limestone bedrock failures caused by the 1869 Eastman tunnel collapse (Figure 4). Two 165-foot tall cableways were constructed on each bank of the river to facilitate material delivery and placement for the bridge's construction.

There appears to have been two different cofferdam designs based on the construction plan and corroborated by construction photos. A timber design was provided for the shallower piers (Piers 3 and 4) and a steel sheeting cofferdam for the deeper Piers 1, 2, 5, 6, 7, and 8 (Figures 14-16). In each case, the cofferdams were set directly on the bedrock; the plans do not indicate that the foundation was subcut into the bedrock. The cofferdam details indicate timber bracing was constructed inside the structure to resist the hydrostatic pressure. There is a high likelihood that this bracing remained within the footing concrete at the conclusion of construction.

Sediment was removed mechanically from within the cofferdam through the use of orange-peel buckets (Figures 17 & 18). Photos show laborers cleaning out cofferdams with hand shovels as well. Evidence suggests that removing the sediment from within the cofferdam was not completed effectively. Figure 19 identifies concrete erosion of Pier 4 soon after construction which is likely related to foreign deleterious material mixing with the concrete at the time of casting.

Sandbags were placed outside the cofferdam around the bottom and supplemented with pumping to control minor infiltration. Where pumping was ineffective, a concrete pad was cast underwater up to 6 feet thick. This occurred at Piers 1, 2, 5, and 7 [4]. This pad utilized a different concrete mix ratio (1:2:4) than the piers cast "in the dry" (1:3:6). The 1:2:4 concrete mix would have provided higher strength and reduced permeability (a common engineering practice at the time). Concrete was batched in units of 1 cubic yard and placed with a bucket that held two batches. There is no evidence in the original plan that the foundations were reinforced with steel. Construction photos show minimal steel reinforcing in the general area of the arch spring line (Figures 20 & 21).

Once the pier footings were constructed, additional pier concrete was placed up to an elevation above the waterline. A section of arch lattice reinforcement (part of the Melan system) used to anchor the arch into the center of the pier was cast with a final pour near the spring line. Pier construction was completed in less than two years with no cessation during the winter months except when temperatures dropped below 0°F. Form and material temperatures were closely monitored and controlled using tarps and heaters [4].

The Pier 5 foundation is a large massive structure with a width of 37 feet, a height of 31 feet, and a length of 114 feet. Fully supported, the bearing pressure is approximately 6600 psf. For reasons indicated in the report, it is assumed the foundation is a plain non-reinforced concrete structure.

5.2 1968 Repair Project and Associated Report

In 1968, after over 50 years of service, a major repair project was programed. The rehabilitation, which took place between 1979 and 1980, included new abutments, new deck, new spandrel columns and caps to accommodate a 5-foot raise in grade, refurbished traffic rail, and more functional pedestrian railings.

Six concrete/bedrock cores were taken in and around Pier 5 during the 1968 bridge inspection (Figure 22). The complete report is included in Appendix B. Two of the cores (3-4 and 3-5) included material from the concrete footing. The log for core 3-4 indicated that decayed wood was hit at the concrete/bedrock interface (this core is in the approximate location of the current void). Core log 3-5 stated “lower portions badly fractured and easier drilling – possibly leaner concrete”.

The logs show the limestone bedrock being relatively level, with the top of bedrock varying from elevation 782.6 to 784.6. Cores taken along the side of the pier closest to the horseshoe dam (logs 3-6, 3-7, 3-8 and 3-9) show loose sand material (it was not sampled). This is consistent with the bed loam sediment found in the area which typically consists of clean “quartz like” particles with occasional debris such as trees or branches.

A repair detail from a 1961 maintenance operation responding to deteriorated concrete in the footings of Piers 1 and 2 was also included in the report (Figure 23). The suggested repairs indicate additional reinforced concrete may have been added to armor or protect the nose.

5.3 1993 Bridge Scour Investigation

Bridge 2440 has many distinguishing characteristics that make its scour evaluation unique. The bridge piers straddle an unusual horseshoe shaped dam and a U.S. Army Corps of Engineers operated river navigation lock. These make for complex geometry and flow patterns near several of the piers. In addition, all of the piers are founded on bedrock [3]. A 1993 scour study concluded that there was very little likelihood that pier stability would be compromised for flows up to and including the 500-year flood event. The limestone upon which the piers rest shows very little tendency towards erosion, even where it has continuously been exposed to rapidly flowing water for 75 years [3].

5.4 Underwater Inspections

The earliest underwater inspection report obtained by HDR indicated small penetrations of the concrete footing seal at the upstream nose of Pier 5 of up to 3 feet. Table 1 summarizes the major underwater inspection findings at Pier 5 since 1992. The inspection reports show that a void has been present since at least 1992 and has gradually increased in size at each inspection.

In November 2012, an underwater inspection commissioned by the Minnesota Department of Transportation was conducted by Collins Engineers, Inc. The inspection report indicated that a concrete void located in the upstream nose of the Pier 5 footing had "increased significantly compared to what was reported during the 2004 and 2008 inspections." Typical horizontal penetrations of the cavity ranged from 6 feet to 14 feet with vertical dimensions between 1 and 3 feet.

Table 1: Summary of Pier 5 Void Measurements from Underwater Inspections

Inspection Date	Pier 5 Findings	Inspection Sketches
1992	Undermining pockets at the upstream nose with maximum penetration under the footing of 3 feet.	Not Available

Table 1 (continued): Summary of Pier 5 Void Measurements from Underwater Inspections

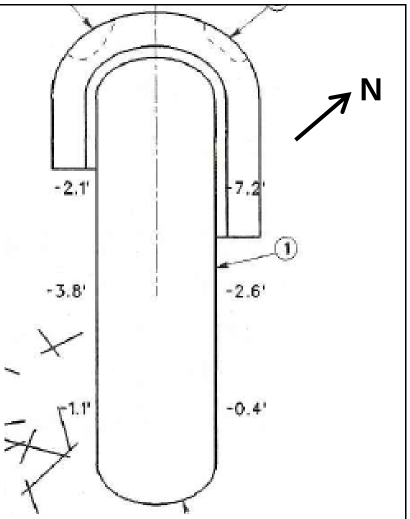
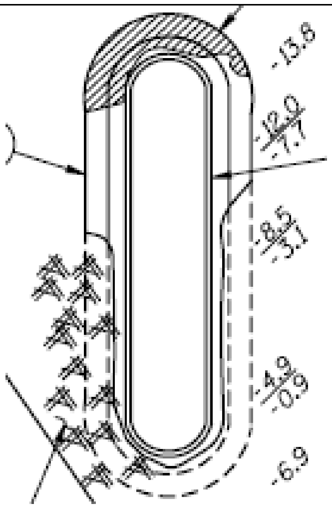
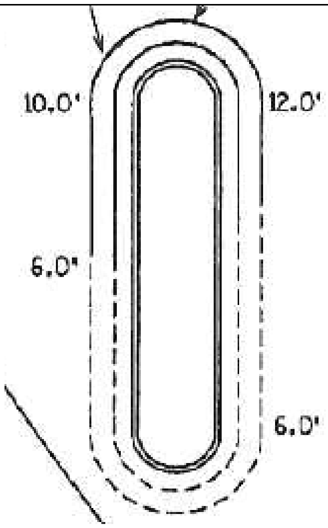
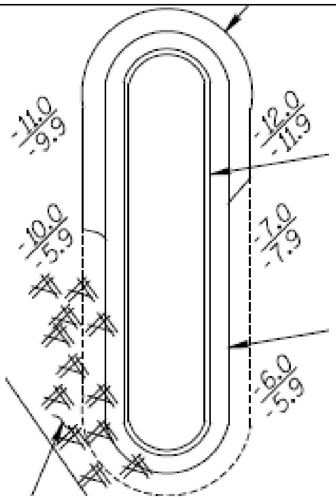
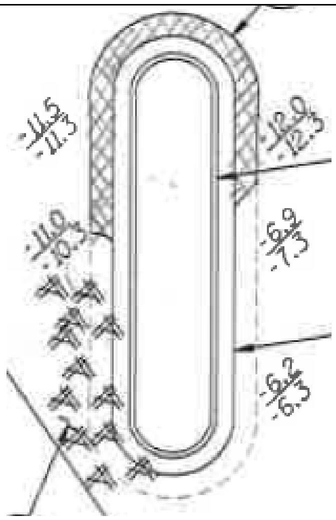
1996	Concrete and steel reinforcing debris on channel bottom around upstream and north side of pier. Vertical face of footing bell exposed as shown with undermining pockets at south and north sides of upstream nose. South cavity was 8' wide x 2.5' high x 6.5' deep. North cavity was 8' wide x 2.5' high x 3' deep. Exposed vertical height of footing was 10' including the footing bevel.	
2000	The upstream nose of Pier 5 was undermined from the start of the nose on the upstream east side to the centerline of the upstream arch on the west side. The void was 1 to 3 feet high with more than 4 feet of penetration.	
2004	Pier 5 has a void 3 feet high, 6 feet long and up to 18 inches of penetration on the west side near the upstream nose. There is also a void of 6 inches x 6 inches x 18 inches on the upstream nose.	

Table 1 (continued): Summary of Pier 5 Void Measurements from Underwater Inspections

2008	The footing at Pier 5 was exposed around the upstream 1/3 of the pier with typical vertical exposure of 3 to 4 feet. The footing was also undermined (undercut) at the upstream nose of the pier with vertical cavity height of 2 feet and typical penetration of 2 to 3 feet deep. The concrete of the footing at the upstream end exhibited deterioration with typical penetrations of 2 feet and maximum penetration of 3 feet.	
2012	The footing at Pier 5 was exposed down to the bedrock around the upstream 1/3 of the pier. The footing was also undermined (undercut) at the upstream nose of the pier. The concrete of the footing at the upstream end exhibited deterioration and a cavity into the footing with typical penetrations of 6 feet and maximum penetration of approximately 14 feet. The vertical cavity height was typically 1 foot with a maximum of 3 feet.	

6 Subsurface Exploration

During the winter of 2014, an underwater inspection using Blue View Imaging Sonar and a geotechnical exploration by AET was performed on Pier 5. The extent of work was limited due to winter conditions and safety concerns. The AET subsurface exploration report is included in Appendix E and sonar screen shots by Blue View are in Appendix F.

AET's subsurface exploration consisted of geotechnical/concrete investigations similar to what had been performed in 1968 at Pier 5 (see Appendix B). The geotechnical information was unchanged and the concrete data was substantially the same. The core was performed from the bridge deck, through the lower portion of the pier concrete footing, the Platteville Limestone, Glenwood Shale, and into the St. Peter Sandstone. A petrographic analysis was performed on representative concrete samples and a pressure transducer was placed in the St. Peter Sandstone to measure any uplift pressures.

The results of the coring investigation indicate that the concrete is fractured. A void was also present as the core approached the bedrock interface. The Platteville Limestone and the Glenwood Shale bedrock had a thickness of approximately 8.7 feet and 1.8 feet respectfully at the core location (see location drawing in Appendix E).

The petrographic analysis of the concrete sample indicated good overall concrete however the concrete was not air entrained and the coarse aggregate was large. The concrete had some crystalline deposits suggesting water movement through the concrete and there were also a few alkali silica reactive quartzite particles, both of these conditions were considered minor and innocuous.

Pressure measurements in the lower St. Peter Sandstone were performed to establish if uplift pressure conditions existed. The potential exists for such conditions due to the unknowns related to the nearby Eastman tunnel collapse and the fact the Platteville Limestone and Glenwood Shale ends a short distance upstream. After the coring operation, a pressure transducer was placed in the St. Peter Sandstone at elevation 768.5 and grouted in place (normal river elevation is 798.8). The transducer did not record any water pressure indicating it was above the phreatic surface. This indicates that the uplift pressure in the St. Peter Sandstone is not a significant factor and that uplift for a cofferdam should not be a significant design issue.

The results of the Blue View sonar inspection are included in Appendix F. They indicate the approximate extent of Pier 5 and the location of the deterioration, and are in good general agreement with the November 2012 Underwater Inspection.

7 Potential Causes of Pier 5 Deterioration

The original plans do not indicate that the pier foundations were sub cut into the bedrock. In addition, no detailed information is available which demonstrates how well the bedrock was cleaned off or the cofferdam was pumped dry prior to casting. It is likely that the deterioration was initiated due to poor construction quality control resulting from placing concrete on a surface that was not completely cleaned of sediment.

Based on the information available at the time of this report the deterioration in the footing of Pier 5 is most likely due to sediment at the bottom of the cofferdam that was not adequately removed prior to placement of the concrete footing seal. A layer or “ribbon” of sediment could have been left on the limestone bedrock. Timbers, used to support the cofferdam, may have also been left in place creating further potential for future voids. A thin "shell" of concrete may have then encapsulated the defect which was gradually washed away over the years leaving a cavity. Available records indicate that Pier 5 was not completely dewatered when the concrete seal was cast [4]. It was also excavated primarily with an orange peel bucket and the rock in the cofferdam was cleaned by divers with water jets. This method would have removed large portions of sediment and debris but left considerable amounts behind.

Other possible, although unlikely, causes of the deterioration are:

- **Scour of Bedrock** – The bedrock, as discussed in the 1993 scour report, is highly resistant to scour. Pier 5 is located in an area of relatively low flow velocities due to its location immediately above the dam in water depths around 16 feet. Other piers are located on bedrock that has been exposed to much more aggressive scour conditions with higher flow velocities without experiencing scour or the type of material loss noted at Pier 5. Also, as reported, the cavity appears to be in the concrete and not the bedrock.

- **Subsidence of the Bedrock** – Any subsidence would have likely impacted the whole structure. For a rigid concrete multiple arch bridge, it would be expected that subsidence would cause structural distress in the form of superstructure cracking. No such distress has been indicated.

- **Washout of Concrete** – The concrete cores taken by AET (Appendix E) indicate that the concrete near the bedrock interface is fractured and of poor quality. This is consistent with the poor construction quality control that presumably occurred during placement of the concrete into the water. However, Pier 5 is a relatively massive pier and, with fractured interlocking pieces of concrete, it would require a lot of force to cause the long, low undermining observed. Since the pier is submerged in relatively deep water, there isn't a lot of differential pressure, and the water velocities are relatively low. This is not a likely cause of the deterioration.

8 Conclusion

It is the conclusion of the authors of this report that certain contemporary construction controls and design standards were not employed during the construction of Bridge 2440 that could have led to its advanced concrete deterioration. These poor construction methods include the casting of concrete into non-potable water, the lack of steel reinforcement, and possibly not cleaning out all sand, sediment, or timbers in the cofferdam before pouring concrete. Thus, it is doubtful the observed deterioration is due to scour; rather, it is most likely due to poor quality control and the difficulty in dewatering and cleaning the cofferdam during the time of original construction.

9 Recommendation

Based on the available information, it is the author's recommendation that the damaged area be encapsulated with reinforced concrete and that the void be cleaned and pressure grouted. The work could be performed with a cofferdam or underwater with divers. However, based on the difficulty, cost, and risks associated with installation of a cofferdam, performing the work underwater is recommended. This may require some special construction quality control procedures.

Cleaning the void and the immediate area around the pier may require displacement of the river bottom material. Available information indicates that the material consists of logs, sand, and gravel sediment. The removal of all wood material is recommended and the sand and gravel material should be able to be displaced with minimal turbulence or "plume". The sand and gravel material is limited in area and will settle out within a few minutes or short distance from the pier.

10 References

1. Minnesota Historic Property Record, "Third Avenue Bridge" *MHPR Identification Number HE-MPC-0165*. See Appendix C.

2. HNTB Bridge Inspection, *Third Avenue Bridge over the Mississippi River Engineering Report*, November 1968. See Appendix B.

3. Bridge Scour Investigation, "Bridge 2440 Saint Anthony Falls Bridge (3rd Avenue) Trunk Highway 65 over the Mississippi River at Minneapolis, Minnesota", *MnDOT Consultant Agreement No. 66083*, Work Order Assignment No. 7, 1993.

4. Richter, A. M., "A 2,223-Ft. Concrete-Arch Bridge Built on Reverse Curve", *Engineering News*, December 30, 1915, Vol. 74, No. 27, p. 1268-1273. See Appendix D.

5. Wikipedia Contributors, "Saint Anthony Falls," *Wikipedia, The Free Encyclopedia*, http://en.wikipedia.org/w/index.php?title=Saint_Anthony_Falls&oldid=595785444 (accessed April 28, 2014).

6. A History of Saint Anthony Falls, National Center for Earth-Surface Dynamics, http://www.esci.umn.edu/courses/1001/1001_kirkby/SAFL/WEBSITEPAGES/5.html (accessed April 28, 2014).

Other Sources of Information

- Conversations with WSB inspectors
- COE Lock and Dam plans
- Original construction photos
- 1916 Bridge plans
- 1979 Repair plans
- 2004 Repair plans
- 1996, 2000, 2004, 2008, 2012 Underwater inspection reports

A

Appendix A

Report Figures

Figure 1: Aerial view of Bridge 2440 and the horseshoe dam.

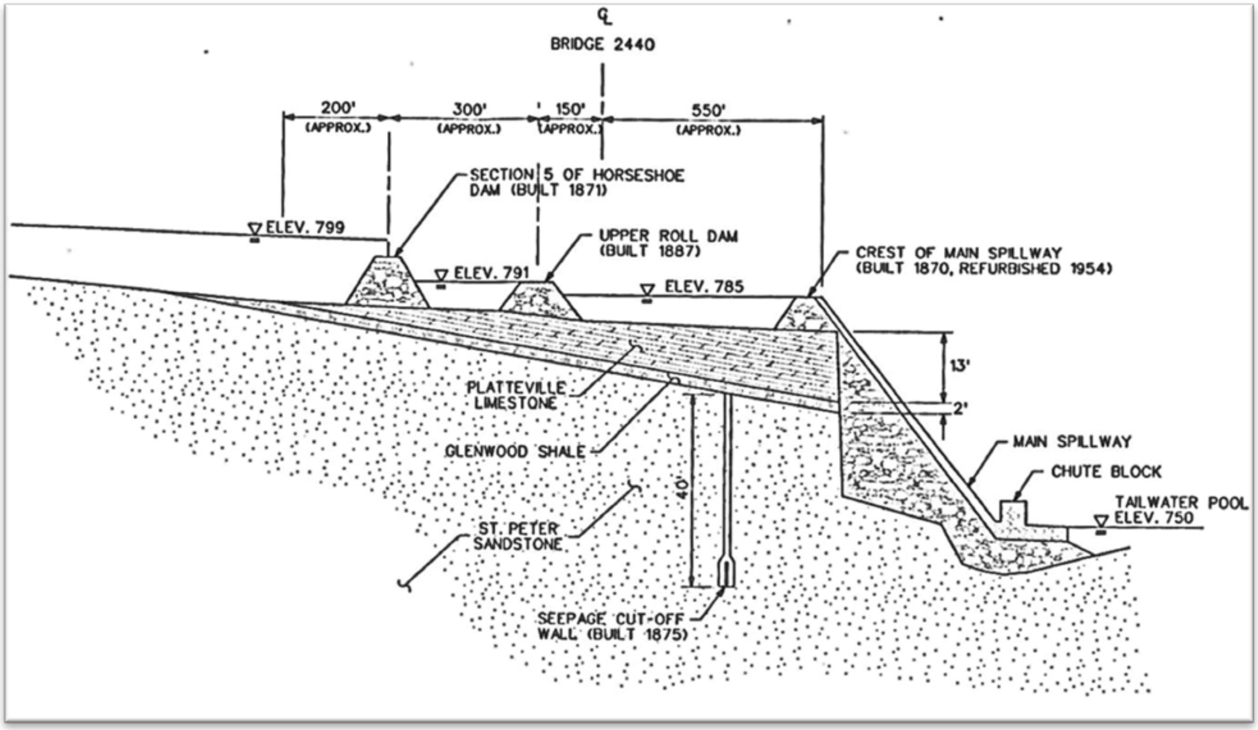


Figure 2: General geologic profile [3].

Figure 3: View of the Saint Anthony Falls before the spillway was constructed showing the limestone ledges [6].

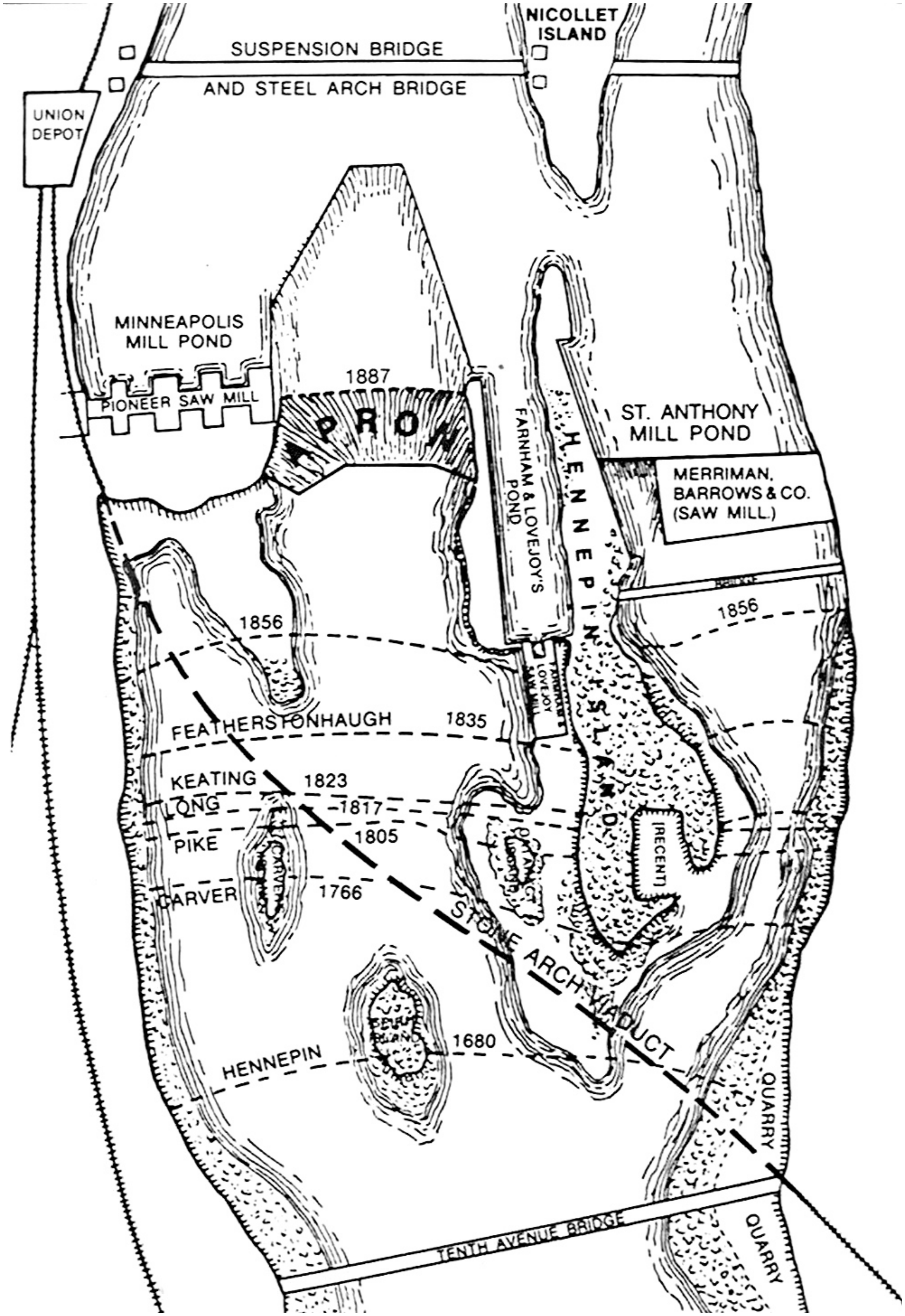


Figure 4: Recession of the falls between 1680 and 1887 [5].

Figure 5: Upper Saint Anthony Falls' first spillway ca. 1896 looking (approx.) north-northeast from current Upper St. Anthony Falls Lock and Dam. The International Stock Food Company (Exposition Building) can be seen in the background.

Figure 6: Aerial view of Saint Anthony Falls ca. 1950 before the Upper Lock and Dam was constructed. looking north

Figure 7: Pier 5 cofferdam and horseshoe dam.

Figure 8: Left; Photo of tunnel collapse of 1869 on Hennepin Island near St. Anthony Falls [5]. Right; A portion of the tunnel collapse showing the East Side Platform Sawmills and Hennepin Island in the background [6].

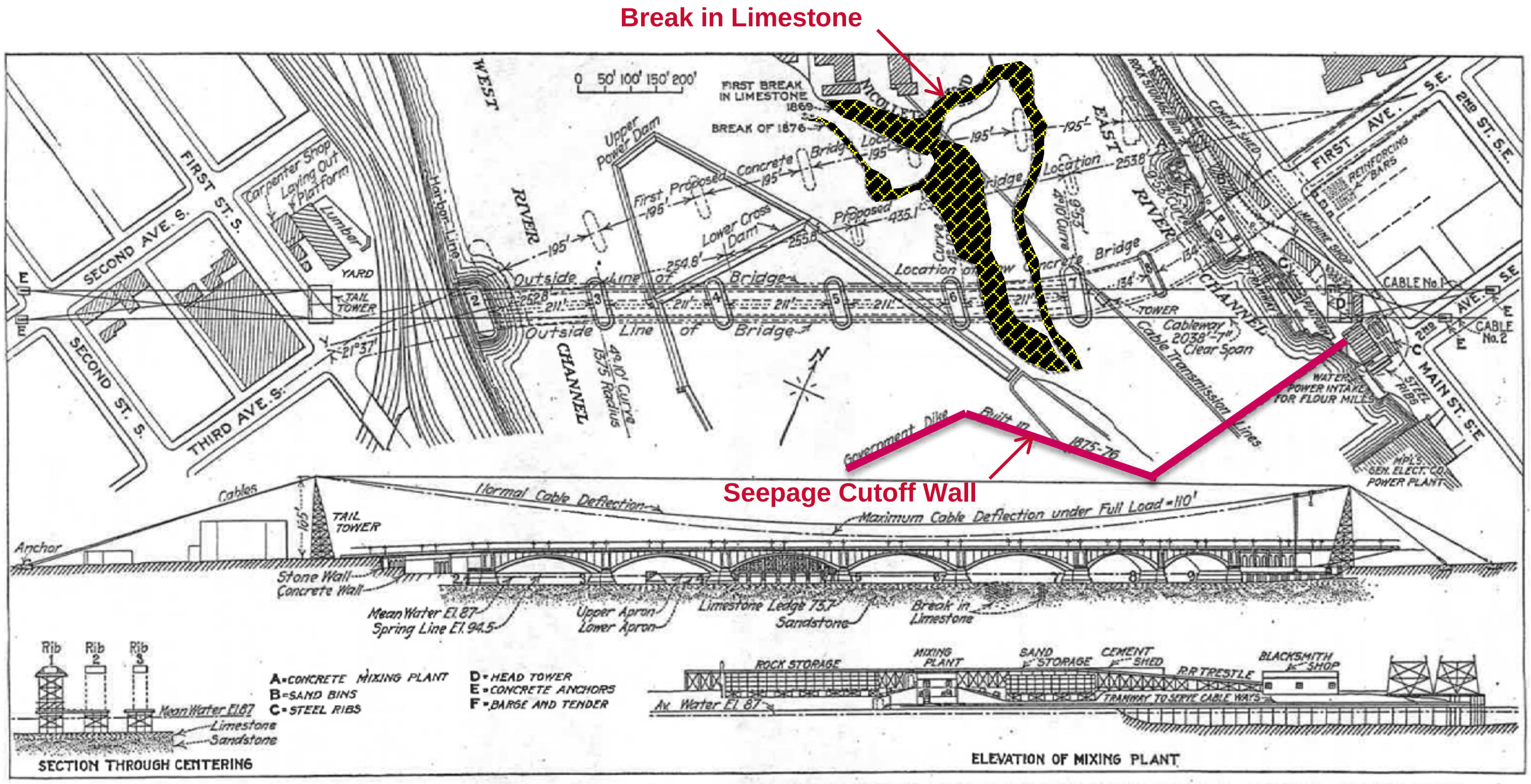


FIG. 2. PLAN AND ELEVATION OF THE THIRD AVE. REINFORCED-CONCRETE ARCH BRIDGE OVER THE MISSISSIPPI RIVER AT MINNEAPOLIS, MINN.

Figure 9 Location of break in limestone due to Eastman Tunnel and seepage cutoff wall [4].

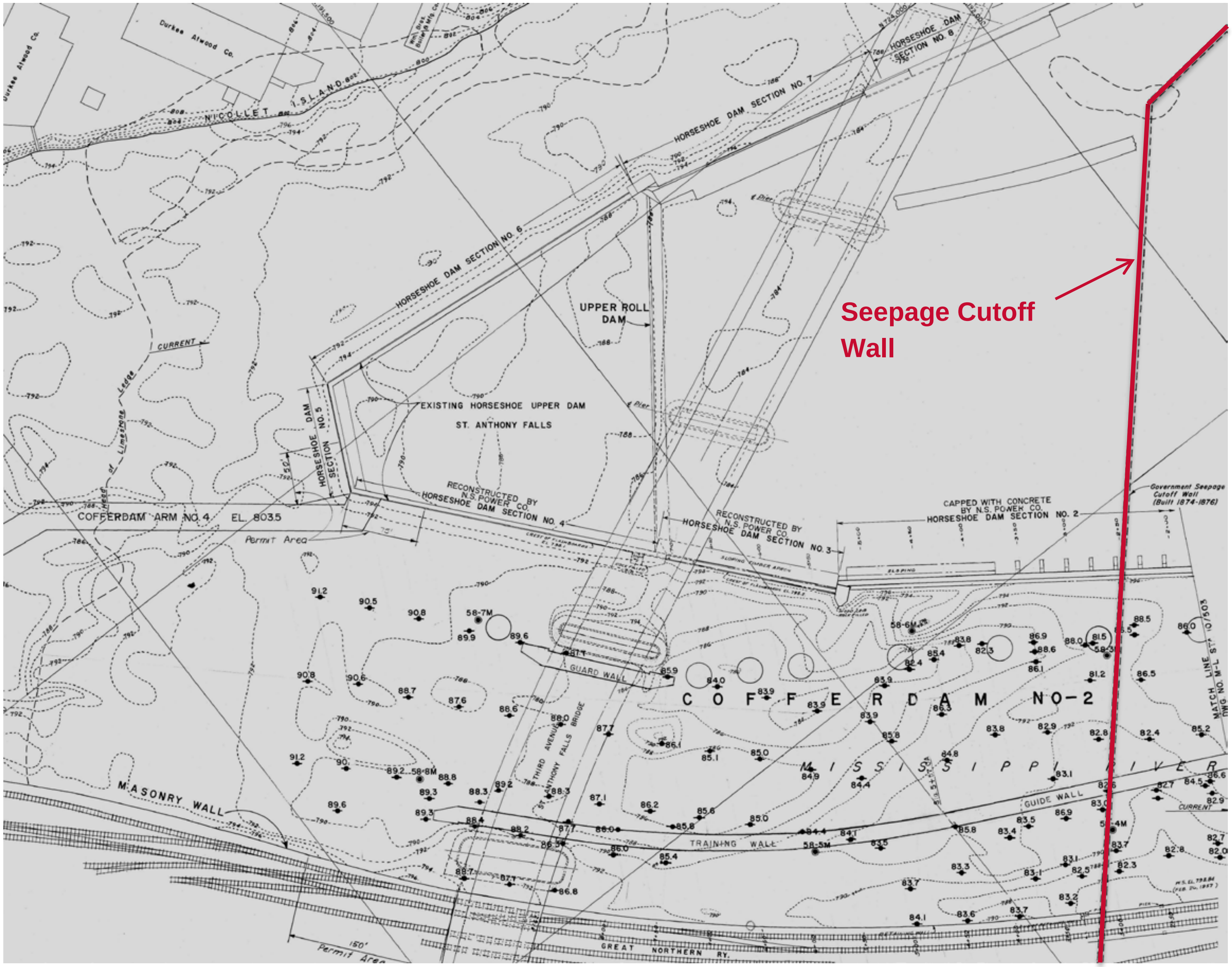


Figure 10: Location of cutoff wall relative to navigation channel.

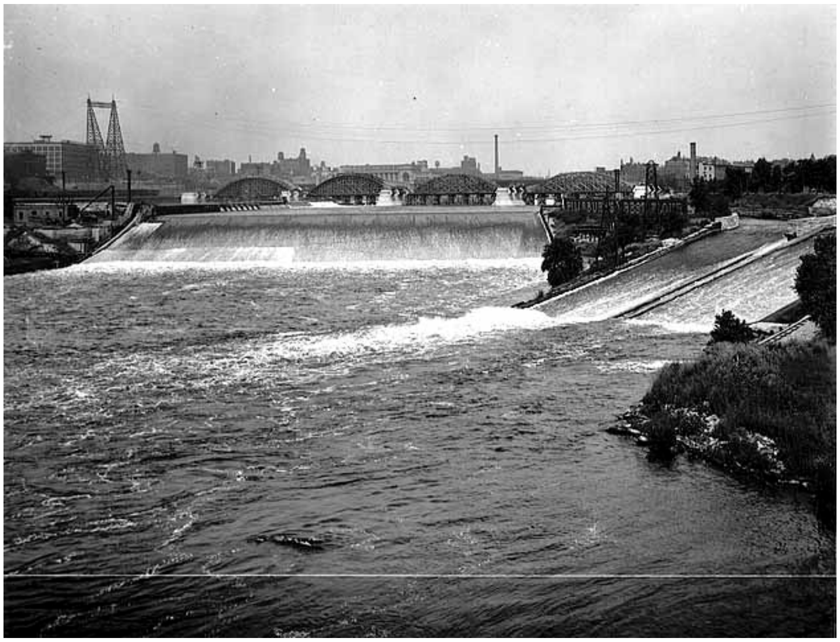


Figure 11: Spillway during construction of Br 2440. One of the construction towers can be seen at the top left.

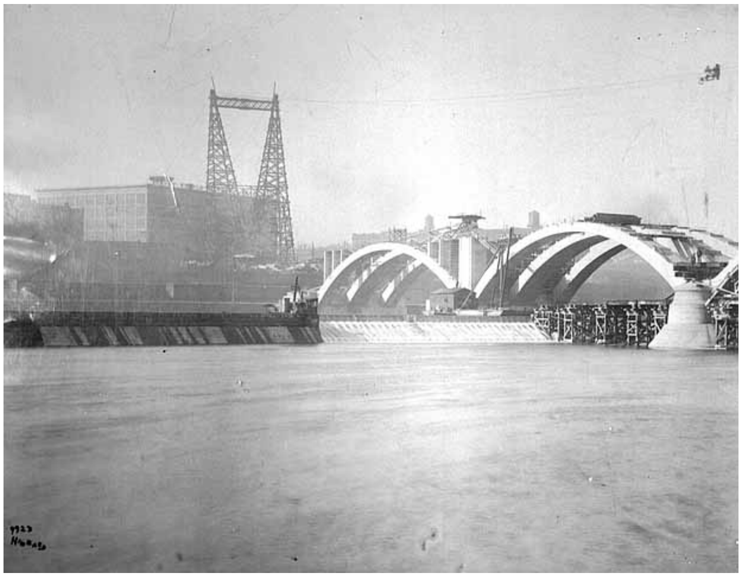


Figure 12: Horseshoe dam and arch ribs during construction.

Figure 13: Saint Anthony Falls area in the 1950's showing the proposed locations of the lock and dam.

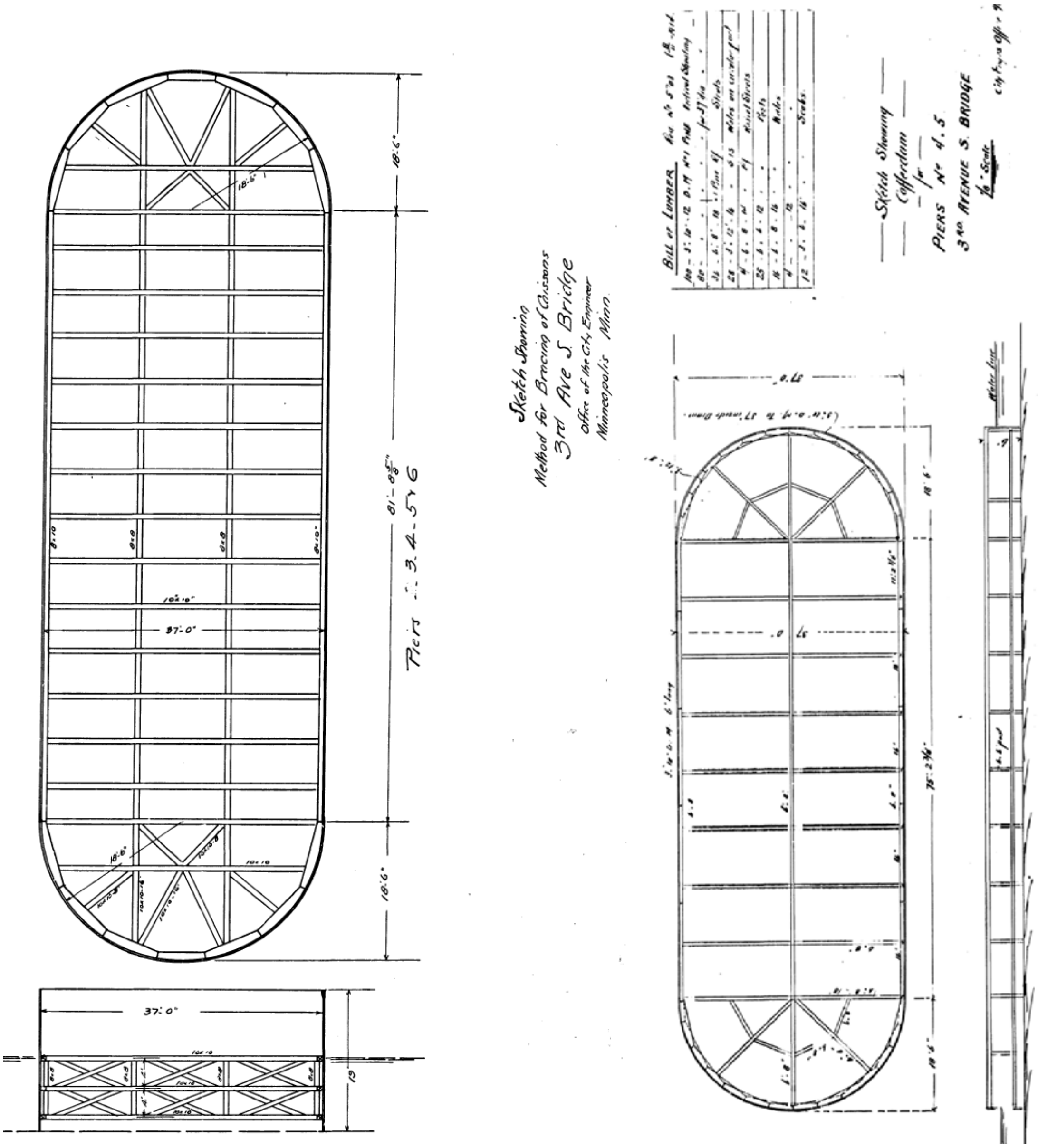


Figure 14: Left) Deep pier cofferdam detail. Note that Pier 2 (originally 1) has been erased and that Piers 3 (4) and 4(5) are erroneously included on this detail. Right) Shallow pier cofferdam detail. This detail refers to Piers 3 (2) and 4(3).

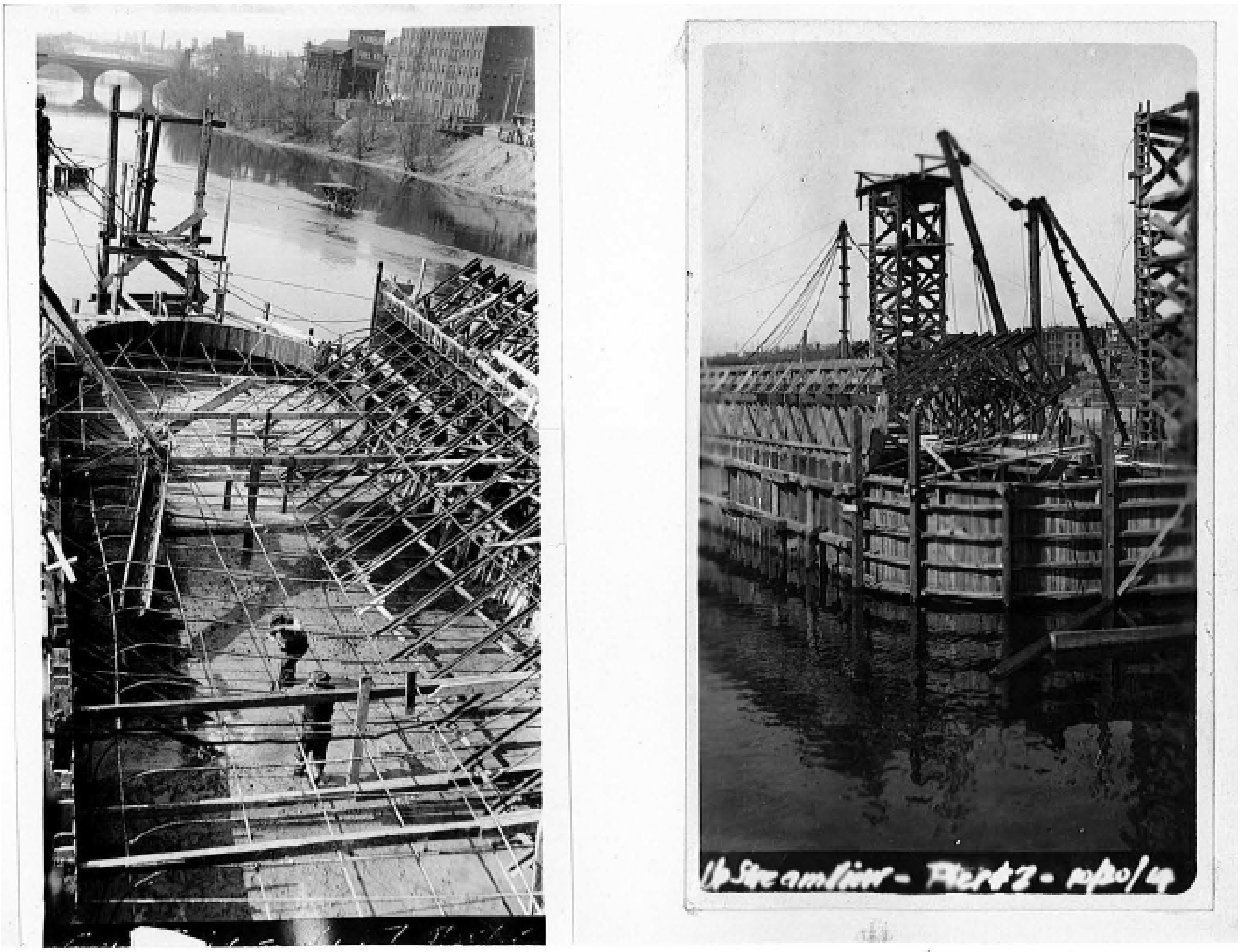


Figure 15: Shallower timber cofferdam during construction.

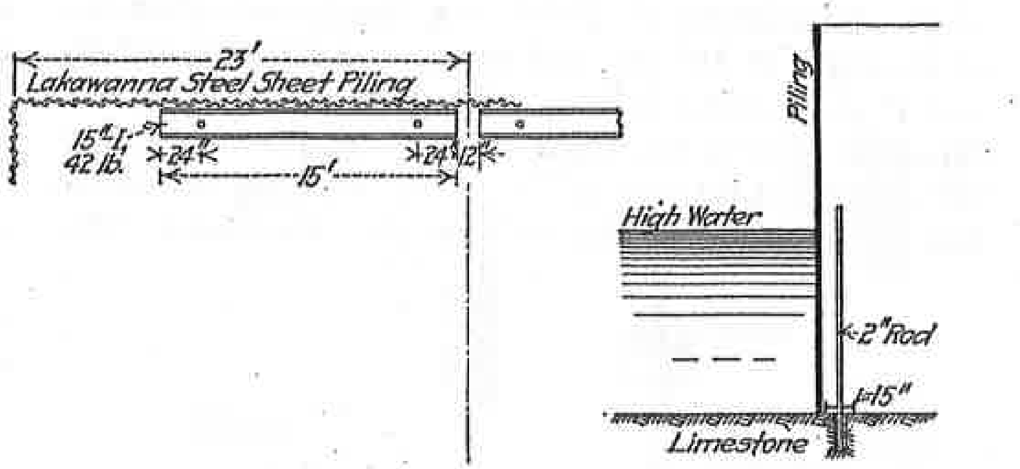


Figure 16: Deep pier cofferdam sheeting details.

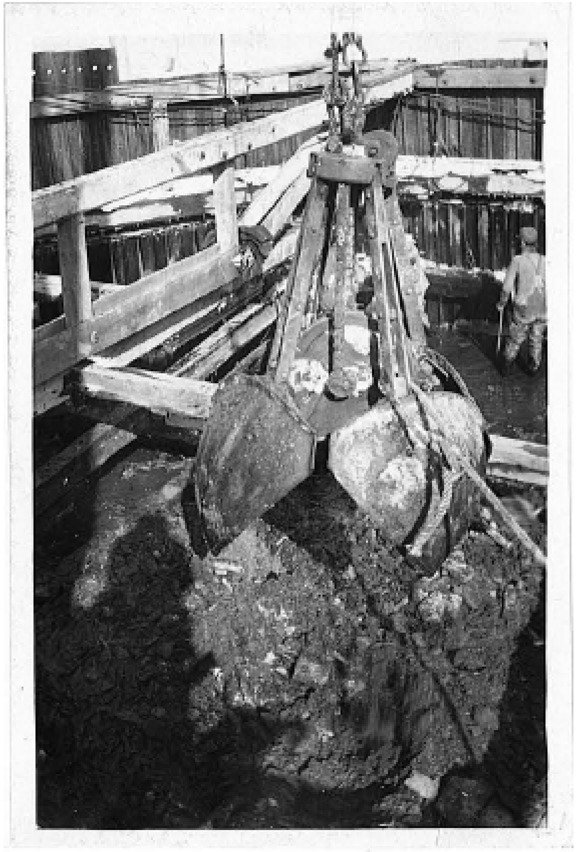


Figure 17: Removing sediment from cofferdam with orange peel bucket.

Figure 18: Pier 5 cofferdam being cleared of debris and silt.

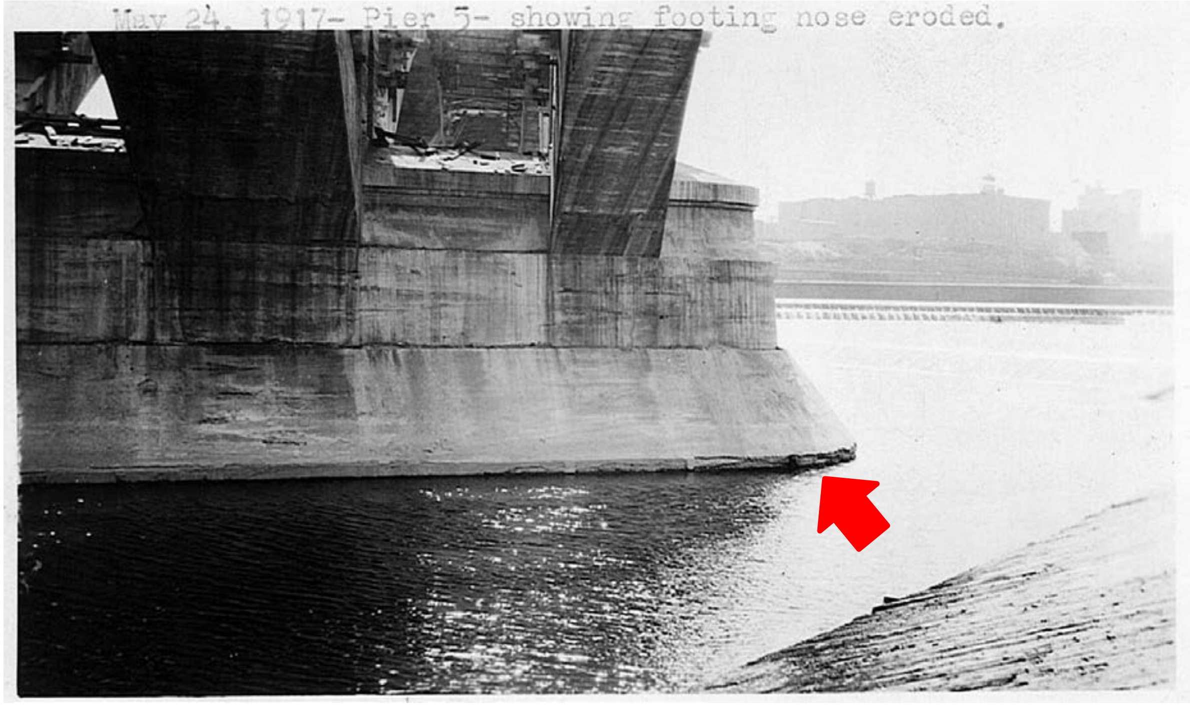


Figure 19: Pier 4 concrete deterioration soon after construction.

Figure 20: Reinforcement where the steel in the Melan arches connects with the pier.

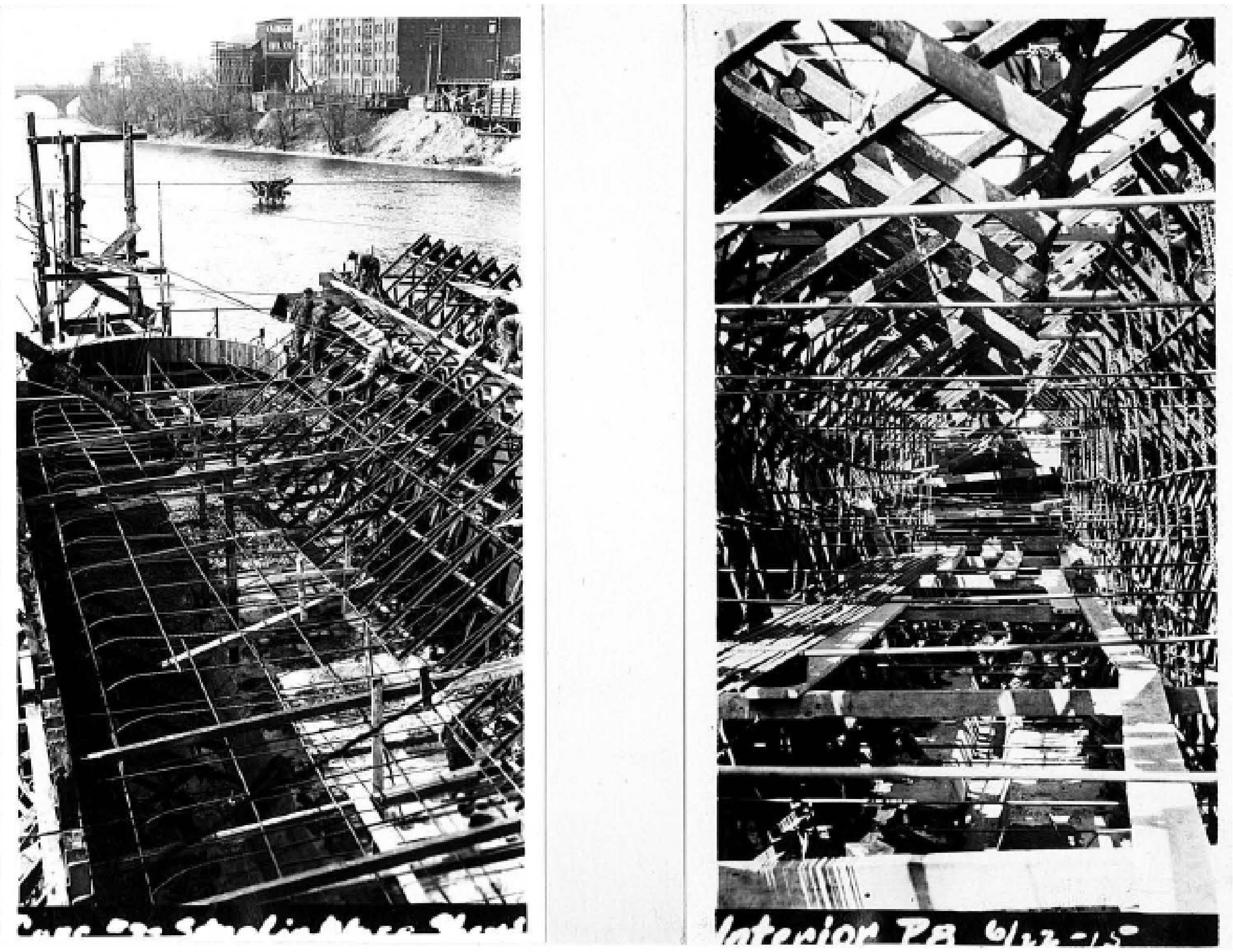


Figure 21: View of reinforcement in the spring line area of pier.

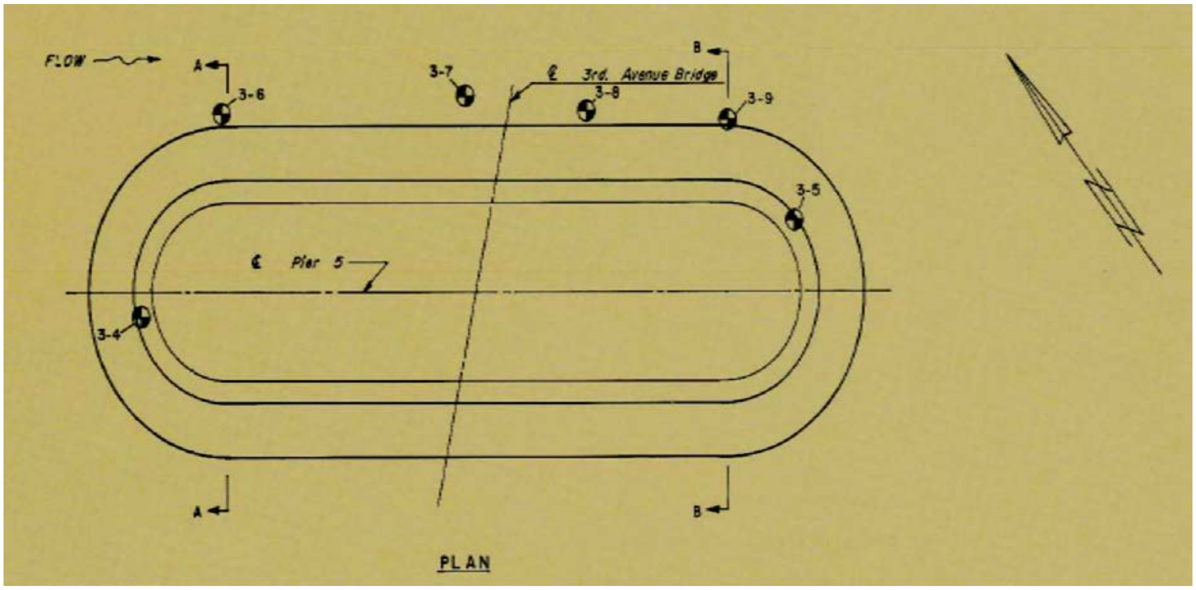


Figure 22: Locations of Pier 5 cores [2].

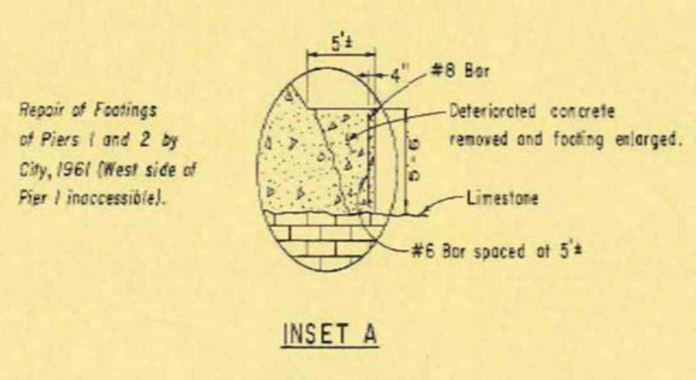


Figure 23: Pier 1 and 2 repair detail [2].

B

Appendix B

1968 Bridge Inspection Report

Minnesota Department of Highways

1858

BRIDGE INSPECTION

THIRD AVENUE BRIDGE

Over the Mississippi River, Minneapolis

Engineering Report

NOVEMBER 1968

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS
HNTB

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PART III ENGINEERING GEOLOGY - FOUNDATION INVESTIGATIONS

PART IV REBUILT STRUCTURE

LIST OF ILLUSTRATIONS

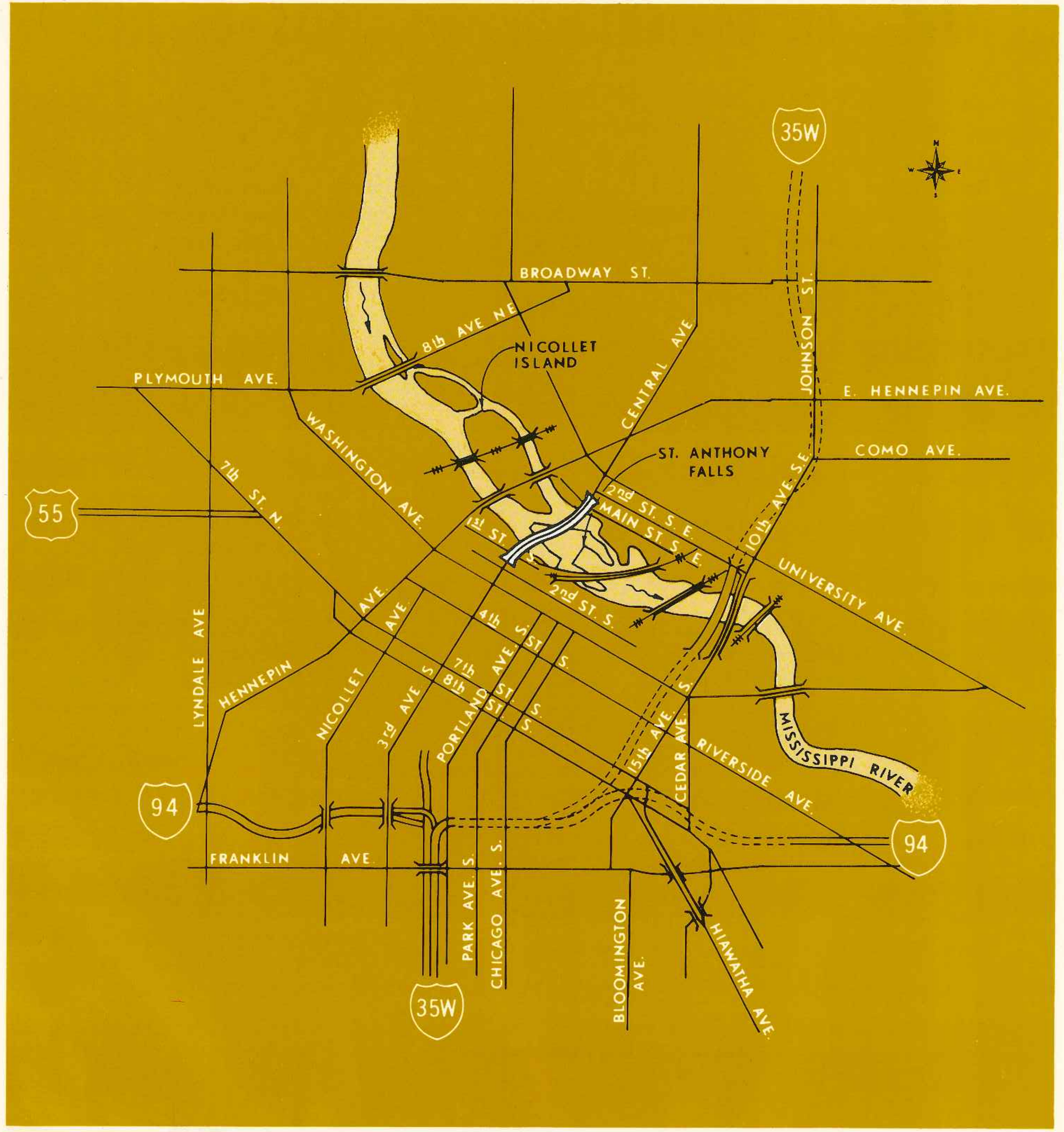


Figure 1

VICINITY MAP
OF THIRD AVENUE BRIDGE

Minnesota Department of Highways

THIRD AVENUE BRIDGE, MINNEAPOLIS

BRIDGE INSPECTION

PART I GENERAL DESCRIPTION

INTRODUCTION

St. Anthony Falls, the head of the navigable length of the Mississippi River and an abundant source of waterpower, was the main factor in the location and initial development of the City of Minneapolis. As the City expanded and the need for suitable links joining both sides of the river became increasingly important, an extensive bridge building program was begun.

The increase in the rail, streetcar, and motor vehicular traffic at the turn of the century and the geographic factors such as the distances to be spanned, the high river banks on both sides, the presence of sound founding rock and availability of good construction materials all suggested the use of reinforced concrete arch bridges. In addition, the technological advances in the design and construction of multiple arch bridges further encouraged their use.

The Third Avenue Bridge across the Mississippi River in Minneapolis was designed with all of the above factors considered. For the past 50 years, it has withstood the forces of nature and man, experiencing decades of weathering and deterioration, increasing loads and traffic densities, yet functioning well with a nominal amount of maintenance. It has met or exceeded its intended life span and has reached a condition of questionable structural safety.

The purpose of this report is to present the results of visual inspections, material testing, and design analysis and to recommend repair and/or reconstruction procedures for the Third Avenue Bridge.

GENERAL DESCRIPTION OF THE BRIDGE

The 1,914 foot Third Avenue Bridge connects the intersection of First Street South and Third Avenue South on the south approach to First Avenue Southeast on the north end. The Bridge consists of six distinct units – the south abutment, four south approach spans, five ribbed arch spans, two barrel arch spans, four north approach spans and the north abutment. The earth filled abutments have reinforced concrete wing walls and abutment walls. Spans 1 and 2 utilize sixteen reinforced concrete girders supported by three-column, reinforced concrete bents while Spans 3 and 4 have five steel girders supported by the same type of bents. Five of the arches have three ribs and a clear distance between springing lines of 211'-0"; the other two are barrel arches with a distance of 134'-0" between springing lines. Open spandrel columns are used above the ribbed arches and spandrel walls above the barrel arches. The four spans of the north approach use sixteen reinforced concrete girders supported by five-column reinforced concrete bents.

The asphalt surfaced roadway is 56'-0" wide between the faces of the traffic railings and is flanked on both sides by 10'-0" concrete sidewalks. Decorative pedestrian railings make up the exterior bridge railing. The out-to-out width of the bridge is 82'-6".

The Third Avenue Bridge (No. 2440) carries Trunk Highway No. 8 and was added to the Minnesota Trunk Highway system on December 30, 1933. The bridge is currently maintained by the City through an agreement with and at the expense of the Minnesota Department of Highways.

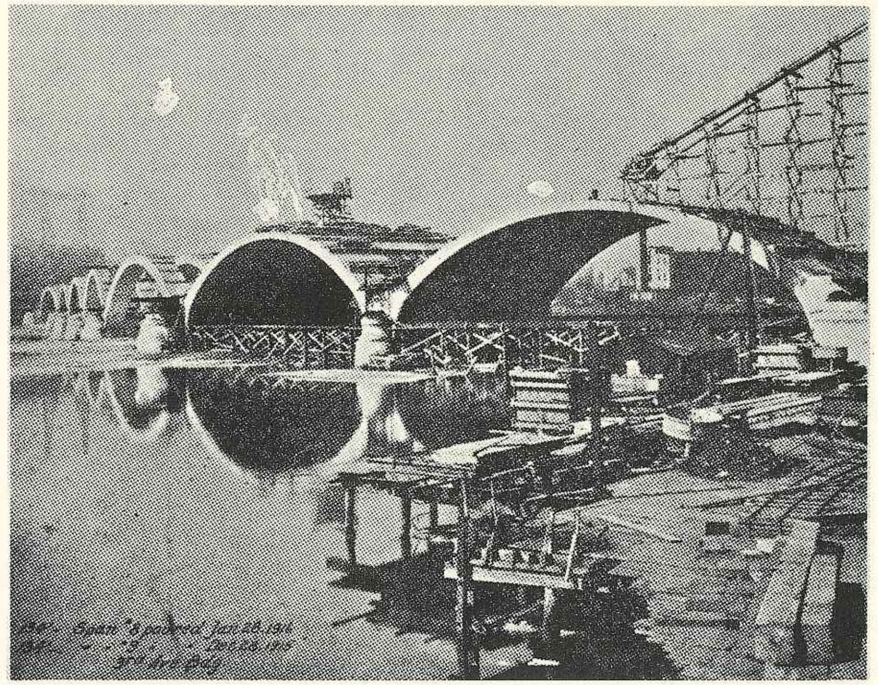
HISTORY OF THE BRIDGE

As the commercial and residential sections of Northeast Minneapolis developed, a substantial bridge to carry two lanes of motor vehicles and two lanes of street cars was needed to cross the Mississippi River in the vicinity of St. Anthony Falls. City Engineer, F. W. Cappelen, proposed several routes for a multiple span, reinforced concrete bridge connecting Third Avenue South to First Avenue Southeast.

Because of four breaks in the limestone strata at St. Anthony Falls between October 1869 and April 1875, about 25 rock borings were made at the proposed pier sites in the winter of 1912-1913. The results of the borings and the proposed pier locations were plotted on a "U.S. Government Map of the Mississippi River," and with a report by Engineer Cappelen, were submitted to the "Special Committee on New Bridge" on November 14, 1913. The decision was to adopt an "S" shaped, multiple span, reinforced concrete arch bridge; the "S" shape was dictated by the need to locate the piers outside of the areas of the breaks.

The Concrete Steel Engineering Company, Park Row Building, New York City was retained to design the bridge, completing the plans in early 1914. Because of railroad clearance problems under each approach, Engineer Cappelen designed steel and reinforced concrete beams and reinforced concrete bents to replace the proposed symmetrical arches.

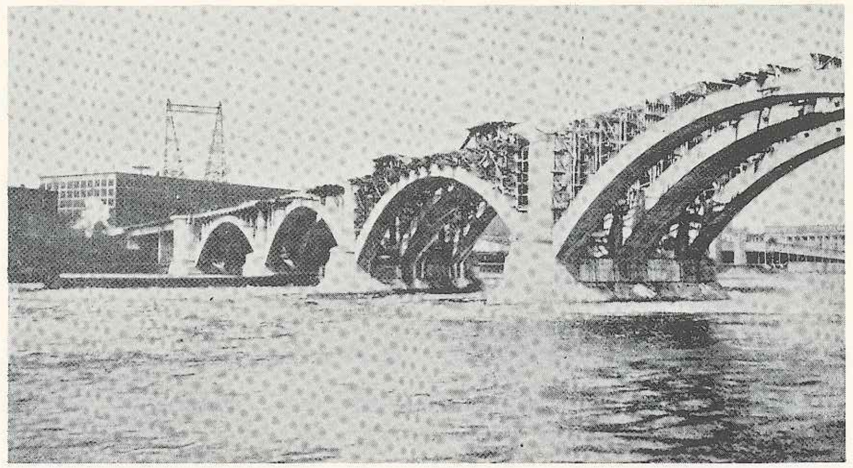
Construction by the City began in early 1915, the arches over the channel being completed on January 28, 1916; see Photo I-1. By late 1916, nearly the entire south approach and most of the pier walls were completed. The year of 1917 saw the completion of the south approach (the spandrel columns and walls, the pier walls, and the deck and railing of the south approach) and the arch spans; work was begun on the north approach. The Minneapolis Street Railway Company, the Minneapolis General Electric Company, the Tri-State Telephone Company, and the Northwestern Telephone Company began to lay conduit under the sidewalks and the steel ties and rails of the double tracks of the Minneapolis Street Railway Company were placed in a 9 inch soil fill above the completed portion of the deck. A strip, 18 inches to the outside of the outer rails, was surfaced with 4 inch thick granite blocks; the remainder of the roadway had a 4 inch thick creosoted wood block wearing surface. See Photos I-2 and I-3.



I-1. View of the completed arches as seen from the north side of the river.

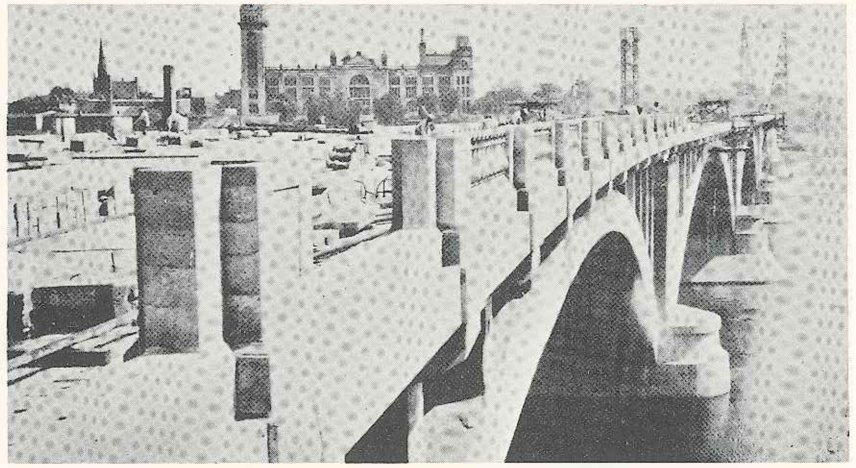
The last yard of concrete was poured at 3:30 p.m., March 29, 1918. The double streetcar tracks and conduit laying was completed, a spiral staircase from the north approach of the bridge to Main Street was erected and the last lamp post and railings were finished by June 6th. Following removal of most of the construction equipment and forms, the bridge was opened to the public at 1:30 p.m. on June 13, 1918.

It is of interest to note that the bridge cost the City of Minneapolis $860,000 and took four years to build. It contains 58,270 cubic yards of concrete, placed at an average unit cost of $14.05 per cubic yard.



I-2. Ribbed arches and south approach on May 17, 1917.

I-3. Deck and railing of Span 3 on August 10, 1917.



Since completion, the visible alterations to the bridge include the following: Replacement of the bridge railings, sidewalks, and curbs by a new traffic railing and new sidewalk deck with conduit space below it, a new exterior bridge railing and repair of about 50% of the cantilevered portion of the spandrel columns and walls, (1939); removal of the original decorative lighting system and replacement by lights mounted on the sidewalk (1939) and, later, mercury vapor lights using the same locations; and replacement of the creosoted paving blocks by an asphalt wearing course, covering the tracks and the cemented sand filler (date unknown).

SCOPE OF THE REPORT

For the presentation of this report, the following studies and investigations were made:

1. Detailed visual inspection of the condition of the structure.
2. Sampling and testing of material from sound members of the structure; included are corings into the deck, spandrel columns, arch ribs and barrel arches, beams and columns of bents.
3. Study of river pier founding conditions.
4. Analysis and rating of the load carrying capacity of the structure.
5. Study of rehabilitation and cost estimate for repair of the structure.
6. Comparative cost estimate for a replacement structure.

Results of the inspection and foundation investigation are included in Parts II and III respectively. Part IV of the report contains the cost estimate for the repair work deemed necessary, as well as the comparative cost estimates for a replacement structure.

SUMMARY OF RECOMMENDATIONS

Repair and reconstruction recommendations are based on visual inspection, foundation investigation, material testing, and design analysis with consideration given to future traffic needs, aesthetics, and the relative economy of construction alternates. These can be summarized as follows:

1. It is recommended that both approaches be removed from the footings up as explained in the "Reconstruction" section of Part IV. This would include the abutment walls and bridge seats.

2. It is recommended, for the approaches, that a new reinforced concrete deck supported by ten continuous steel beams be constructed; these in turn supported by four-column reinforced concrete bents on pile supported footings. See the Appendix for the proposed section. The new bents should be constructed in the same locations as the existing bents.

3. It is recommended, for the arch spans, that the bridge deck, sidewalks, railings, the cantilever portion of the spandrels and pier walls, and the area of the spandrels and pier walls with unsound concrete be reconstructed. Deck crown should be provided by the spandrels and pier walls. Exhibits 2 and 3 show sections of the proposed new deck above the ribbed and barrel arches and Exhibit 8 shows the recommended spandrel reconstruction.

4. It is recommended that the new reinforced concrete slab provide for a roadway of four 12 foot lanes, two 3'-9" curb reaction widths and two 13'-6" sidewalks (roadway face of curb to outside face of the deck). The use of a traffic rail between roadway and sidewalks and a 4'-6" high exterior pedestrian rail is advisable.

5. It is recommended that the piers, the arch ribs and barrel arches, and the portion of the reusable columns be repaired as necessary and that the exposed surfaces of the entire bridge be treated to resist future deterioration.

6. It is recommended that a temporary relocation of the utility system be provided during deck removal and reconstruction.

7. It is recommended that a new drainage system using deck drains and a closed system to the river be constructed for the entire bridge.

PART II INSPECTION

GENERAL

Visual inspection of the surface condition was begun on Dec. 20, 1967, and was completed by January 16, 1968. Extremely cold weather from January 5th through the 14th, halted the inspection for ten days. Temperatures ranged from 42° F to -19° F with a 15° F average temperature; skies were frequently overcast with accompanying strong and gusty winds.

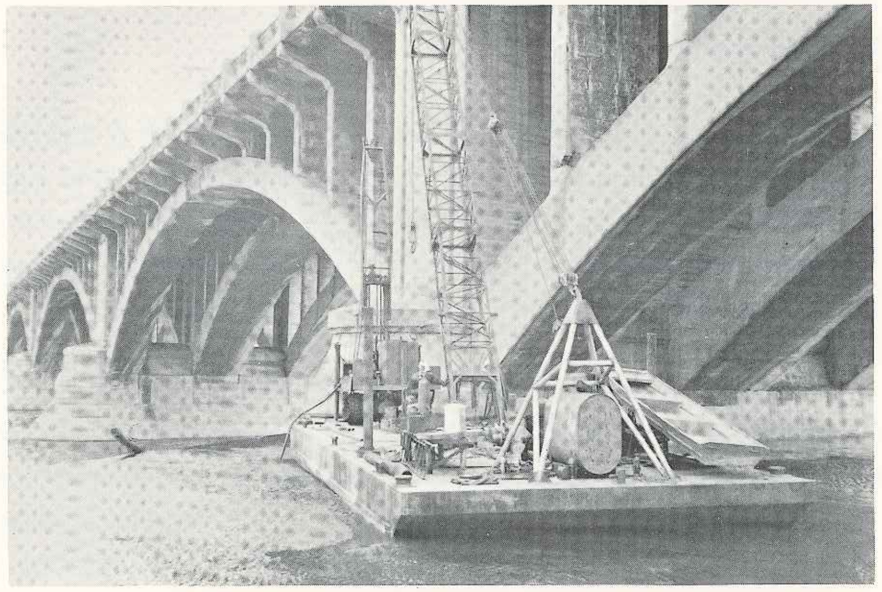
Borings for the piers and footings were started on April 8, 1968, and completed on July 23, 1968. Core sampling of members of the superstructure began May 9, 1968, and was completed on June 17, 1968. Temperatures during the boring and core sampling phase ranged from 96° F to 12° F and the weather was generally fair.

During the morning and evening rush hours, traffic was extremely heavy on all four lanes, whereas, traffic between the peak hours could be adequately handled with one lane blocked. When it was necessary to block a lane, the Minneapolis Traffic Bureau was advised and cones and/or barricades were used to direct traffic.

Access to all members of the bridge was provided by means of a hydraulic boom, shown in Photo II-1. From the truck on the deck of the bridge, the multiple boom, hydraulic lift could reach over the rail and under the deck to provide a platform for the inspector. The barge, giving access to the piers is shown in Photo II-2.

To aid in the systematic and comprehensive visual inspection of the members of the bridge, all members were assigned numbers and/or letters. Preprinted inspection forms for each basic type of member were used to record the observations and photograph numbers pertaining to the particular member. These forms were also used to tabulate the dimensions of areas

II-1. "Snooper" used for visual inspection.



II-2. Barge and drilling equipment used for pier exploration.

requiring repair and additionally listed quantities for which repair costs could subsequently be determined. See Figure 2 for a typical inspection form used for the inspection of the spandrel columns and walls.

DESCRIPTION OF THE EXISTING STRUCTURE

Exhibit 1 of the Appendix shows a general plan and elevation view of the entire bridge. Note the member identification procedure used during the inspection.

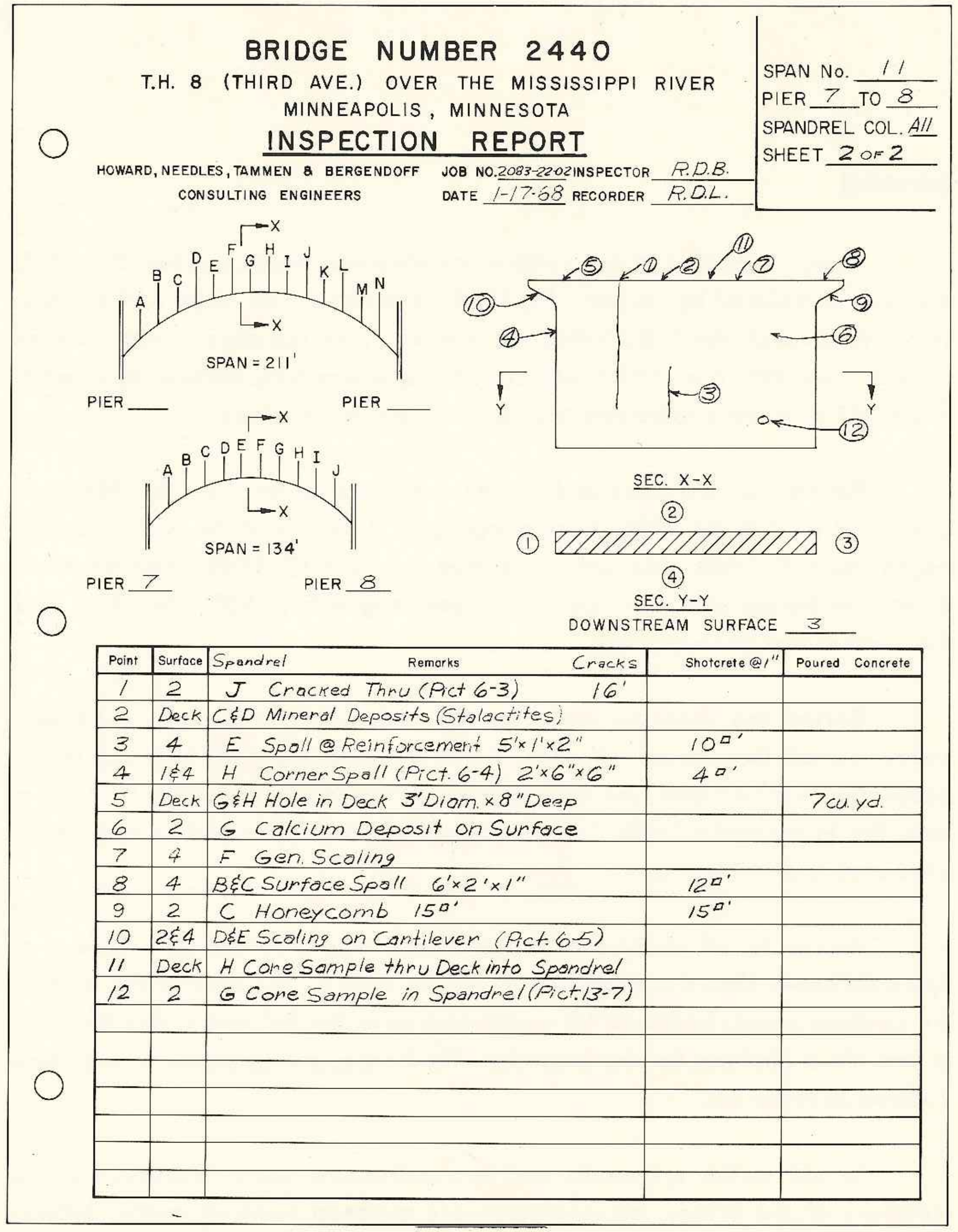
South Abutment

The south abutment consists of a reinforced concrete abutment wall supported on a spread footing and wing walls of reinforced concrete, also on spread footings. Soil fill is used to bring the roadway to grade.

South Approach Spans

Spans 1 and 2 consist of sixteen concrete T-beams, with 6 inch slab and overall depth of 3'-0", spaced at an average of 5'-1" centers and supported by three-column, reinforced concrete bents. In Span 1 these beams have been shored-up with concrete blocks and the entire area under the deck has been walled-off by rocks and timbers; See Photo II-3 for view of Bent 1.

Spans 3 and 4 consist of 24 inch "I" floorbeams spaced at 3 foot on centers and framed into five 8 foot deep steel girders, in turn supported by three-column, reinforced concrete bents. In addition, to help support the floor beams in Span 3, new 36WF 230 beams with a 14WF 95 column at approximately midspan have been erected 3 feet inside of the exterior girders. Also, end bearing supports, on each side of Bent 3 help support the center girder.



BRIDGE NUMBER 2440
T.H. 8 (THIRD AVE.) OVER THE MISSISSIPPI RIVER
MINNEAPOLIS, MINNESOTA
INSPECTION REPORT

HOWARD, NEEDLES, TAMMEN & BERGENDOFF
CONSULTING ENGINEERS

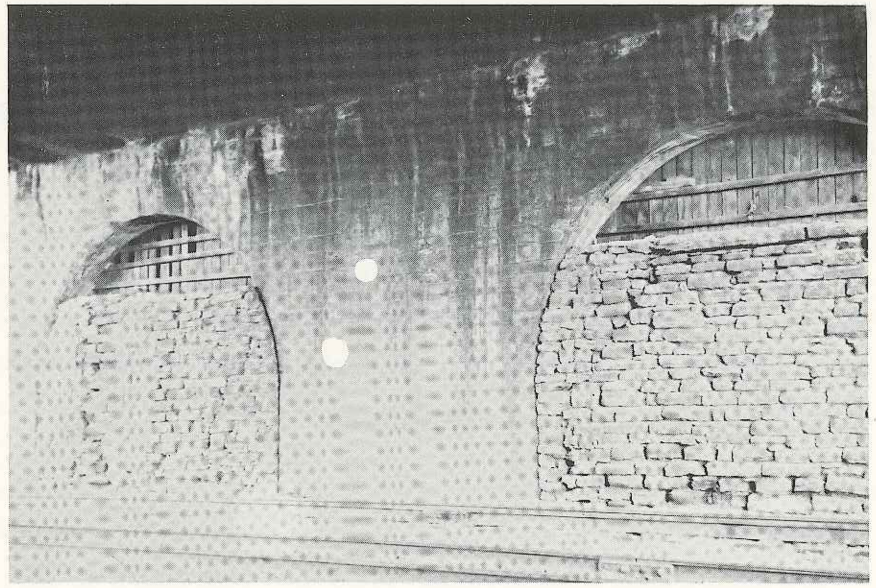
JOB NO. 2083-22-02 INSPECTOR R.D.B.
DATE 1-17-68 RECORDER R.D.L.

SPAN No. 11
PIER 7 TO 8
SPANDREL COL. All
SHEET 2 of 2

PIER ____ PIER ____
PIER 7 PIER 8
DOWNSTREAM SURFACE 3

Point	Surface	Spandrel	Remarks	Cracks	Shotcrete @1"	Poured Concrete
1	2	J	Cracked Thru (Pict 6-3)	16'		
2	Deck	C&D	Mineral Deposits (Stalactites)			
3	4	E	Spall @ Reinforcement 5'×1'×2"		10□'	
4	1&4	H	Corner Spall (Pict. 6-4) 2'×6"×6"		4□'	
5	Deck	G&H	Hole in Deck 3' Diam. × 8" Deep			7 cu. yd.
6	2	G	Calcium Deposit on Surface			
7	4	F	Gen. Scaling			
8	4	B&C	Surface Spall 6'×2'×1"		12□'	
9	2	C	Honeycomb 15□'		15□'	
10	2&4	D&E	Scaling on Cantilever (Pict. 6-5)			
11	Deck	H	Core Sample thru Deck into Spandrel			
12	2	G	Core Sample in Spandrel (Pict. 13-7)			

Figure 2
INSPECTION FORM



II-3. *Bent 1 with rock and timber wall enclosing Span 1.*

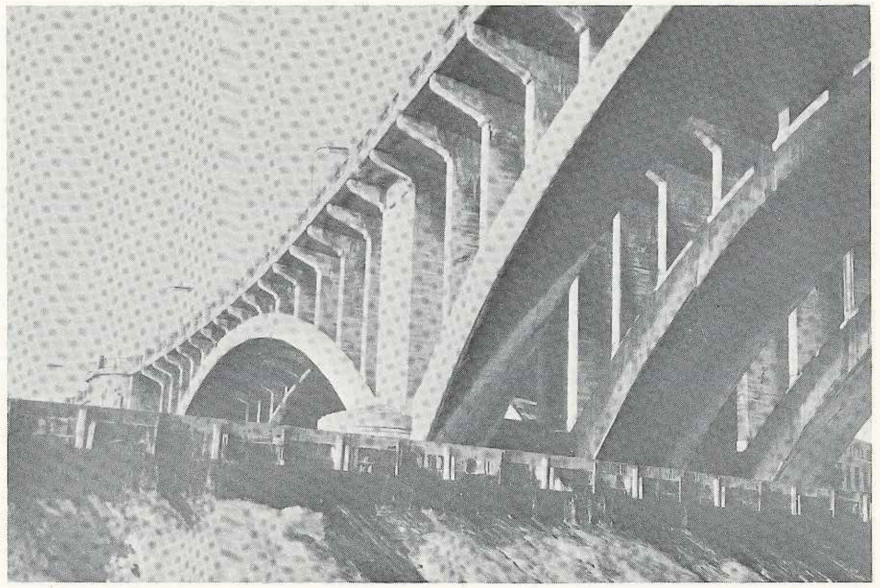
Arch Spans

The center portions of the bridge consists of five spans of three ribbed arches (Spans 5, 6, 7, 8 and 9) and two spans with barrel arches (Spans 10 and 11); see Exhibit 1 and Figure 2 for layout and numbering sequence. The ribbed arch span length from pier face to pier face at the springing line is 211′-0″ while that for the barrel arches is 134′-0″.

For the ribbed arches, the center arch rib is a constant 16′-0″ wide, whereas, the exterior ribs are 10′-0″ wide for the tangent sections of the bridge and 10′-0″ or 12′-0″ for the inside or outside of the curve, respectively, for the curved sections. The face to face distance is a constant 16′-0″ and the arch thickness varies from 8′-0″ at the pier to 4′-6″ at the crown. Fourteen three-column spandrels spaced center to center at 14′-6 3/4″, support the roadway; column height varies from about 25.6 feet to about 3.3 feet, width is a constant 6 inches less than the arch rib width, and the thickness is a constant 2′-0″.

The barrel arch has a constant 76′-0″ width and varies in thickness from 6′-1″ at the pier to 2′-6″ at the crown. Supporting the roadway are ten spandrel walls spaced at a center to center distance of 11′-2″. The thickness of each wall tapers from 2′-0″ at the top to 1′-6″ at 1′-0″ below the top. Width is 75′-6″ while height varies from about 16.2 feet to about 1.2 feet.

The width at the springing line of a typical pier for the ribbed arches is 20.345 feet and for the barrel arches is 13.79 feet. The width at the transition from the south approach to the ribbed arch is 30.402 feet; for the transitions from ribbed arch to barrel arch and barrel arch to the north approach, the pier widths are 30′-0″. Photo II-4 shows the general configuration of a ribbed arch.



II-4. *View of ribbed arches and horseshoe dam.*

North Approach

The north approach spans consist of sixteen reinforced concrete T-beams with tapered haunches supported by five-column, reinforced concrete bents. The beams are spaced at an average of 5′-1″ on centers, have a six inch slab thickness, and overall depths of 4′-3″ for Spans 12, 13 and 14 and 6′-0″ for

Span 15. Due to the deteriorated state of several of the beams on the downstream side, new 33 WF beams have been placed on each side of the beam under the curb in Span 12 and on each side of the beam under the curb and the adjacent interior beam in Spans 13 and 14.

North Abutment

The north abutment consists of reinforced concrete abutment wall on the spread footings and two long wing retaining walls on spread footings. The approach is filled with soil to bring the roadway up to grade.

Roadway

The 56′-0″ roadway is flanked on both sides by 13′-3″ sidewalks. A tubular steel traffic safety rail separates the roadway from the sidewalks and sturdy concrete posts with ornamental steel railings comprise the exterior bridge railing. Because the bridge was built to carry two lanes of street cars on tracks centered 6′-3″ on each side of the bridge centerline, the cross-section has a reinforced concrete deck, a cemented sand filler to dampen vibrations and provide crown and an asphalt wearing surface.

The deck of the approaches consists of the six inch flange of the T-beams for Spans 1, 2, 12, 13, 14 and 15 and a six inch reinforced concrete slab supported by floorbeams and steel girders in Spans 3 and 4; spandrels above the ribbed and barrel arches support 12 and 10 inch slabs, respectively. A cemented sand filler of variable depth separates the deck from an asphalt wearing surface, also of variable thickness. The steel rails and ties, along with 4 inch thick granite blocks, extending 1′-6″ outside of the rails, are covered by the asphalt wearing course. Within each sidewalk is a sizeable area allotted to utility conduits. See Exhibits 2 and 3 for existing cross-sections of the deck above the ribbed arch and the barrel arch, respectively.

DEFINITION OF TERMS

The condition of all members inspected was recorded on the inspection form using the following definitions for types of deterioration:

Scale – A patterned discoloration identifying an area in which water, seeping through the concrete, is beginning to leach calcium salts and deposit them on the surface. As further leaching occurs, mineral deposits will form. Photo II-5 shows a good example of scale on the underside of the deck.

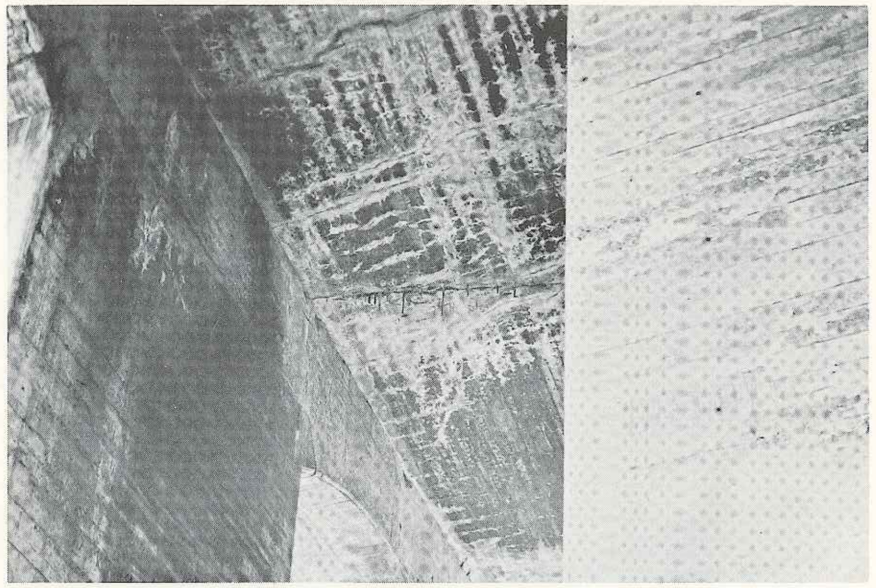
Mineral Deposit – A lightweight, brittle material deposited by water seeping through the pores or over the surface of concrete. This deposit consists of calcium salts leached from the concrete through or over which the water has passed. When the water has passed through the concrete, the concrete looses its cohesiveness and becomes soft or spongy. Typical examples on vertical and horizontal surfaces are shown, respectively, in Photos II-6 and II-7. Note that these deposits under horizontal surfaces form small stalactites.

Spall – A physical separation of a section of the concrete either cracked parallel to the surface or broken away such that this section cannot assist in the structural integrity of the member. Spall often occurs in areas of marginal cover over reinforcement, areas subjected to excessive water flow, or corners or edges of members. A good example of spall that has bared the reinforcement in a column of Bent 4 is shown in Photo II-8.

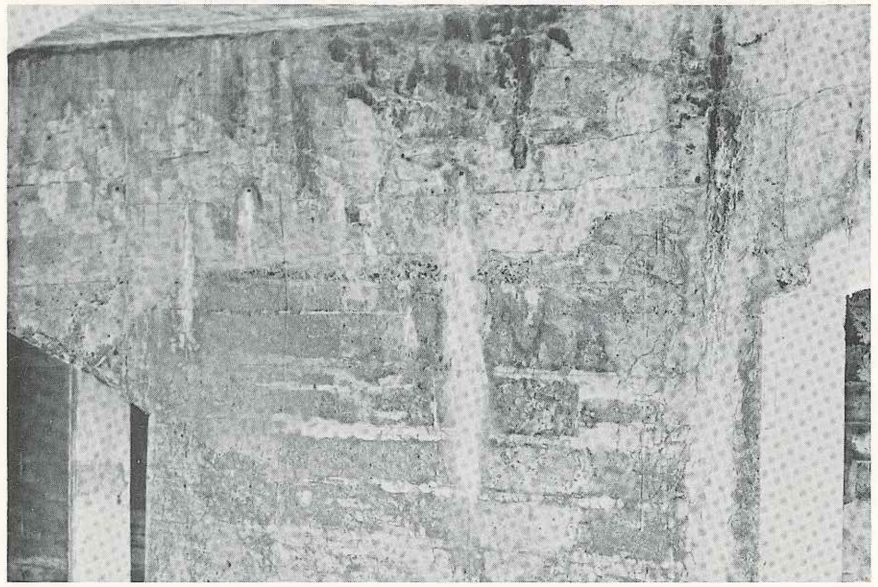
SUMMARY OF VISUAL INSPECTION

Roadway Deck, Sidewalks, and Railings

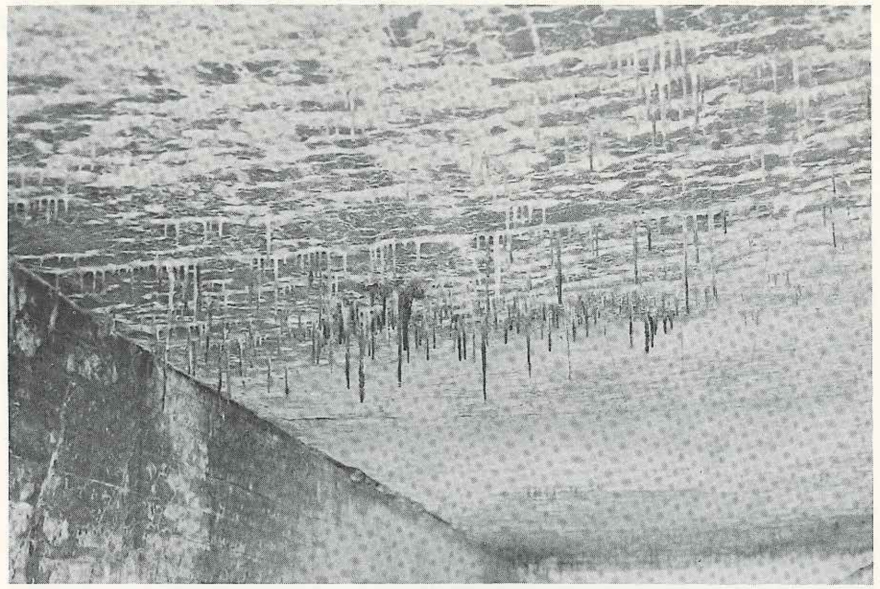
The good appearance of the asphalt wearing surface, the sidewalks, and the railings, as seen from the top of the deck, is very misleading. Underdeck inspection reveals: (1) The south approach spans have extensive deterioration on both sides of the deck expansion joints at each bent, scale formations are frequent, and some mineral deposits exist adjacent to the deck expansion joints; see Photo II-9. (2) Each arch span has scale throughout, has frequent



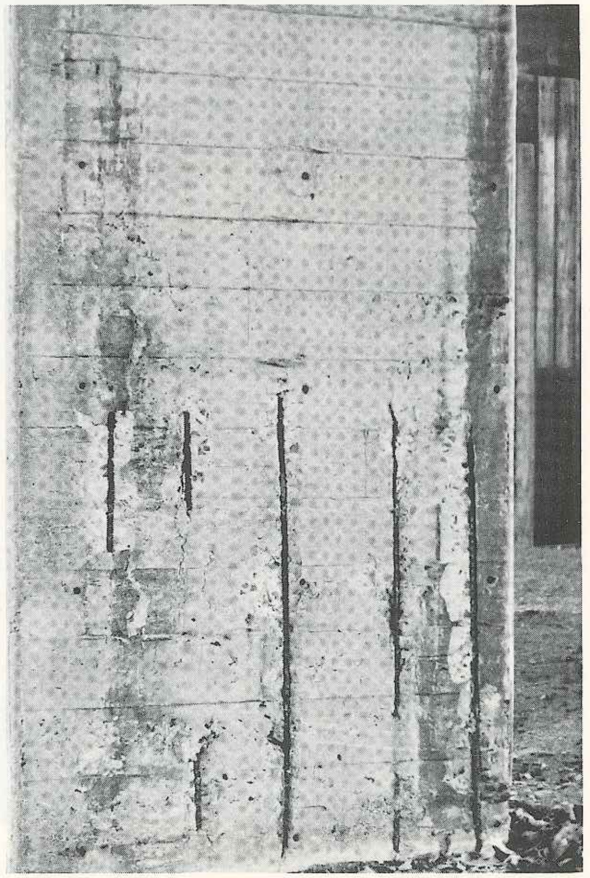
II-5. *Patterned discoloration identifying scale on the underside of the deck.*



II-6. *Mineral deposits on the vertical face of a spandrel column.*

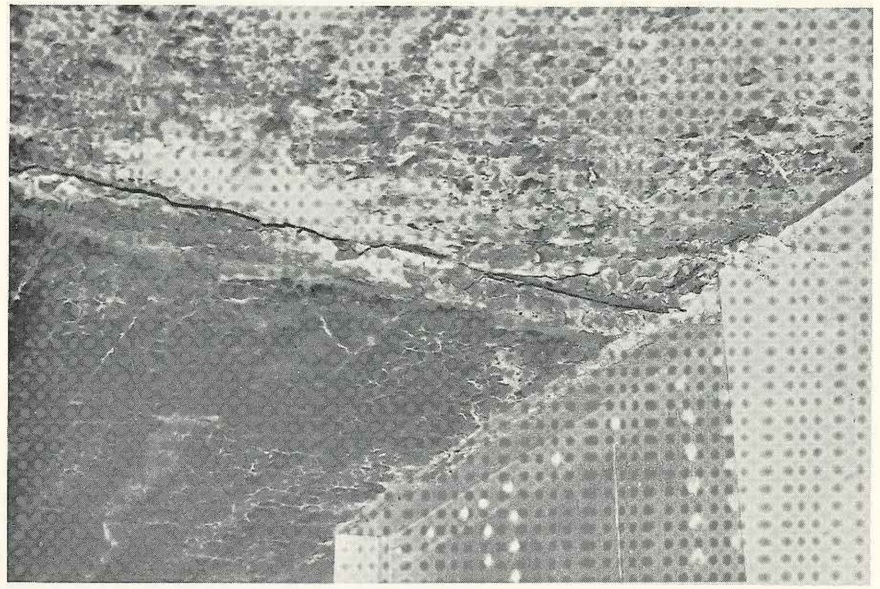


II-7. *Stalactites on the underside of the deck.*

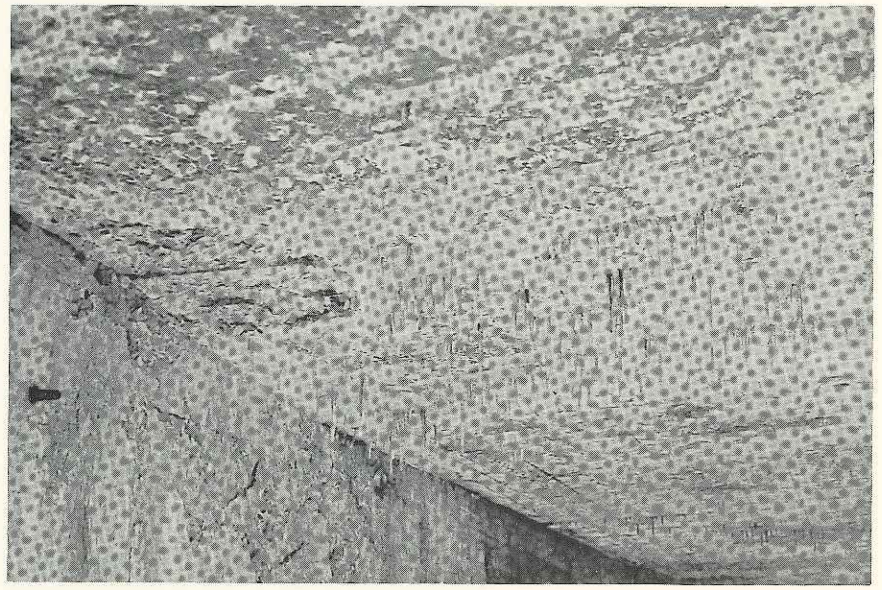


II-8. *Typical example of spall baring the vertical reinforcement of a column.*

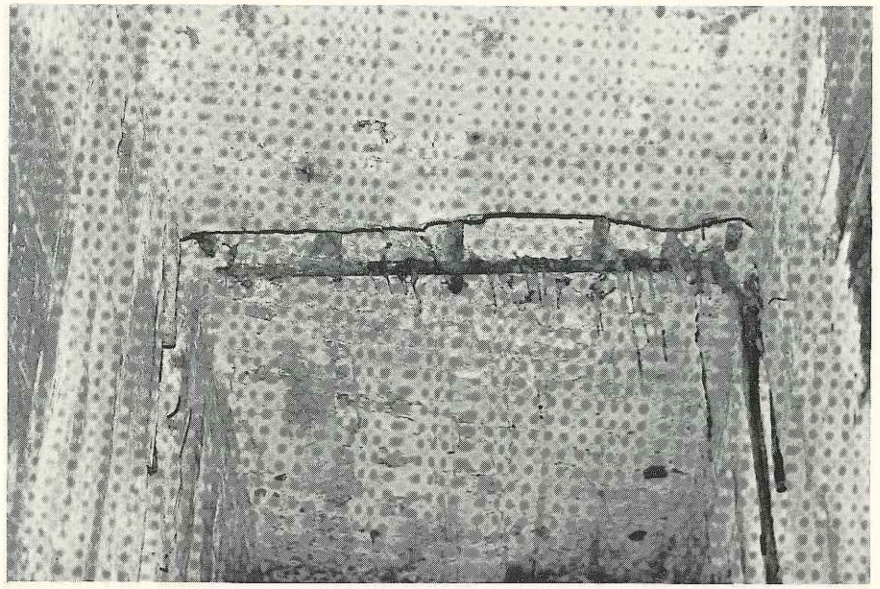
areas of well developed mineral deposits and has some areas of spalling, especially in the vicinity of the spandrels. Representative examples of the above types of deterioration are shown in Photo II-10. (3) In addition to widespread areas of scale, the north approach spans have areas of extensive deck deterioration in the vicinity of the deck expansion joints; see Photo II-11. (4) Throughout the bridge, the deck at the downspouts has undergone severe spalling, completely exposing the deck reinforcement grid for areas frequently as large as 6 feet in diameter; see Photos II-12 and II-13.



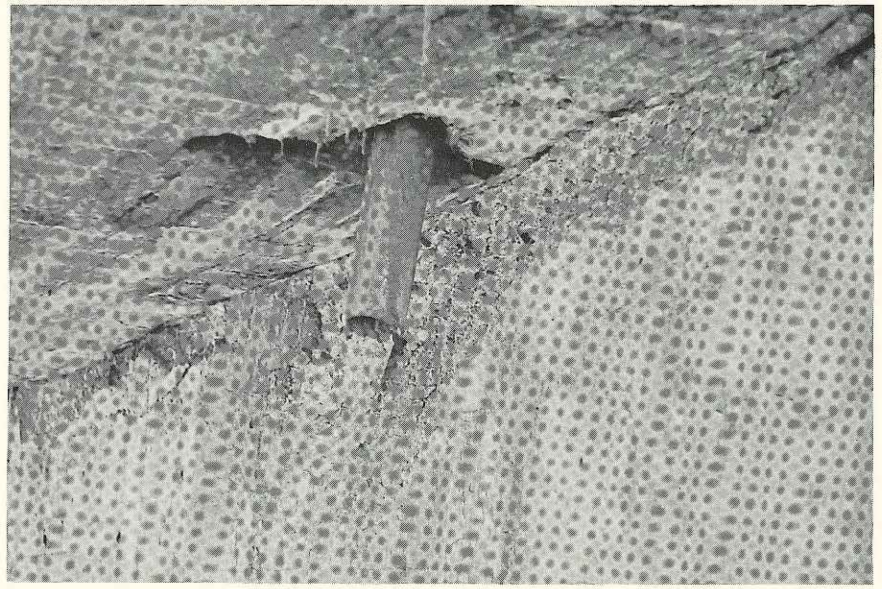
II-9. *Deck expansion joint in Span 3.*



II-10. *Well developed examples of scale, mineral deposits, and spall in Span 7.*

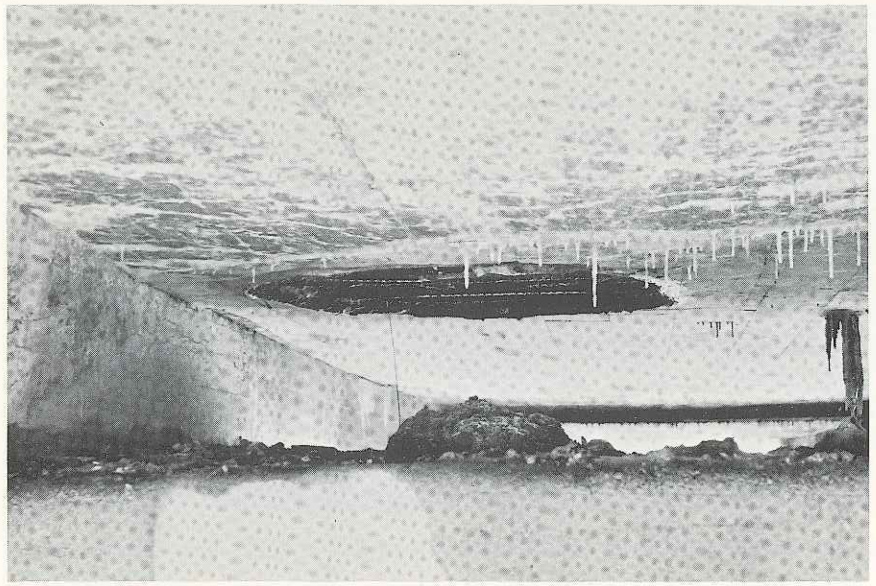


II-11. *Spalled deck expansion joint and butt ends of T-beams at Bent 4.*

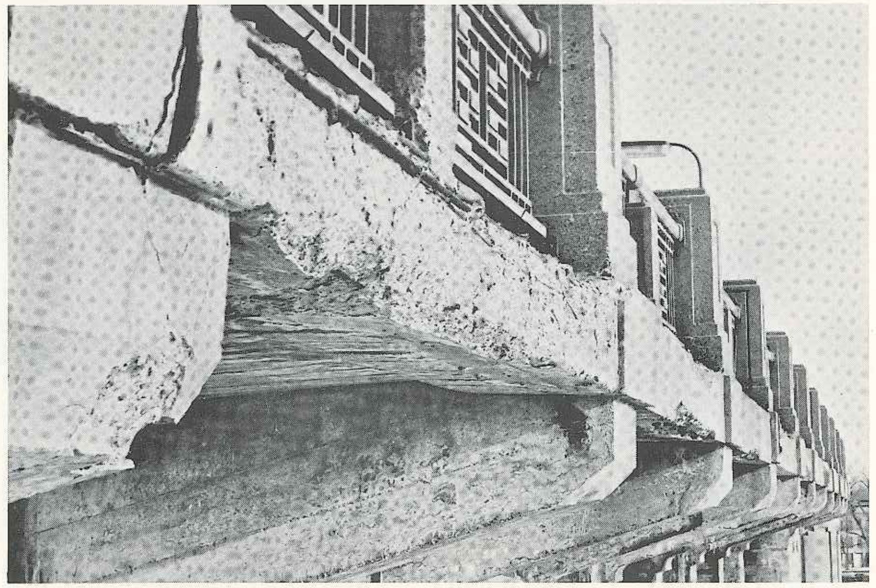


II-12. *Spall at deck and spandrel column due to deteriorated floor drain.*

The sidewalks and the concrete posts for the bridge rail are structurally sound throughout; there are some areas, however, of spall on the outside face of the posts and the underside of the deck as shown in Photo II-14. Some scale has formed on the underside of the deck also.



II-13. Spalled area of approximately 6 feet in diameter at floor drain.



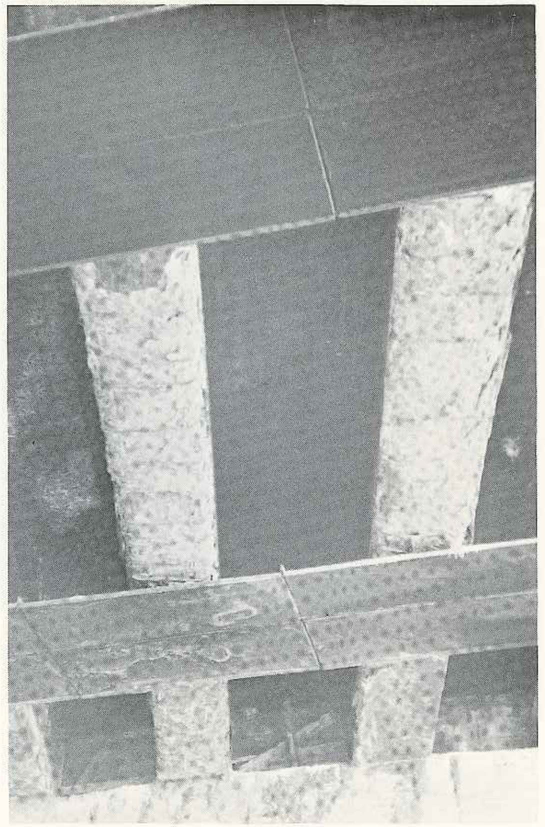
II-14. Areas of spall at the bridge railing posts, sidewalk deck, and spandrel cantilevers.

Concrete and Steel Girders

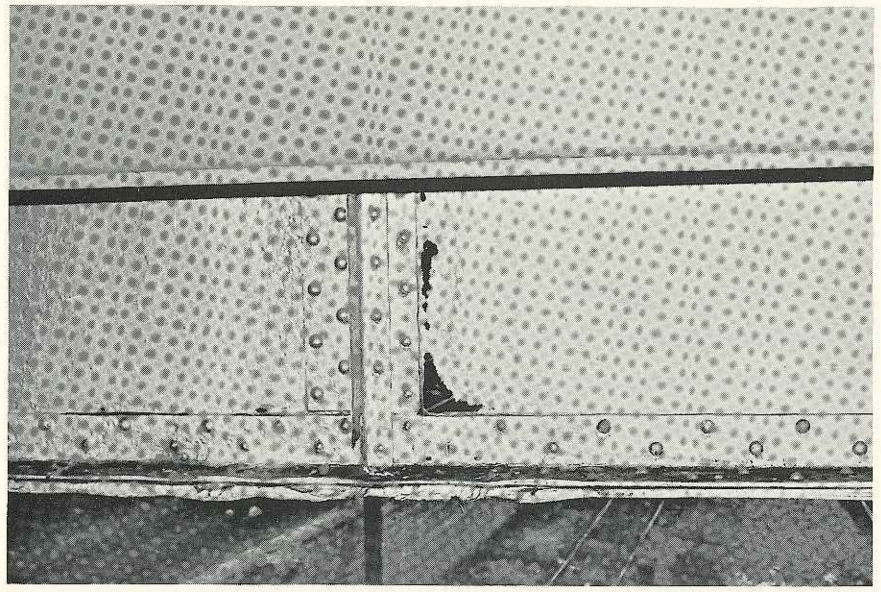
The concrete girders of the approach spans have undergone extensive deterioration. All spans have girders with areas of scale and mineral deposits and areas in which spalling has bared the vertical stirrups and the tensile reinforcement, invariably reducing the net section critically.

The corrosive exhaust of the locomotives has helped expose the reinforcement of the concrete beams of Span 2 and has aided the rusting of the webs, flanges, and bearing stiffeners of the steel plate girders of Spans 3 and 4; see Photos II-15, II-16, and II-17.

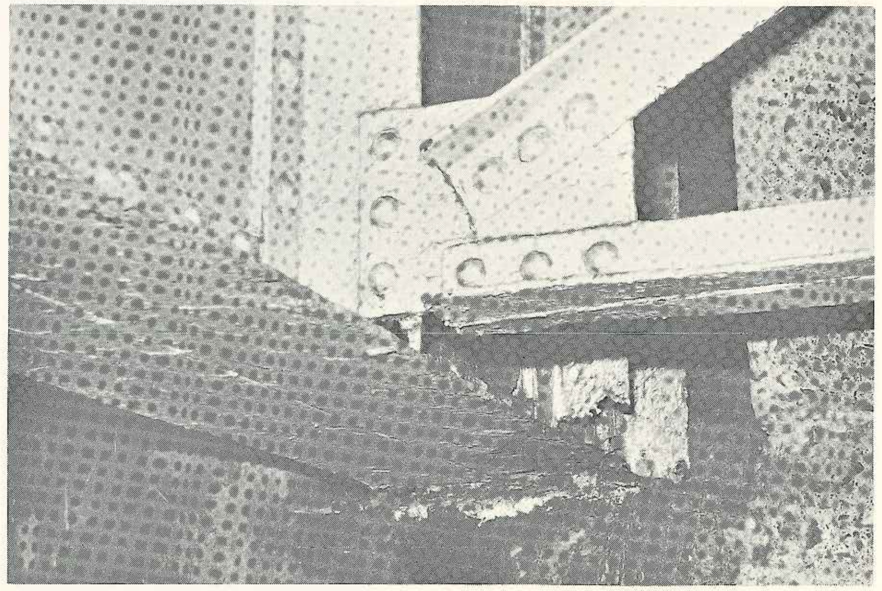
In addition to the usual baring of the reinforcement, the haunches of 50% of the concrete beams of the north approach spans have crumbled or crushed concrete at the support and differential settlement between colinear beams is evident in several areas. See Photos II-18 and II-19.



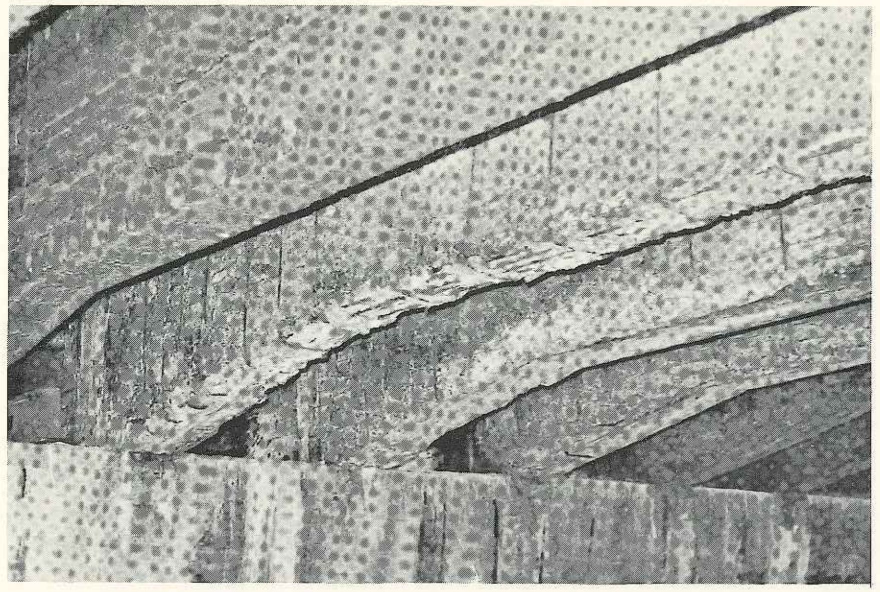
II-15. Deteriorated T-beams and blast plates above railroad tracks under Span 2.



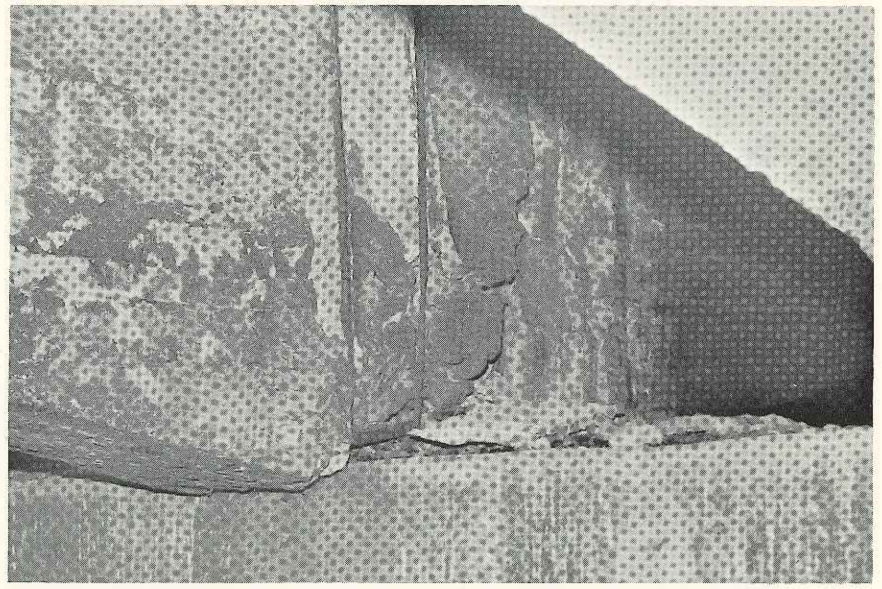
II-16. View of the rusted web and flange of Span 3 plate girder and the new 36WF230 adjacent to it.



II-17. Rusted through bearing stiffener at Bent 2.



II-18. Bared stirrups and tensile reinforcement of the T-beams in Span 13.



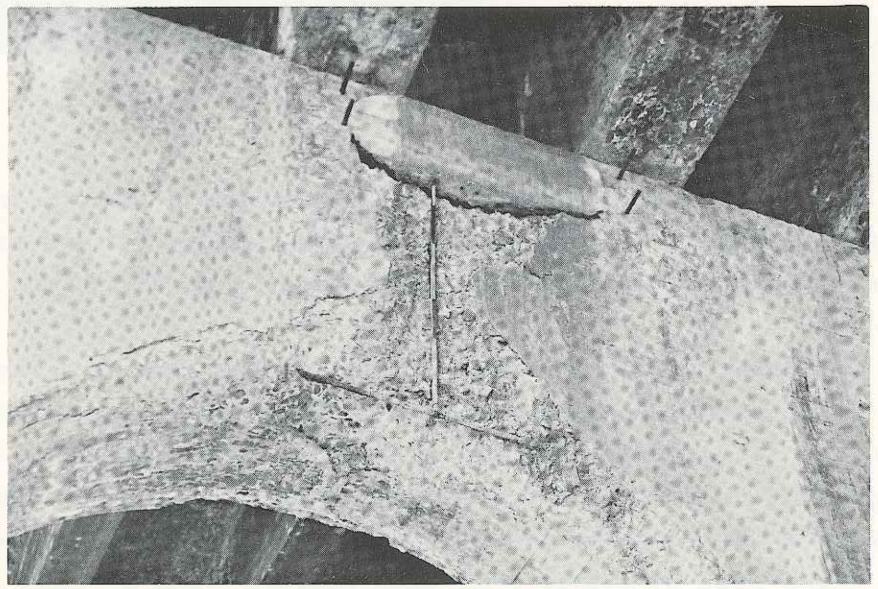
II-19. Crushed concrete at a beam haunch above Bent 5.

A design analysis of the concrete girders indicates that nearly the entire tensile steel areas are necessary to carry the present loads. As the net steel section is reduced through deterioration to the state in which the bottom row of tensile steel is ineffective, the steel stress approaches twice the allowable stress.

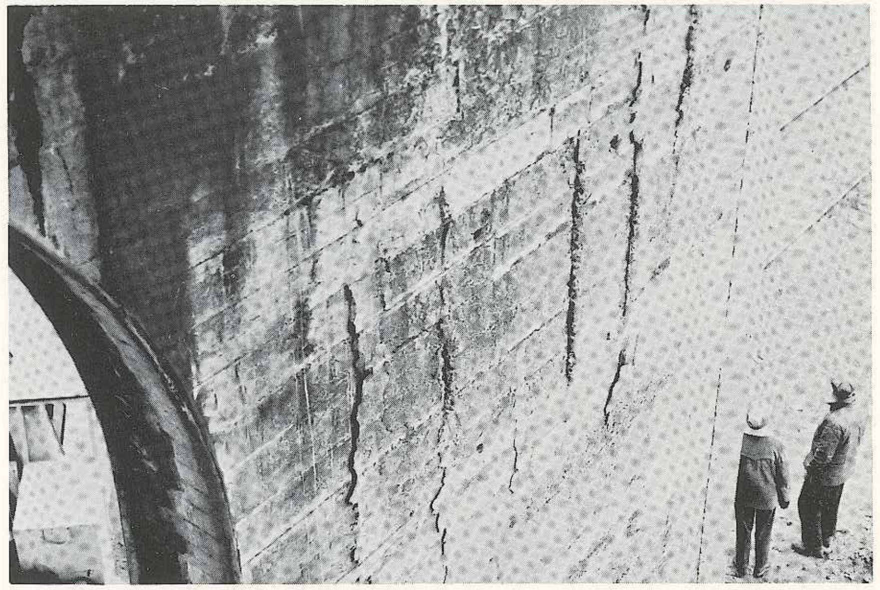
As outlined previously under "Description of Existing Structure," additional supporting members have been placed adjacent to the beams of both approaches with very questionable load carrying capacity.

Concrete Bents

The surface deterioration of Bents 1, 2, 3, and 6 consists of areas of scale on the vertical surfaces and areas of severe spalling, exposing the reinforcement of the columns and cap beams. The cap beams of Bent 1, 2, 3, and 6 have diagonal cracks that are radial to the arches of the cap beams and have completely penetrated the beam. Examples of the above are shown in Photos II-20 and II-21.



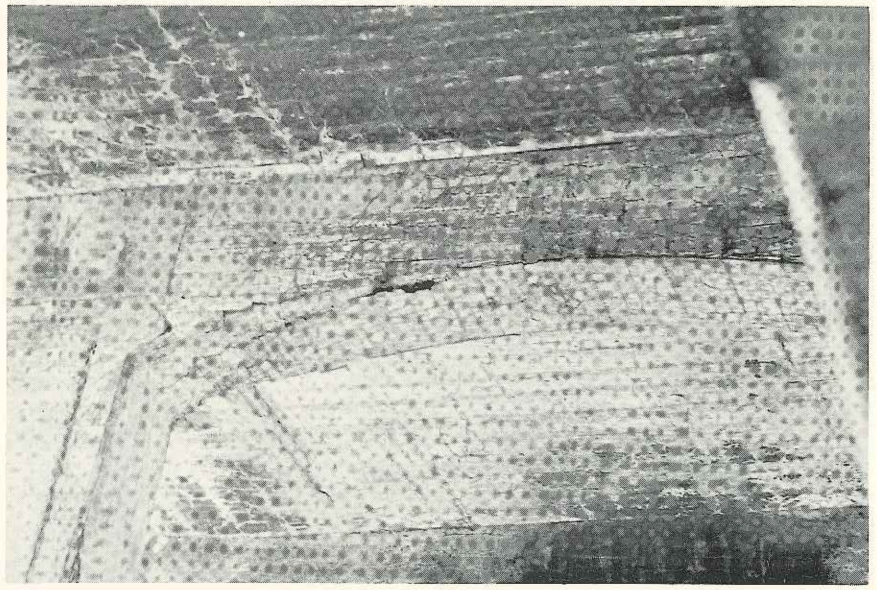
II-20. Spalled area of Bent 6.

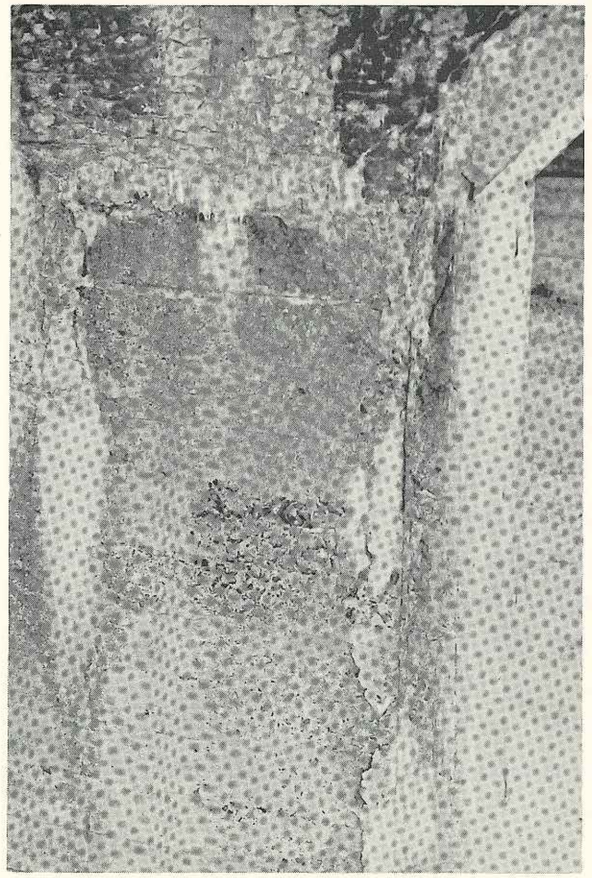


II-21. Bared column reinforcement of Bent 3.

Spandrel Columns and Walls

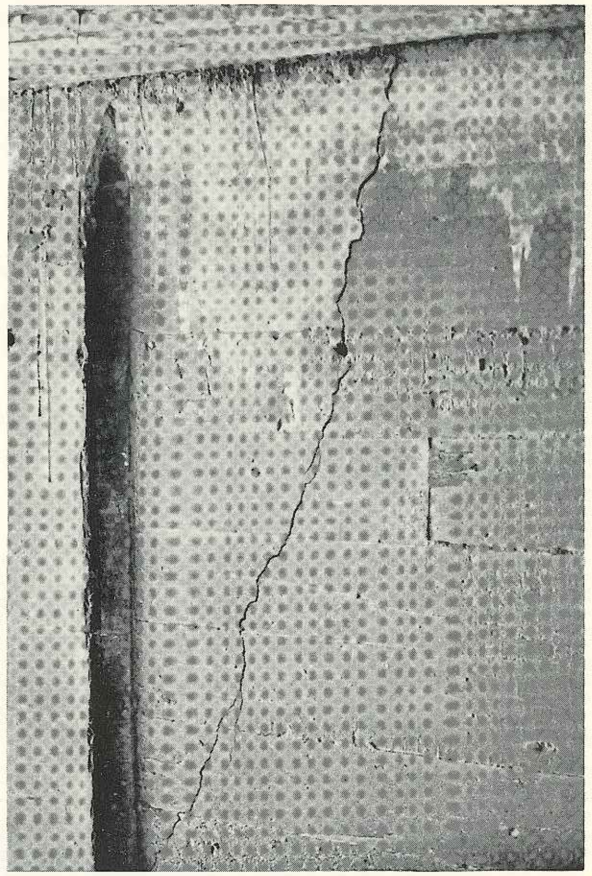
Water, seeping through the deck, the expansion joints and over the outer edge of the sidewalks, has caused considerable deterioration to the upper portion of many spandrel columns and walls. Approximately one-third of the columns above the ribbed arches have areas of mineral deposits and spall on the cap beams and the upper three feet of the columns. The lower portion of the columns have some areas of light spalling that do not impair the structural capacity of the column. See Photos II-22 and II-23, for typical examples. About one-third of the tall spandrel walls above the barrel arches have numerous vertical cracks, usually the length of the column and completely penetrating it. The short walls have well developed mineral deposits from the deck to the back of the barrel arch and the cantilevers have areas of spall under the deck; see Photo II-24.





II-22. Typical examples of deterioration to the capbeam portion of spandrel columns.

II-23. Mineral deposits and bared reinforcement at the column of a spandrel column in Span 9.

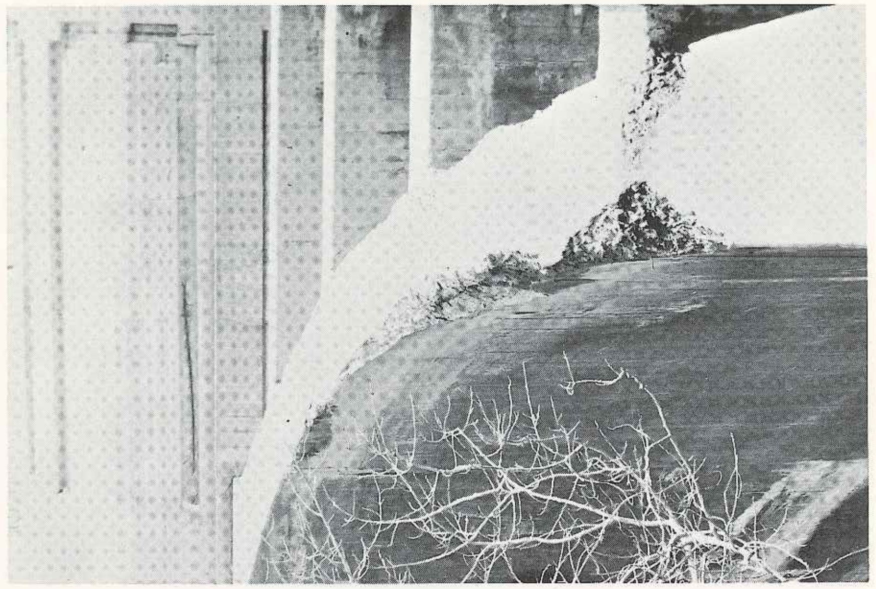


II-24. Nearly vertical crack in a spandrel wall of Span 11.

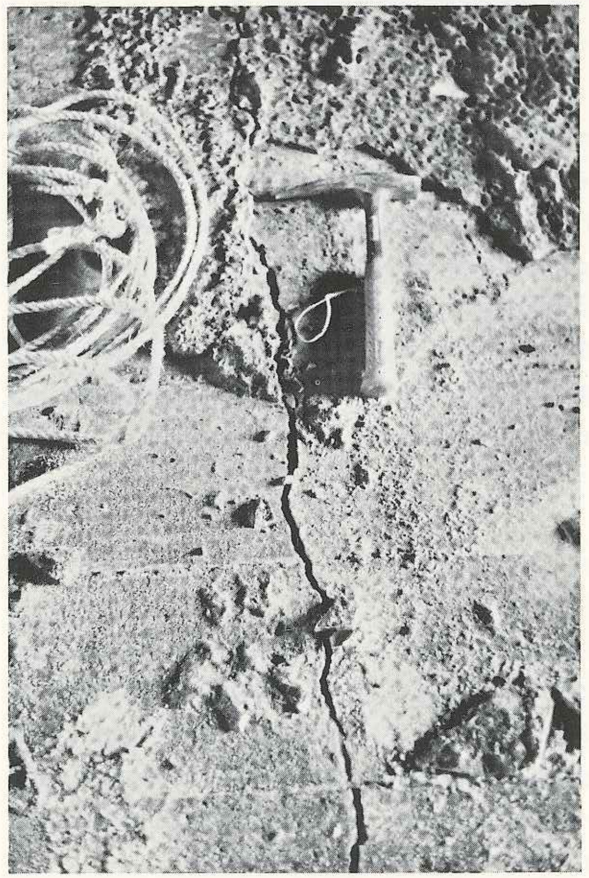
Ribbed and Barrel Arches

Except for some minor spalling at the pedestals for the spandrel columns and the edges of the arch ribs and numerous shallow hairline cracks in the faces of the arch ribs, the arch ribs appear to be structurally sound.

The barrel arches have areas of spalling at the edges throughout the length of the barrels, in several cases baring the reinforcement. Also the following cracks, completely penetrating the arch, were found: Span 10 has six longitudinal cracks between spandrel walls A and B, two between B and C, six between H and I, and one between spandrel wall J and the south wall at Pier 7. One small transverse crack exists between F and G. Span 11 has one small inclined crack, transverse to the arch, between spandrel walls C and D and no longitudinal cracks. See Photos II-25 and II-26.



II-25. *Surface spalling along the edge of the Span 10 barrel arch.*



II-26. *Longitudinal crack through the barrel arch of Span 10.*

The hairline cracks in the arch ribs are commonly associated with shrinkage and/or creep and indicate no loss of structural integrity; test results of the corings bear this out. The longitudinal cracks in the barrel arches, which incidently often propagate into the spandrel walls, are believed to be caused by temperature. Since they are parallel to the principal stresses in the arch, they do not reduce the load carrying capacity of the arch. The two transverse cracks are believed to be the result of loss of bond between concrete pours; since their width is small compared to the width of the arch, they do not weaken the arch to any measurable degree.

Since the above deterioration and cracks do not materially reduce the load carrying capacity of the arches, it is reasonable to assume that the arches are structurally sound. Shotcreting of spalled areas and sealing of all cracks with epoxy would inhibit further deterioration and extend the life of the arches.

Abutments

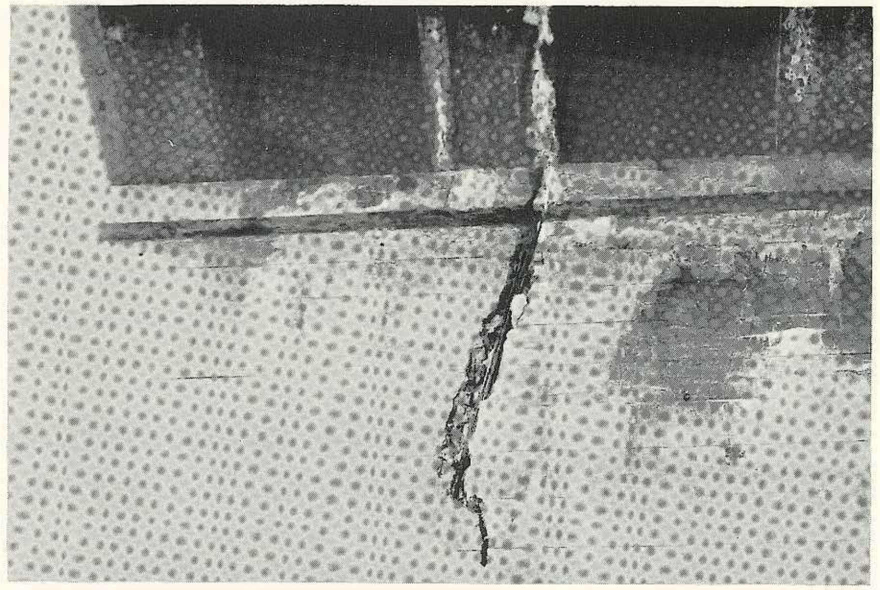
Because the south abutment was inaccessible (for reasons discussed in "Description of the Existing Structure"), it could not be inspected for deterioration. However, no differential settlement or tipping of the abutment wall was observed, but this cannot be construed to mean that the abutment is sound.

As shown in Photo II-27, the north abutment has an area of spalling at the beam seat and has two deep, wide cracks from the beam seat through the abutment wall and, very possibly, into the spread footing. Water, seeping through the expansion joint, has accelerated the deterioration.

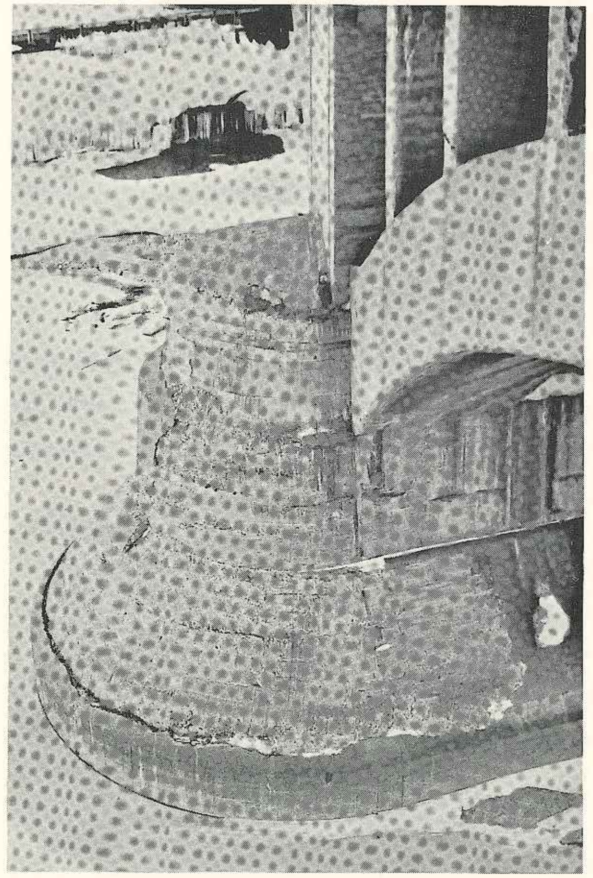
Piers

The bases of all piers show some areas of surface spalling from the water line to the top of the pier base; see Photo II-28.

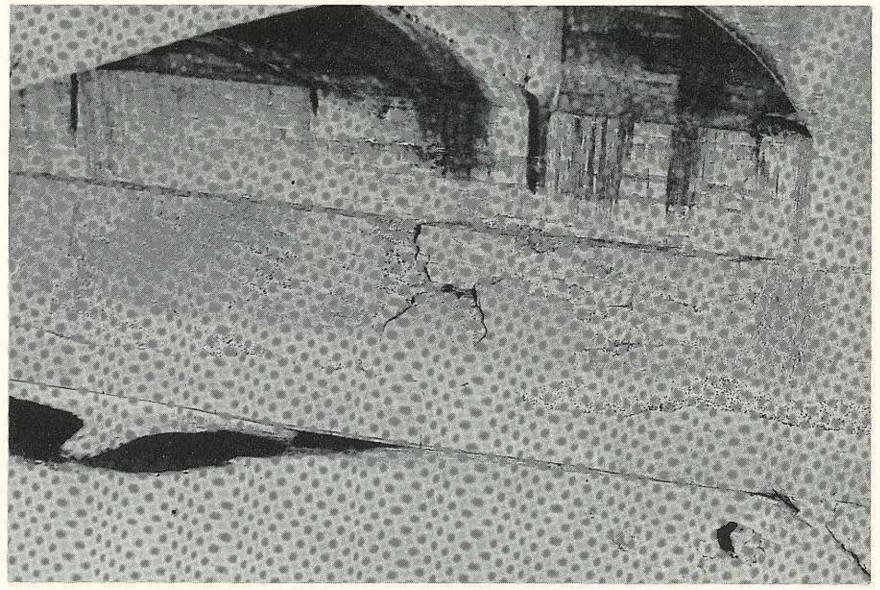
Pier 4, as seen in Photo II-29, has an area of more severe deterioration but the reinforcement is not bared and thus presents no repair problem.



II-27. *Wide vertical crack near the upstream edge of the north abutment.*



II-28. *Surface spalling at the downstream face of Pier 3.*

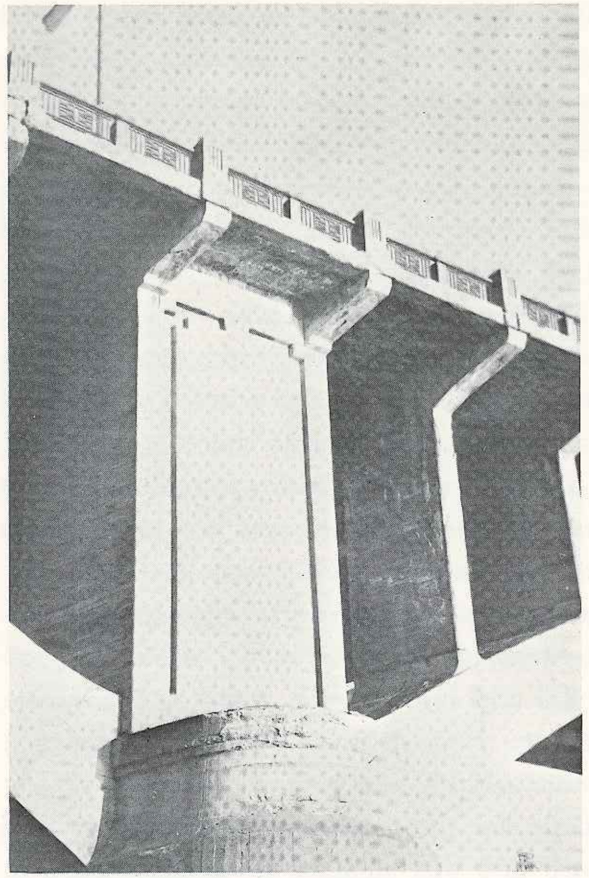


II-29. *Deterioration to the south wall of Pier 4.*

Borings show that the piers are structurally sound and that they are founded on undisturbed bedrock as discussed in Part III. The walls of all piers except Piers 1 and 7 are sound; Piers 1 and 7 have several vertical cracks in the upper two-thirds of the pier wall, completely penetrating the walls facing the river banks. Since the deck expansion joints are not above the pier wall, the concrete in the upper portion of the wall has not deteriorated, except at the edges of the cantilevers supporting the sidewalk which have some scale and spall. See Photo II-30 for a view of a typical pier.

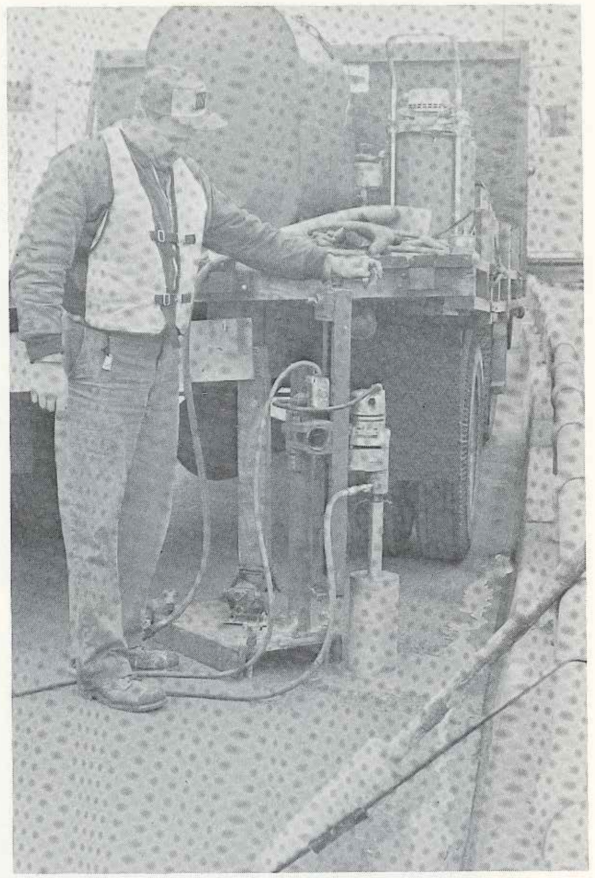
Core Sampling

Because the original design stresses were not indicated in the existing plans, it was assumed that the concrete used had an allowable strength of approximately 4,000 psi for the arch ribs, the barrel arches, the T-beams, and the deck, and a concrete strength of about 3,000 psi for the abutments, bents, spandrel columns and walls, and the pier walls. These assumptions are in accordance with known design stresses of similar bridges of the time.

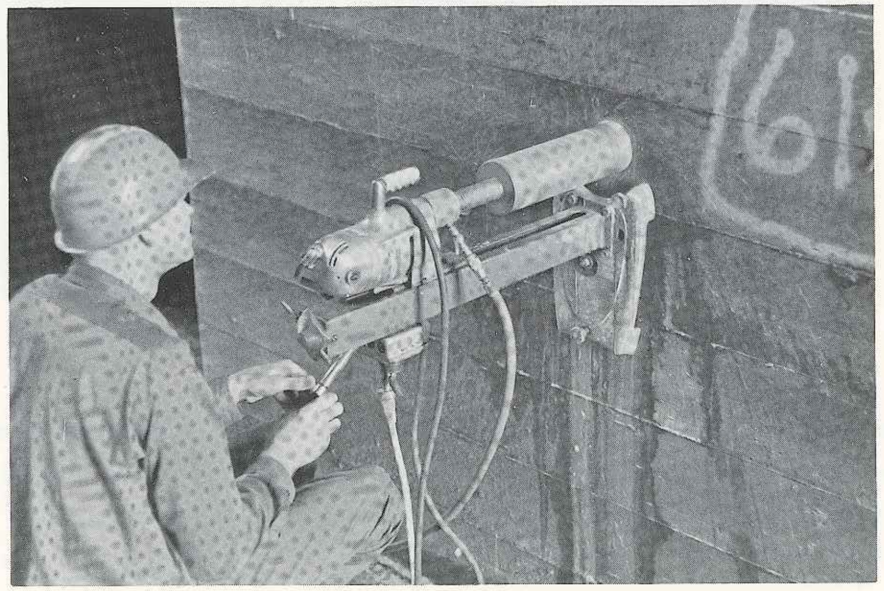


II-30. General view of the walls and the upper portion of the base of Pier 3.

II-31. Longyear Model 330 core drill used to obtain deck cores.

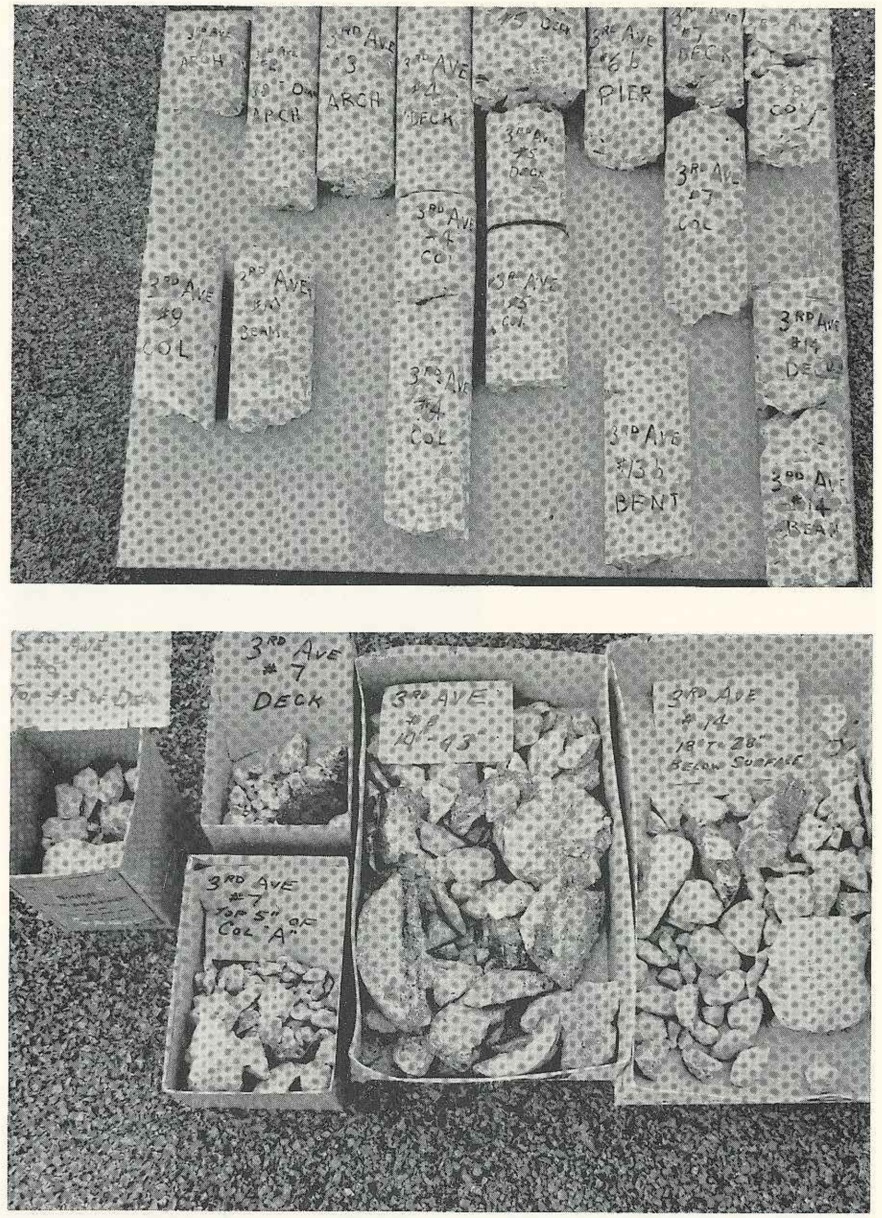


II-32. Drilling a core in a wall of Pier 8 with a Longyear Model 300.



To confirm these assumptions and to determine the state of the concrete in several key areas, it was considered necessary to take core samples at twelve locations of the bridge; see Exhibit 4 for exact locations. The coring firm of Capital Carbide, 1397 Selby Avenue, St. Paul, Minnesota, was retained for this phase. To obtain samples from the deck a Longyear Model 330 core drill with 4 inch and 6 inch cylindrical bits was used. A Longyear Model 300 drill with a cylindrical 4 inch bit was used to take samples of the other members of the bridge. Photos II-31 and II-32 show the two types of drills in operation and Photo II-33 shows the core samples recovered.

Core 1 (arch rib) was drilled down 9 inches but the sample recovered was 4 1/2 inches long which is too short for testing. Core 2 (arch rib) was drilled down 12 inches and recovered as 11 inches of sound concrete. Core 3 (barrel arch) was drilled down 12 inches and recovered as 10 inches of good

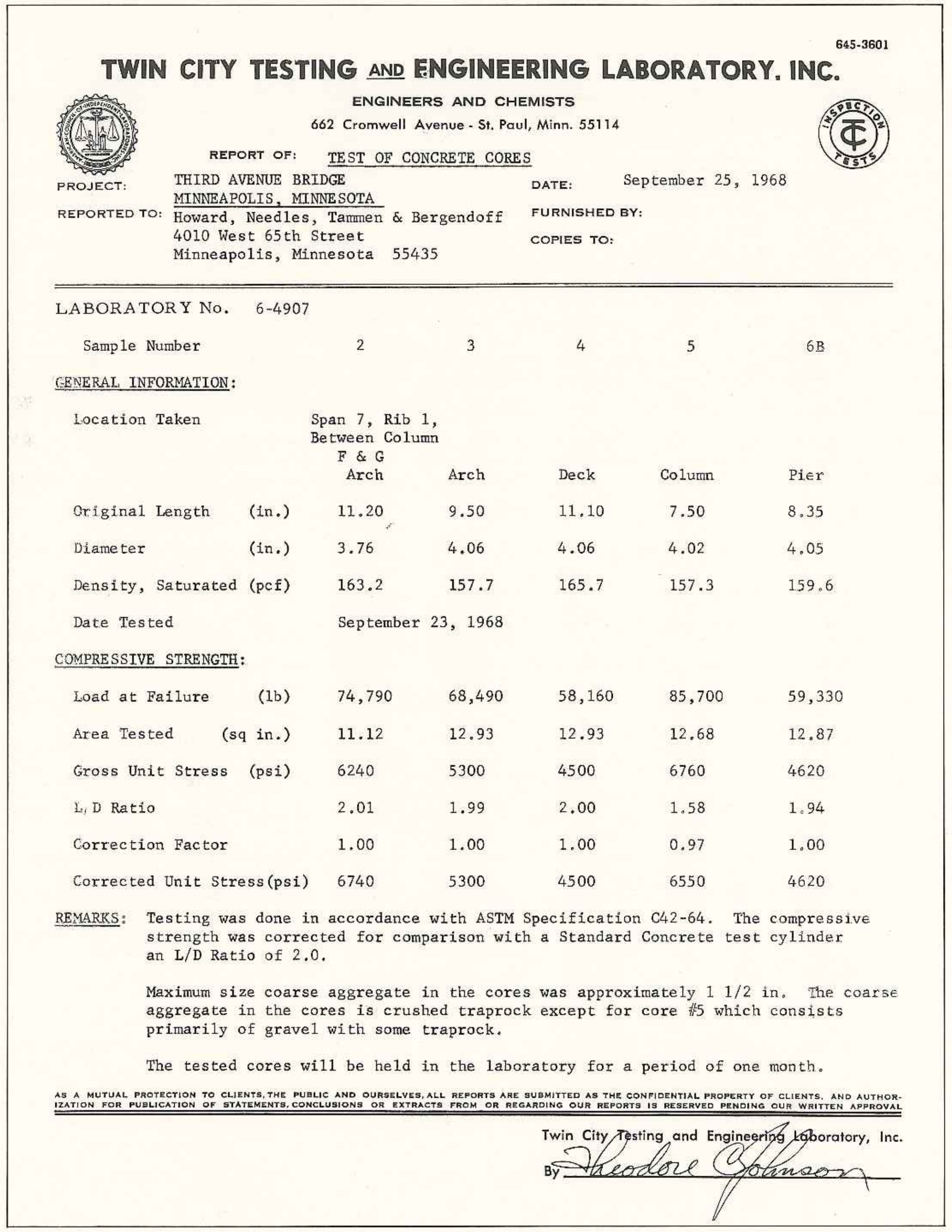


II-33. Cores as recovered from all coring operations.

concrete. Core 4 (deck to spandrel column) consisted of 8 inches of asphalt, 4 inches of sand/gravel fill, 11 1/2 inches of concrete deck, and 16 1/2 inches of concrete spandrel column (reinforcement hit at 5 1/2 inches). The 11 1/2 inch deck core was suitable for testing. Core 5 (deck to spandrel column) consisted of 3 1/2 inches of asphalt, 4 inches of granite blocks, 9 inches of fill material (sand with granite and concrete chunks), 12 1/2 inches of sound concrete deck, and 8 inches of good concrete in the spandrel column. The 8 inch core, which did not contain trap rock as coarse aggregate, was tested. Core 6 (pier wall) was recovered as 9 inches long. Core 7 (through a deck expansion joint into a spandrel) found 8 inches of asphalt, 4 1/2 inches of sand/gravel fill, 11 1/2 inches of concrete deck (7 inch sample recovered), and 14 inches of concrete, the top 5 inches being too deteriorated to recover. A 9 inch sample in the column was recovered but was not suitable for testing. Core 8 (through a deck expansion joint into a spandrel column) revealed 9 inches of mushy asphalt, 1 inch of sand, 14 inches of round concrete chunks, 6 inches of bare aggregate, 4 inches of mostly bare aggregate, 4 inches of round concrete chunks, and 11 inches of soft, cracked concrete, that broke into two sections upon removal. No core was suitable for testing. Core 9 (spandrel wall) consisted of an 8 inch sample (containing trap rock as coarse aggregate) recovered from a 10 inch hole. Core 10 (deck into T-beam) found 7 inches of good asphalt, 1 inch of sand, 7 inches of wet and soft concrete, 3 inches of concrete chips, and 9 inches of sound concrete. There is a definite construction joint between slab and beam of the T-beam and a 1 inch square compressive reinforcement was found 2 inches below the slab. An 8 1/4 inch core sample with loose aggregate was recovered and was suitable for testing. Core 13 (column of bent) was recovered as a 9 1/2 inch sample. Core 14 (deck into T-beam) showed 8 1/2 inches of asphalt, 3 inches of concrete fill material, 6 1/2 inches of concrete deck, 10 inches of concrete beam recovered as round concrete chunks, and 8 inches of solid concrete beam. The 6 1/2 inch deck section and the 8 inch beam section were suitable for testing.

Following inspection and evaluation by the Consultant, the selected ten samples that were suitable for testing were taken to Twin City Testing and Engineering Laboratory, Inc. 662 Cromwell Avenue, St. Paul, Minnesota, for further evaluation. Test results are given in Figure 3.

As shown, the allowable unit stresses of the cores from the arches exceed the assumed allowable stresses by 33% to 69%. For the deck, the allowable unit stresses of the cores suitable for testing exceeded the assumed



645-3601

TWIN CITY TESTING AND ENGINEERING LABORATORY, INC.

ENGINEERS AND CHEMISTS
662 Cromwell Avenue - St. Paul, Minn. 55114

REPORT OF: TEST OF CONCRETE CORES

PROJECT: THIRD AVENUE BRIDGE
MINNEAPOLIS, MINNESOTA

DATE: September 25, 1968

REPORTED TO: Howard, Needles, Tammen & Bergendoff
4010 West 65th Street
Minneapolis, Minnesota 55435

FURNISHED BY:

COPIES TO:

LABORATORY No. 6-4907

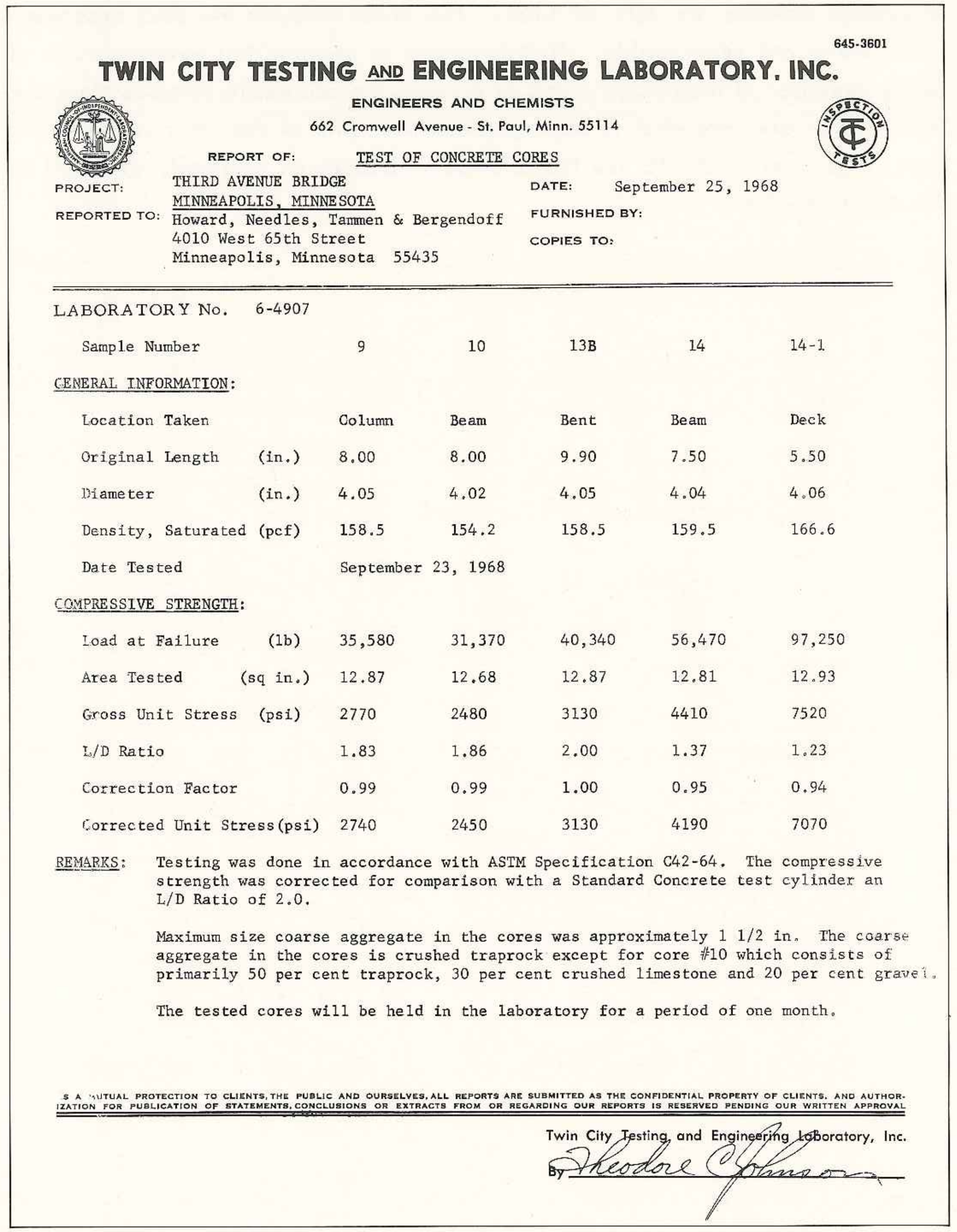
Sample Number	2	3	4	5	6B
GENERAL INFORMATION:					
Location Taken	Span 7, Rib 1, Between Column F & G Arch	Arch	Deck	Column	Pier
Original Length (in.)	11.20	9.50	11.10	7.50	8.35
Diameter (in.)	3.76	4.06	4.06	4.02	4.05
Density, Saturated (pcf)	163.2	157.7	165.7	157.3	159.6
Date Tested	September 23, 1968				
COMPRESSIVE STRENGTH:					
Load at Failure (lb)	74,790	68,490	58,160	85,700	59,330
Area Tested (sq in.)	11.12	12.93	12.93	12.68	12.87
Gross Unit Stress (psi)	6240	5300	4500	6760	4620
L/D Ratio	2.01	1.99	2.00	1.58	1.94
Correction Factor	1.00	1.00	1.00	0.97	1.00
Corrected Unit Stress(psi)	6740	5300	4500	6550	4620

REMARKS: Testing was done in accordance with ASTM Specification C42-64. The compressive strength was corrected for comparison with a Standard Concrete test cylinder an L/D Ratio of 2.0.

Maximum size coarse aggregate in the cores was approximately 1 1/2 in. The coarse aggregate in the cores is crushed traprock except for core #5 which consists primarily of gravel with some traprock.

The tested cores will be held in the laboratory for a period of one month.

Twin City Testing and Engineering Laboratory, Inc.
By Theodore Johnson



645-3601

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4010 West 65th Street
Minneapolis, Minnesota 55435

FURNISHED BY:

COPIES TO:

LABORATORY No. 6-4907

Sample Number	9	10	13B	14	14-1
GENERAL INFORMATION:					
Location Taken	Column	Beam	Bent	Beam	Deck
Original Length (in.)	8.00	8.00	9.90	7.50	5.50
Diameter (in.)	4.05	4.02	4.05	4.04	4.06
Density, Saturated (pcf)	158.5	154.2	158.5	159.5	166.6
Date Tested	September 23, 1968				
COMPRESSIVE STRENGTH:					
Load at Failure (lb)	35,580	31,370	40,340	56,470	97,250
Area Tested (sq in.)	12.87	12.68	12.87	12.81	12.93
Gross Unit Stress (psi)	2770	2480	3130	4410	7520
L/D Ratio	1.83	1.86	2.00	1.37	1.23
Correction Factor	0.99	0.99	1.00	0.95	0.94
Corrected Unit Stress(psi)	2740	2450	3130	4190	7070

REMARKS: Testing was done in accordance with ASTM Specification C42-64. The compressive strength was corrected for comparison with a Standard Concrete test cylinder an L/D Ratio of 2.0.

Maximum size coarse aggregate in the cores was approximately 1 1/2 in. The coarse aggregate in the cores is crushed traprock except for core #10 which consists of primarily 50 per cent traprock, 30 per cent crushed limestone and 20 per cent gravel.

The tested cores will be held in the laboratory for a period of one month.

Twin City Testing and Engineering Laboratory, Inc.
By Theodore Johnson

Figure 3
CORING RESULTS

allowable stresses by 50% to 136%. The cores through the deck expansion joints were not recoverable. It is incorrect to assume that every portion of every member of the bridge meets or exceeds the allowable stresses from the tests, which are intended to be representative only of the various elements. Thus, it is reasonable to use the allowable stresses as originally assumed in the design analysis.

PART III ENGINEERING GEOLOGY - FOUNDATION INVESTIGATIONS

AREA GEOLOGY

The rocks exposed at the surface in metropolitan Minneapolis–St. Paul are geologically relatively old. The marine and continental sediments deposited during the Cambrian and Ordovician Periods were later consolidated and formed sandstone, limestone, and shale. The area bedrocks are overlain by varying thicknesses of Glacial Drift of the Pleistocene Period and River Alluvium of the Recent Period. The general stratigraphy of the study area is:

Period	Formation	Average Thickness, Ft.	Approx. Range* in Thickness (Ft.)
Recent	River Alluvium	-	0-150
Pleistocene	Glacial Drift	100	0-400
Ordovician	Galena	(Top Eroded)	0- 20
	Decorah Shale	75	0- 75
	Platteville Limestone	30	25- 35
	Glenwood Beds	5	2- 7
	St. Peter Sandstone	158	145-165
	Shakopee Dolomite	45	35- 60
	New Richmond Sandstone	11	0- 15
	Oneota Dolomite	80	70- 90
Cambrian	Jordan Sandstone	90	80-105
	St. Lawrence Formation	180	160-200
	Franconia Sandstone	65	45- 80
	Dresbach Formation	155	125-200

* From Well Logs

The structure foundations for the Third Avenue Bridge involve the River Alluvium, Glacial Drift, and that portion of the Ordovician through the St. Peter sandstone.

SUBSURFACE EXPLORATIONS

The initial phase of this investigation included an office review of available geological literature, original (1913) foundation explorations, records of the bridge construction during the period, and water well logs and foundation boring data collected through the Minnesota Department of Highways, City of Minneapolis, U.S. Army Corps of Engineers, and Minnesota Geological Survey. It was concluded that the original explorations provided considerable documentation as to foundation conditions at most of the substructure units. Foundation borings were deemed necessary to verify conditions at River Piers 5, 6, and 7 and the north and south land approaches. With the approval of the Minnesota Department of Highways, an agreement was entered into with Soil Engineering Services, Inc. to perform the drilling, sampling, and coring operations required for borings at the selected land and river piers.

Foundation investigations were planned with the intent of developing a generalized geological profile along the project alignment and to develop detailed profiles wherever considered necessary. Standard penetration tests were performed at selected intervals in the Glacial Drift and Recent Alluvium overburden with the borings being taken to top of rock in all cases. NK size cores were taken in the rock. Samples of the sandstone were recovered by the split barrel sampler used in conjunction with the Standard Penetration Test. All planned field explorations were completed between April 8, 1968, and July 23, 1968.

The locations of all borings completed in this foundation investigation are shown on Exhibit 5. Generalized geological profiles are shown on Exhibits 5 and 6 and the actual Log of Borings are included as Appendix B.

OBSERVATIONS AND CONCLUSIONS

Review of borings completed in this foundation investigation and also the initial phase office review of data indicates the following general conclusions with respect to existing pier founding conditions.

South Approach

The south abutment and Piers B-1, B-2, and B-3 comprise the land approach from the south side of the Mississippi River. Access for borings could not be obtained for Boring 3-2 originally planned to define the subsurface conditions in the vicinity of Piers B-1 and B-2. The top of the Platteville Limestone formations varies from Elevation 801.9 near the south abutment to Elevation 785.7 at Pier B-3. The Platteville Limestone at this location is overlain by surficial, miscellaneous fill, including rock and concrete debris and Glacial Drift. The south abutment and Piers B-1 and B-3 appear to be founded on soil-bearing footings in the dense Glacial Drift. The plan footing elevation for Pier B-2 indicates that in all probability it is founded in or near the top of the Platteville Limestone.

Arch Pier P-1

The original 1913 borings indicate that with an average founding elevation of 786.5, this pier is immediately underlain by approximately 10 feet of the Platteville Limestone.

Arch Pier P-2

Five borings made in the area of this footing prior to construction revealed approximately 5 to 8 feet of Platteville Limestone would remain in place beneath the average pier founding Elevation 786.

Arch Pier P-3

The original 1913 borings near this pier revealed approximately 6 feet of Platteville Limestone would remain in place beneath the average pier founding Elevation 784.

Arch Pier P-4

The original 1913 borings near this pier revealed approximately 4 feet of the Platteville Limestone would remain in place beneath the average pier founding Elevation 784.

Arch Pier P-5

Borings 3-4 and 3-5 made through the upstream and downstream edges of this pier indicated approximately 9 feet of Platteville Limestone in place beneath an average pier founding Elevation 783. Borings alongside the east face of this pier confirmed the existence of 9 feet of the Platteville Limestone.

Arch Pier P-6

Three new borings (3-10, 3-11, and 3-12), drilled just east of this pier, confirmed a 7 foot thickness of the Platteville Limestone in place beneath the average pier founding Elevation 783.

Arch Pier P-7

Three new borings adjacent to this pier confirmed a 9 to 12 foot thickness of the Platteville Limestone in place beneath the average pier founding Elevation 784.

Arch Pier P-8

The original 1913 borings in the vicinity of this pier indicate a 10 foot thickness of the Platteville Limestone in place beneath the average pier founding Elevation 785.

North Approach

The north abutment and Piers B-4, B-5, and B-6 comprise the land approach from the north side of the Mississippi River.

Borings 3-16, 3-17, 3-18, and 3-19 drilled in this area indicated the top of Platteville Limestone formation rises from Elevation 791 to 801. The Platteville Limestone is overlain by miscellaneous fill and Glacial Drift. The plan footing elevations for Piers B-4, B-5, and B-6 and the north abutment indicate that these piers are founded on soil-bearing footings in fill or Glacial Drift.

General Observations

Detailed study of the original 1913 borings and the twelve supplementary river borings completed as part of this 1968 investigation indicates that Arch Piers P-1 thru P-8, located immediately adjacent to or within the Mississippi River, are founded on the Platteville Limestone. The general material condition of the river piers is excellent and, with the recent confirmation of the 4 to 12 foot thick Platteville Limestone as the founding material beneath these piers, it may be concluded that Arch Piers P-1 thru P-8 will continue to provide the required support for the existing bridge, as well as the proposed alteration of the main bridge deck and the arch spans.

The borings indicate that possibly all substructure units for the north and south approaches are founded on soil-bearing footings in the miscellaneous fill or Glacial Drift. In the event any additional loading is anticipated for these piers as they now exist, or if consideration is to be given to construction of new approaches involving different span lengths, a final design exploration program will be required to permit further evaluation of these foundation units.

PART IV REBUILT STRUCTURE

GENERAL

Results of the Consultant's investigation and evaluation, which includes visual inspection, foundation exploration, material testing, and design analysis suggest that: (1) the approaches be reconstructed entirely, (2) the deck, sidewalks, and railing above the arch spans be reconstructed, (3) the piers, arches, and spandrels be repaired or reconstructed as necessary.

A more detailed discussion of the above can be found in the section of Part IV titled "Reconstruction."

LAYOUT AND TRAFFIC STUDIES

Because of the proximity of the Third Avenue bridge to Downtown Minneapolis, the proposed deck layout of four 12'-0" lanes flanked by curb reaction widths of 3'-9", one-line traffic railings and 13'-6" sidewalks is highly functional and aesthetically pleasing. The wide four lane roadway balanced by broad sidewalks provides for ease in maneuvering around a stalled vehicle and in snow removal; the spacious sidewalks allow rapid snow removal and encourage viewing the river and rail activity.

Considering the varied functions of the bridge and the geometry of the repairable portions, the proposed deck layout provides the most suitable arrangement for present and future needs.

Traffic records maintained by the Minneapolis Traffic Bureau indicate that adjusted traffic counts at the bridge on May 15 and 16, 1968, averaged 32,592 vehicles per day (VPD), a considerable increase over the adjusted 1966 count of 24,002 VPD. This increase in traffic reflects the restriction imposed on trucks and buses for the portion of the Hennepin Avenue Bridge between Nicollet Island and Northeast Minneapolis.

RATING OF EXISTING MEMBERS

A design analysis was made of the three types of arch ribs and the barrel arches to determine their live load carrying capacity. The same was not done for the members of the approach spans because it is believed that these members cannot economically be repaired, and, hence, require replacement.

The ratings were based on the following criteria:

1. The analysis is based on the assumed allowable stresses for appropriate material which would be used in design rather than the unit working stresses allowed in determining the load-carrying capacity of a member crossed by a vehicle operating under a special overload permit.

2. The assumed allowable stresses of the arch ribs and the barrel arches are in accordance with known design stress of similar bridges of the same era and are less than the ultimate stresses found in testing representative core samples. They are summarized as follows:

Concrete:	Ultimate Compressive Strength	$f'_c = 4,000$ psi
	Design Strength	$f_c = 1,600$ psi
	Shear	$f_v = 90$ psi
	Modular Ratio E_s/E_c	$n = 8$
Steel Reinforcement:	Deformed Bars	$f_s = 18,000$ psi
	Fabricated Shapes	$f_s = 18,000$ psi

These stresses are in accordance with the 1965 Specifications of the American Association of State Highway Officials (AASHO).

3. Ratings of the load carrying capacity of members of the bridge are given as a loading proportional to the AASHO H or HS truck loading. For example, the H20-44 truck designates a 20 ton truck using the loading specification adopted by AASHO in 1944. This consists of a two-axle truck, the front axle carrying 4 tons and the rear axle, spaced 14 feet behind the front axle, carrying 16 tons. The HS20-44 loading is the H20-44 truck followed by a 16 ton axle weight trailer, the distance between the rear axle of the truck and the trailer axle varying from 14 feet to 30 feet. Thus, if the capacity of a member is 80% of the force in the member due to the HS20-44 truck-trailer combination, the allowable axle loads would be 3.2 tons, 12.8 tons, and 12.8 tons; and the rating would be HS16-44. Similiarly, if the capacity of a member is 20% greater than the force in the member due to the HS20-44 truck-trailer, the rating would be listed as HS24-44.

In rating the members of the bridge, the three types of arch ribs and the barrel arches were the only members considered. The piers and the spandrel columns and walls would have ratings far exceeding the ratings of the arch members, the members of the approach spans are not considered repairable so they were not rated, and the unknown state of the deck above the arch spans rules out the rating of the deck. The ratings are as follows:

211'-0" Ribbed Arch

10'-0" Rib	HS27.8-44
12'-0" Rib	HS27.0-44
16'-0" Rib	HS21.6-44

134'-0" Barrel Arch	HS69-44

The rating of HS21.6-44 of the 16'-0" rib of the ribbed arch (controlled by the stress in the steel at the springing line) is the maximum bridge rating. Since there is no deterioration in the concrete and hence in the steel in the vicinity of the piers and because there would be no stress reversals in the steel, it is reasonable to assume that the proposed HS20-44 loading can be adequately supported. The other three ratings are controlled by the stress in the concrete and allow an adequate margin of safety for variations in material, loading, and deterioration.

REPAIR

Repairs are made to basically sound components of the structure, to restore members to their original design strength, and to impede further deterioration. If the repairs are made in accordance with the proposals outlined herein, it is reasonable to assume that the reconditioned structure will have a serviceable life equivalent to that of a new structure. Repairs consist of the following four basic operations:

1. Replacement of deteriorated and/or ineffective components. This operation consists of replacement of the approach spans and the deck system above the arches. In addition, it consists of assessing the condition of the concrete and reinforcement in the spandrel columns and walls, and the pier walls, and repairing or reconstructing these components as required.

2. Restoration and/or replacement of corroded reinforcing steel. For areas in which the reinforcement is bared, the concrete must be chipped away to expose a length of undeteriorated steel at each end and the steel area must be sandblasted. If the area of the section has been reduced through corrosion to less than 50% of the original, the corroded length must be removed and replaced by a new bar of equivalent area, lapped 10 diameters, and welded to the sound material at each end.

3. Replacement of spalled or deteriorated concrete areas. In all areas where the concrete has deteriorated, it must be removed, the extent depending on the proximity to sound concrete. In general, it is suggested that shotcrete be used to replace the concrete in areas of

spalls on the vertical and bottom horizontal surfaces and that poured concrete be used on top horizontal surfaces and all areas in which the complete section must be replaced. In the repair of surface areas or edges, it is necessary to remove all deteriorated or loose concrete and square-up the edges to avoid "feathering" of the new concrete.

Shotcrete is concrete applied with a "cement gun" – a relatively dry mix of sand and cement is carried by compressed air, mixed with water, and "shot" onto the surface, resulting in a deposit of dense, uniform concrete with good adherence. Keys are notched into the concrete to which the shotcrete is to be applied. If the thickness of the shotcrete is to be 2 inches or more, 2 x 2 x 12 mesh (12 gage wire spaced 2 inches each way) with anchors at 18 to 24 inches on center should be used.

Although shotcrete costs three to four times more per unit volume than poured-in-place concrete, it eliminates the necessity of conventional forming and is more economical to use in the repair of areas of shallow deterioration or areas difficult to form and/or pour. It should be noted that the effectiveness of the placed shotcrete depends almost entirely on the surface preparation and the diligence of the operator.

In the repair of hairline cracks in the arch ribs, the longitudinal and transverse cracks in the barrel arches, and the vertical cracks in the pier and spandrel walls, it is recommended that the cracks be thoroughly cleaned and then sealed with epoxy. If cleaning reveals deteriorated concrete, the area will require chipping out and sealing with epoxy mortar. See Exhibit 7 for typical examples of the above types of deterioration and the proposed repair procedures.

4. Provision for protection against further deterioration. The two main causes of deterioration have been the improper removal of the runoff water from the deck and the lack of sufficient concrete covering the reinforcement. The deck drainage system has completely lost its effectiveness in channeling the runoff away from the structure. Downspouts from the deck catch basins have become plugged or corroded, allowing the water to drain into the cemented sand fill area between the asphalt wearing coarse and the concrete deck. This water then seeps through the deck, leaches salts from the fill material and the concrete and deposits them in the form of scale and mineral deposits on the under surface of the deck. In addition, water seeping through the deck expansion joints causes considerable deterioration to the concrete in the deck and spandrel columns or walls in the vicinity of these expansion joints. The lack of sufficient concrete covering the reinforcement has allowed moisture to reach and corrode the steel; the accompaning expansion causes the spalling and the cracking of the concrete. The above deterioration is further accelerated by the freezing–thawing cycle and the use of corrosive salts for snow removal.

To prevent recurrence of the deterioration caused by water, it is recommended that:

1. Neoprene seal expansion joints be used in lieu of the mastic filled joints.

2. An extensive drainage system using catch basins at the curbs with downspouts be used to channel the runoff through a closed system, directly to the river or storm sewer system.

3. The surface of all members to be reused in the rebuilt structure be treated to seal out the water and corrosive chemicals.

An analysis was made of the wind stability of the spandrel columns and walls when the deck is removed. It was found that the maximum stable heights of the spandrel columns and the spandrel walls would be 24 feet and 28 feet respectively. Thus it is recommended that spandrel columns "A" and "N" of the ribbed arches be laterally braced when the deck above is removed.

UTILITIES

At present, the active utilities found in the bridge, grouped with respect to the owners, are:

Minneapolis Traffic Bureau – Electrical conduits for the bridge lights, in the curb portion of the traffic rails between the roadway and the sidewalk.

Northern States Power Co. – In the slab of the upstream sidewalk: two 3 1/2″ ∅, four 4″ ∅, and two 4 1/2″ ∅ conduits. In the slab of the downstream sidewalk: six 3″ ∅ and four 3 1/2″ ∅ conduits.

Northwestern Bell Telephone Co. – In the slab of the upstream sidewalk: a "telephone cable runway" of reinforced concrete 2′-9″ wide and 6 to 8 inches deep covered by a 3′-3″ wide ribbed type steel roof. In the slab of the downstream sidewalk: four 4″ ∅ fiber ducts.

Space in the new deck for the following conduits has been informally requested:

Minneapolis Traffic Bureau – one 2″ ∅ conduit under each sidewalk for bridge lighting and one 3″ ∅ conduit for traffic signal lights.

Northern States Power Co. – approximately twelve to fifteen 4″ ∅ galvanized ducts with pull-through manholes every 400 to 500 feet.

Northwestern Bell Telephone Co. – approximately twelve to fifteen 4″ ∅ galvanized ducts with pull-through manholes every 900 feet maximum.

In view of the fact that the existing utility lines are either inside fiber ducts or a reinforced concrete box, both of which are embedded in mortar and enclosed between a 5 1/2 inch reinforced concrete sidewalk and the 10 inch bridge deck, it is reasonable to assume that continuity of service could not be provided during removal of the deck. It is therefore advisable, that an alternate utility system, temporarily rerouted possibly on a nearby bridge, be provided during deck removal and reconstruction.

RECONSTRUCTION

Arch Spans

In addition to removal of the asphalt wearing surface, cemented sand filler, street car tracks, and the granite blocks, it is recommended that the deck, sidewalk, railings, and all cantilevers be removed and that the spandrel columns and walls and the pier walls be removed as necessary. It is estimated that, on the average, the top 5 feet of the spandrel columns and walls at deck expansion joints and the top 1 foot of all other spandrel columns and walls and pier walls contain unsound concrete and need replacement.

Reconstruction of the spandrels and pier walls would include the attaching of new forms to the sound portion of the spandrels and the pier walls, replacing reinforcement as necessary, and pouring new concrete. A new reinforced concrete deck would then be poured and the railings, the sidewalks, and the lighting and drainage systems installed. Sectional views of the proposed deck above the ribbed arch and the barrel arch are shown in Exhibits 2 and 3, respectively. Proposed reconstruction of the spandrels is illustrated in Exhibit 8.

Approach Spans

It is recommended that a new deck supported by ten steel beams and four-column, reinforced concrete bents replace the existing approaches. Because of the location of Main Street and the railroad tracks, the new bents should be constructed in the same locations as the present bents; see Exhibit 8 for a proposed section.

Because the existing footings are not compatible with the proposed four-column bents (the south approach has individual footings for the three columns, and the north approach has individual footings for the five columns of Bent 4 and 5 and a continuous footing for Bent 6), it is recommended that new footings be constructed.

Figure 4 REBUILT THIRD AVENUE BRIDGE

The use of four-column bents was dictated by the width of the deck and the height of the bents. Aesthetically, the use of four slender columns and relatively shallow cap beams is more pleasing than three more massive columns and deep cap beams and would appear more balanced when considering the width of the deck.

Abutments and Retaining Walls

The location of the abutments would not change but new footings, abutment walls, and bridge seats are recommended. The wide, vertical cracks in the north abutment wall are believed to be deteriorated temperature cracks; hence the replacement. The accessible portions of the retaining walls of the approaches were visually inspected and appear to be structurally sound. A new slab above the walled part of the approaches as shown in Exhibit 8, is recommended to maintain the continuity of the deck and railings.

COST ESTIMATES

The nature and extent of repairs are based on conditions of the bridge in early 1968. Because of the poor condition of the deck, it was not feasible to estimate repair costs but to assume an entire new deck, utilizing current roadway width, traffic rail design, and more functional pedestrian railings.

The quantities are classified into removal and disposal, repair, and reconstruction. The unit prices for removal and disposal reflect the accessibility of the existing structure and the ease of disposal. The unit prices for

Figure 5

COST ESTIMATE

FOR REPAIR AND RECONSTRUCTION

Item	Quantity	Unit	Unit Cost	Amount
REMOVAL AND DISPOSAL:				
Approach Spans	Lump Sum			$ 55,550
Arch Spans	Lump Sum			466,000
REPAIR:				
Sandblasting Spalled Areas	3,500	S.F.	$ 4.00	14,000
Crack Repair - Epoxy	900	L.F.	4.00	3,600
Replacing Deficient Reinforcement	700	S.F.	40.00	28,000
Poured Concrete	2	C.Y.	150.00	300
Shotcrete	1,200	Bags	63.00	75,600
RECONSTRUCTION:				
Approach Spans				
South Approach	18,600	S.F.	17.50	325,500
North Approach	15,300	S.F.	17.00	260,100
Arch Spans				
Spandrels & Pier Walls				
Concrete	1,400	C.Y.	100.00	140,000
Reinforcement	95,500	Lbs.	0.15	14,325
Deck and Sidewalks				
Concrete	4,800	C.Y.	100.00	480,000
Reinforcement	736,400	Lbs.	0.15	110,460
Traffic Railings	3,000	L.F.	30.00	90,000
Pedestrian Railings	3,000	L.F.	32.00	96,000
Deck Expansion Joints	3,100	L.F.	2.00	6,200
Deck Drainage System	Lump Sum			18,200
Surface Treatment				
Deck	13,800	S.Y.	7.00	96,600
Spandrels, Arches & Piers	34,500	S.Y.	7.00	241,500
Deck Lighting System	Lump Sum			12,200
Navigation Lighting System	Lump Sum			1,000
Utility System Replacement				*
TOTAL CONSTRUCTION COST				$2,535,085
ENGINEERING & CONTINGENCIES				507,017
TOTAL ESTIMATED COST				**$ 3,042,102**

* Since the utility companies informally stated that new utility lines would be placed in the new deck, the costs of temporary relocation and replacement of the utility systems would probably be shared by the Department of Highways and the utility companies; thus, these were not included.

repair and reconstruction are based on current figures for construction in this area. Repair quantities for shotcrete, poured concrete, and reinforcement include a reasonable amount of overrun in the patching.

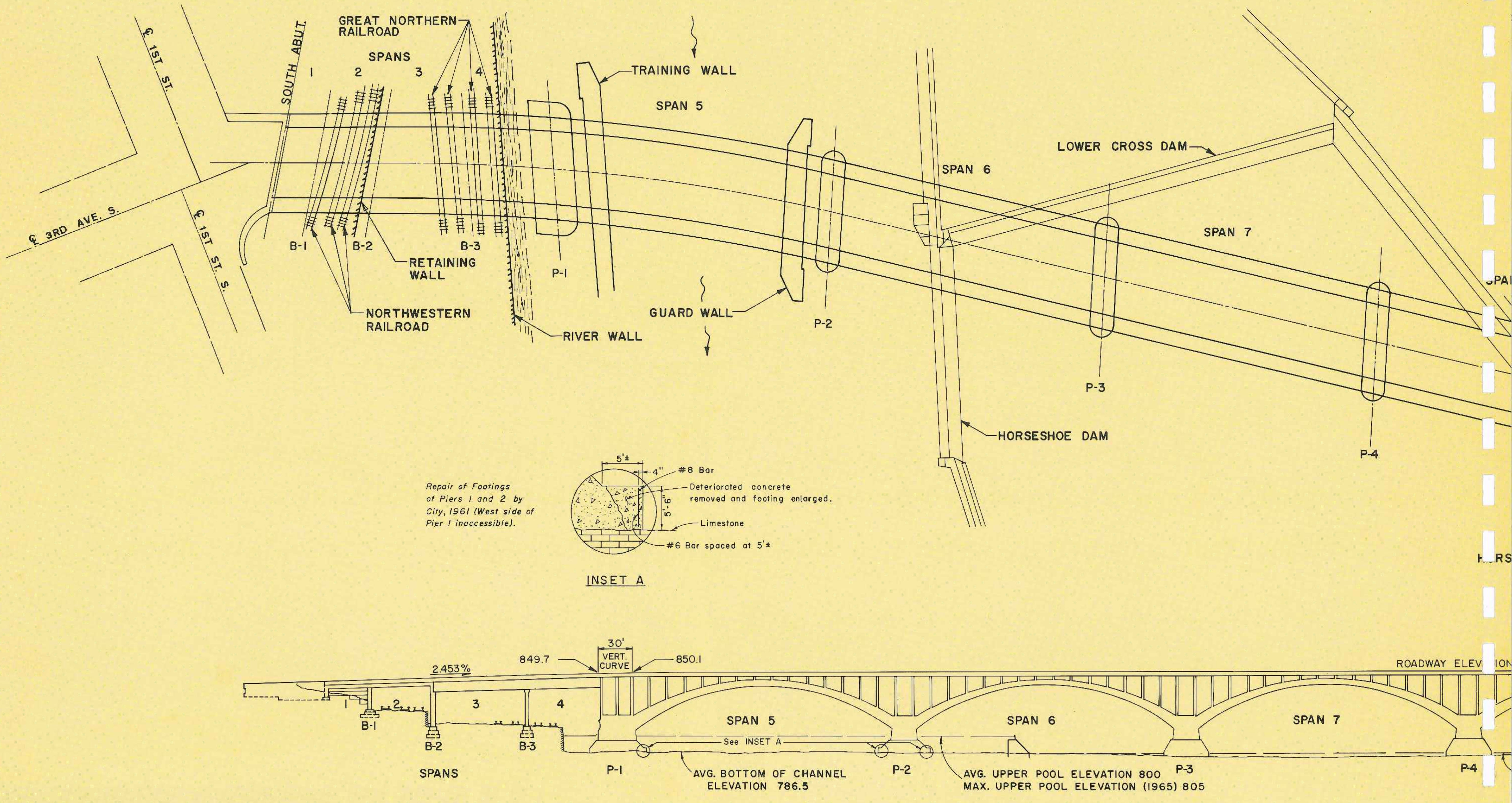
A breakdown of the estimated costs is as follows:

Unit	Unit Cost	Amount
Approaches	$22.69	$ 769,320
Arch Spans	18.33	2,272,782

The total cost of replacement of the Third Avenue Bridge with a new structure is estimated to be $8,100,000. This total is comprised of 2.3 million for removal of the existing structure, 4.8 million for a replacement structure, and 1.0 million for engineering and contingencies.

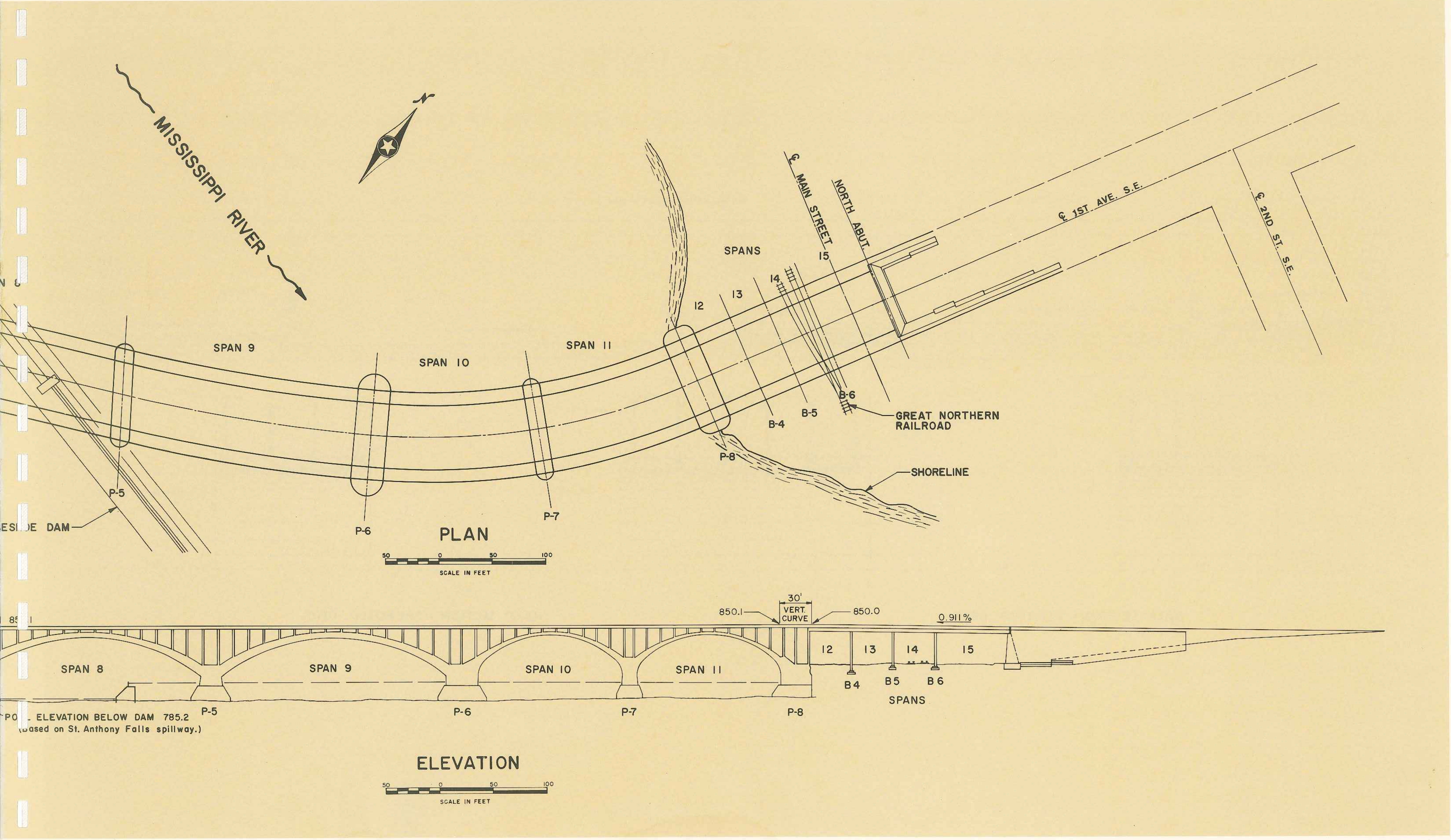
The cost of structure removal is based on the following unit costs: $1.75/S.F. for the south approach spans, $1.50/S.F. for the north approach spans, and $40/C.Y. for the arch spans. The cost of a replacement structure is based on the assumption that economical structure studies would be made and that the structure type would not have to conform to existing construction. Possible additional right-of-way is not included in the estimate for a replacement river crossing.

APPENDIX A


℄ 1ST. ST.
℄ 3RD AVE. S.
℄ 1ST. ST. S.
SOUTH ABUT.
GREAT NORTHERN RAILROAD
SPANS
1
2
3
4
B-1
B-2
B-3
RETAINING WALL
NORTHWESTERN RAILROAD
RIVER WALL
P-1
TRAINING WALL
SPAN 5
GUARD WALL
P-2
SPAN 6
LOWER CROSS DAM
SPAN 7
P-3
HORSESHOE DAM
P-4
SPA
H RS
Repair of Footings of Piers 1 and 2 by City, 1961 (West side of Pier 1 inaccessible).
5'±
4"
#8 Bar
Deteriorated concrete removed and footing enlarged.
5'-6"
Limestone
#6 Bar spaced at 5'±
INSET A
30'
VERT. CURVE
849.7
850.1
2.453%
ROADWAY ELEV ION
1
2
3
4
B-1
B-2
B-3
SPANS
SPAN 5
SPAN 6
SPAN 7
See INSET A
P-1
P-2
P-3
P-4
AVG. BOTTOM OF CHANNEL ELEVATION 786.5
AVG. UPPER POOL ELEVATION 800
MAX. UPPER POOL ELEVATION (1965) 805

GENERAL PLAN AND ELEVATION

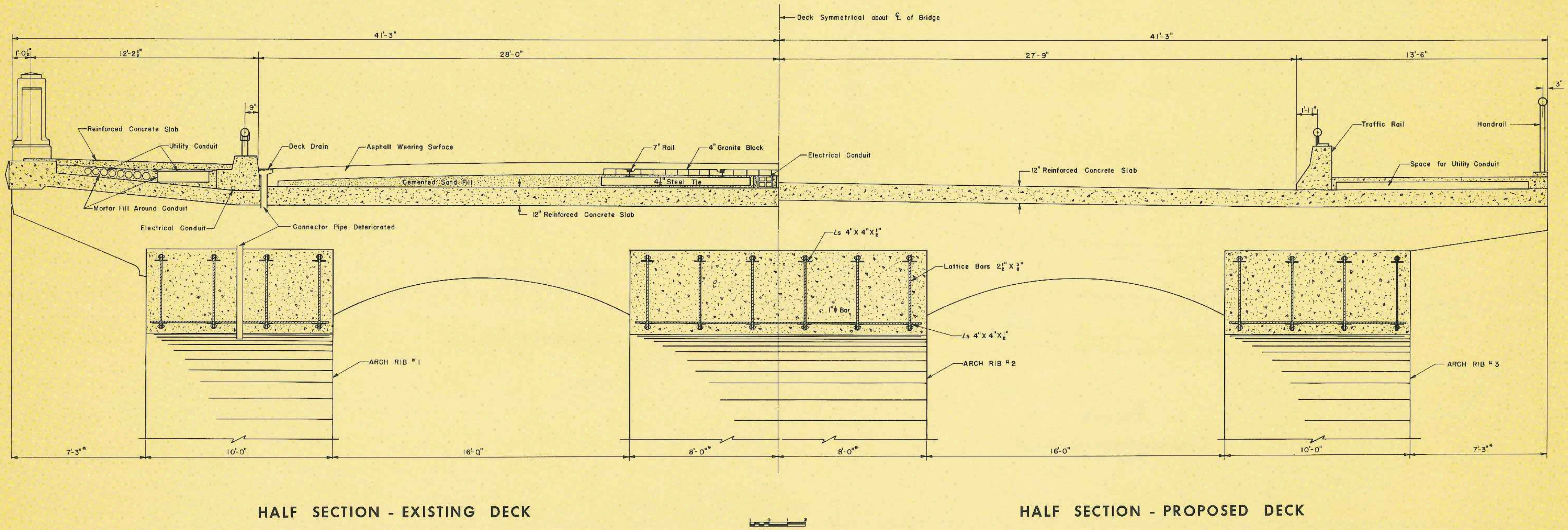
EXHIBIT 1



THIRD AVENUE BRIDGE

EXHIBIT 2

CROSS SECTION OF DECK ABOVE THE RIBBED ARCH



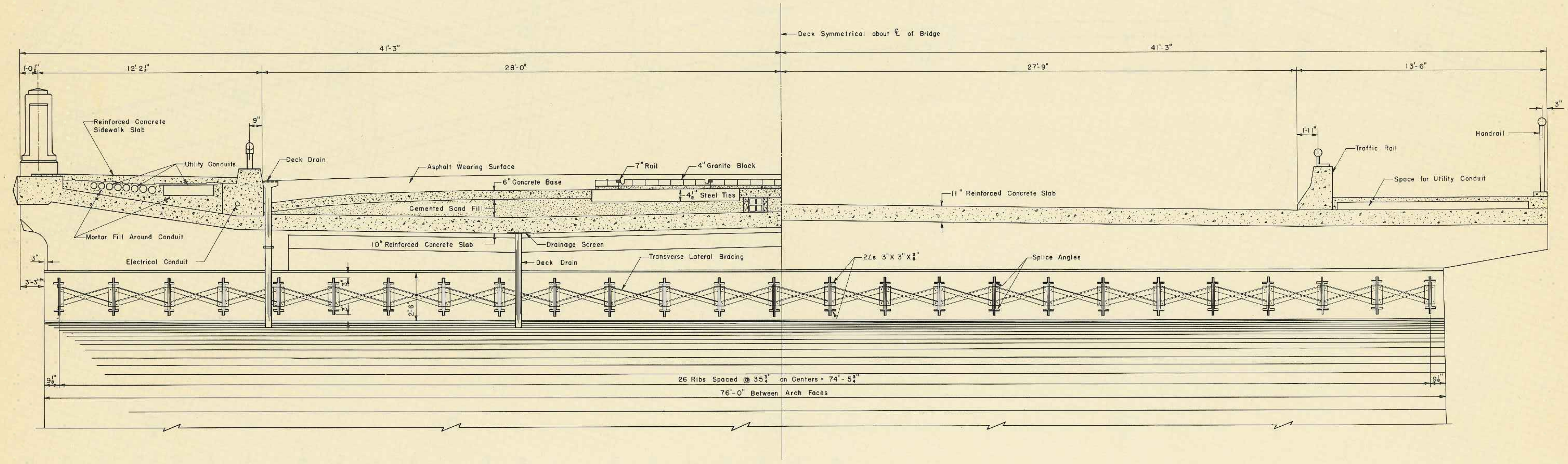
HALF SECTION - EXISTING DECK

HALF SECTION - PROPOSED DECK

* Varies on Curve.

CROSS SECTION OF DECK ABOVE THE BARREL ARCH

EXHIBIT 3

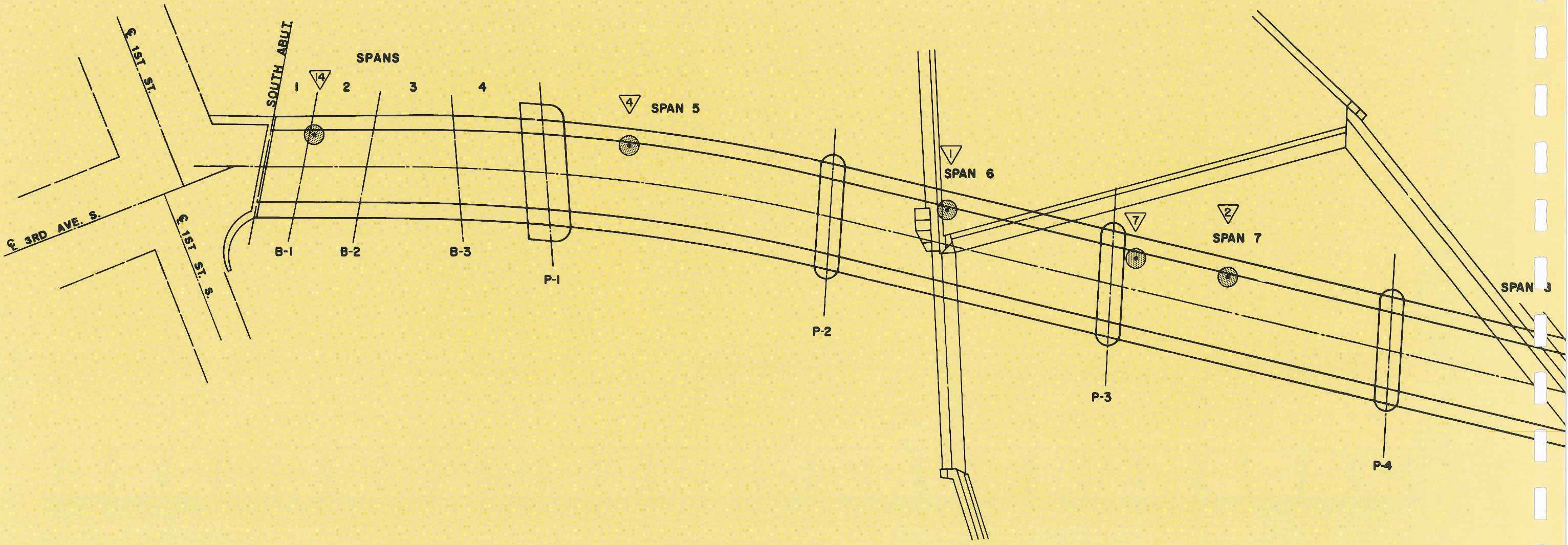


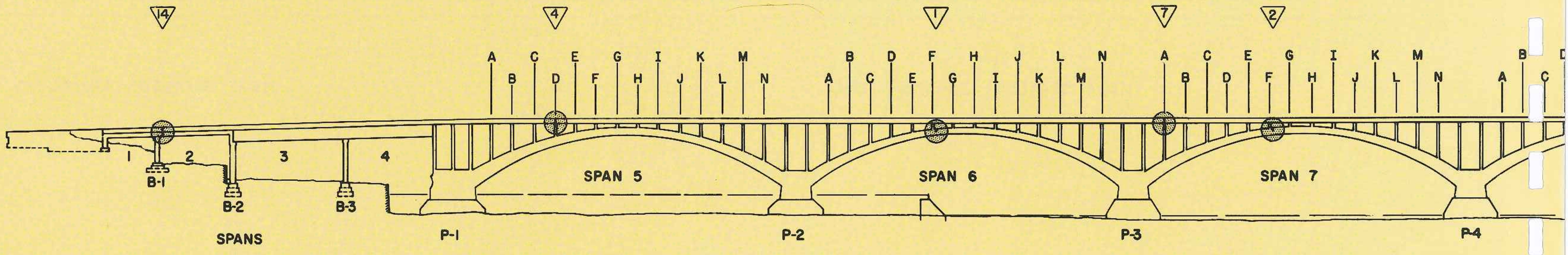
HALF SECTION - EXISTING DECK

SCALE IN FEET

HALF SECTION - PROPOSED DECK

* Varies on Curve.

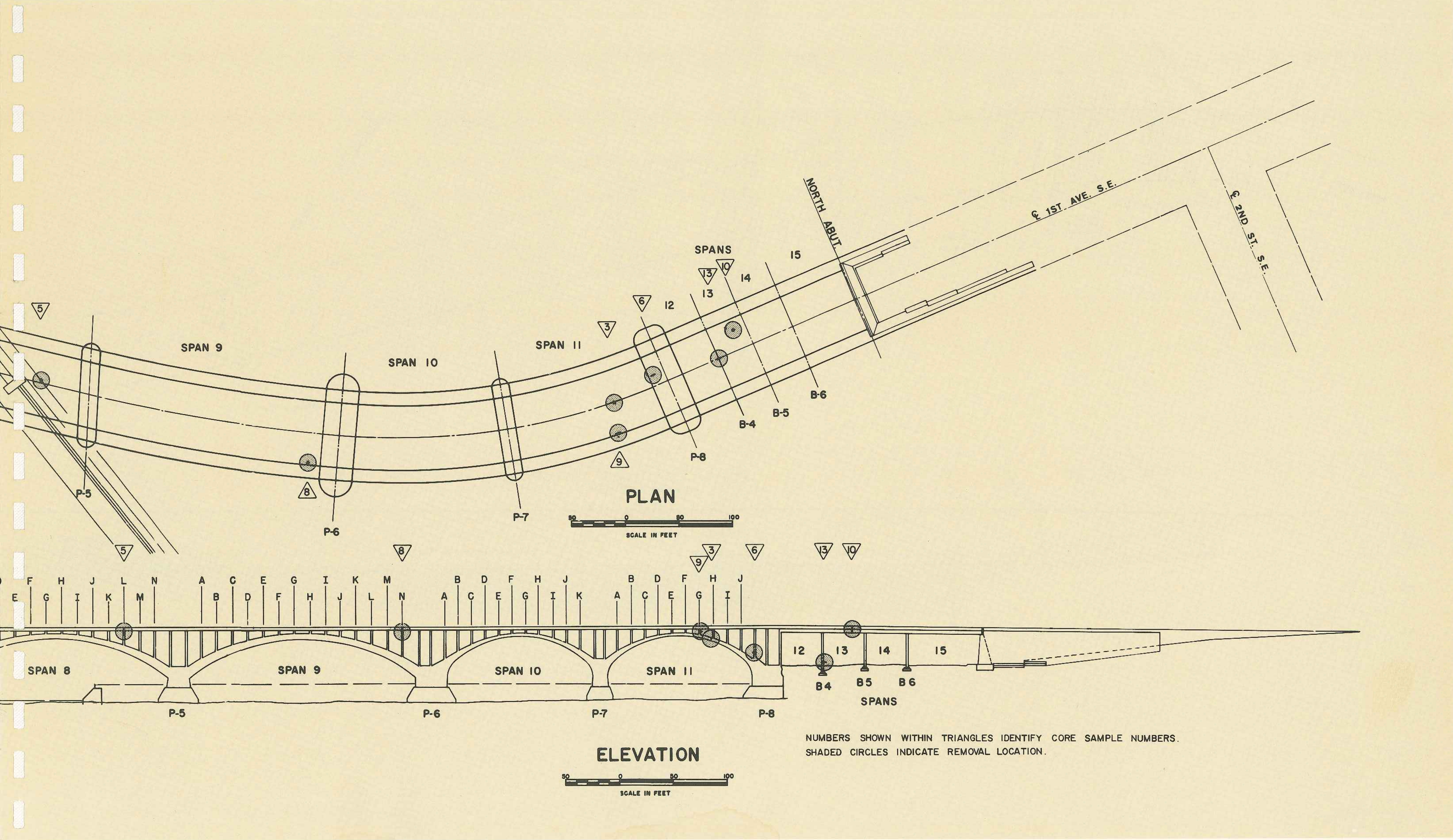




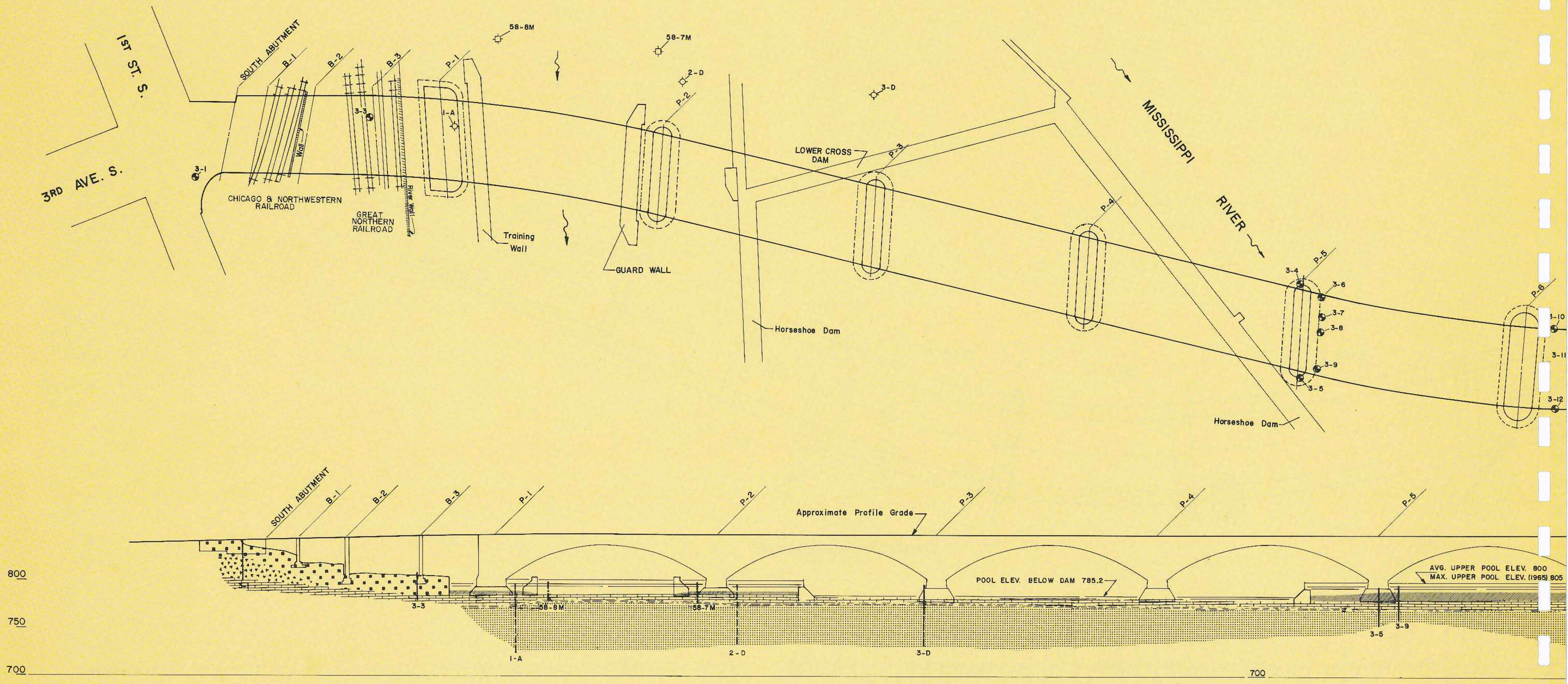
HOWARD, NEEDLES, TAMMEN & BERGENDOFF

LOCATION OF CORES

EXHIBIT 4

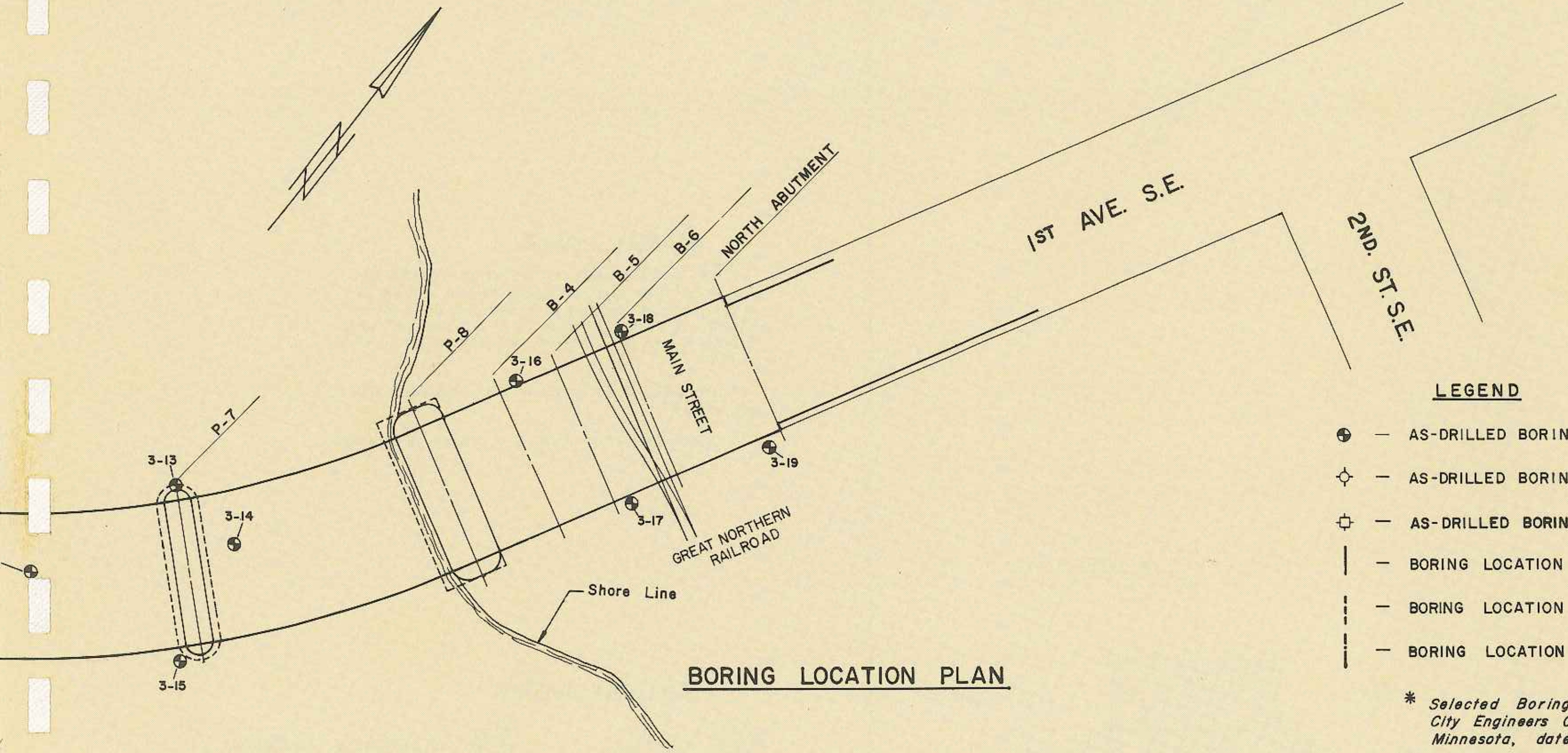


NUMBERS SHOWN WITHIN TRIANGLES IDENTIFY CORE SAMPLE NUMBERS.
SHADED CIRCLES INDICATE REMOVAL LOCATION.


1ST ST. S.
3RD AVE. S.
SOUTH ABUTMENT
B-1
B-2
B-3
P-1
P-2
P-3
P-4
P-5
P-6
58-8M
58-7M
2-D
3-D
3-1
3-3
1-A
3-4
3-5
3-6
3-7
3-8
3-9
3-11
3-12
Wall
River Wall
CHICAGO & NORTHWESTERN RAILROAD
GREAT NORTHERN RAILROAD
Training Wall
GUARD WALL
LOWER CROSS DAM
Horseshoe Dam
Horseshoe Dam
MISSISSIPPI RIVER
Approximate Profile Grade
POOL ELEV. BELOW DAM 785.2
AVG. UPPER POOL ELEV. 800
MAX. UPPER POOL ELEV. (1965) 805
800
750
700
700

BORING LOCATION PLAN AND GEOLOGICAL PROFILE

EXHIBIT 5



BORING LOCATION PLAN

LEGEND

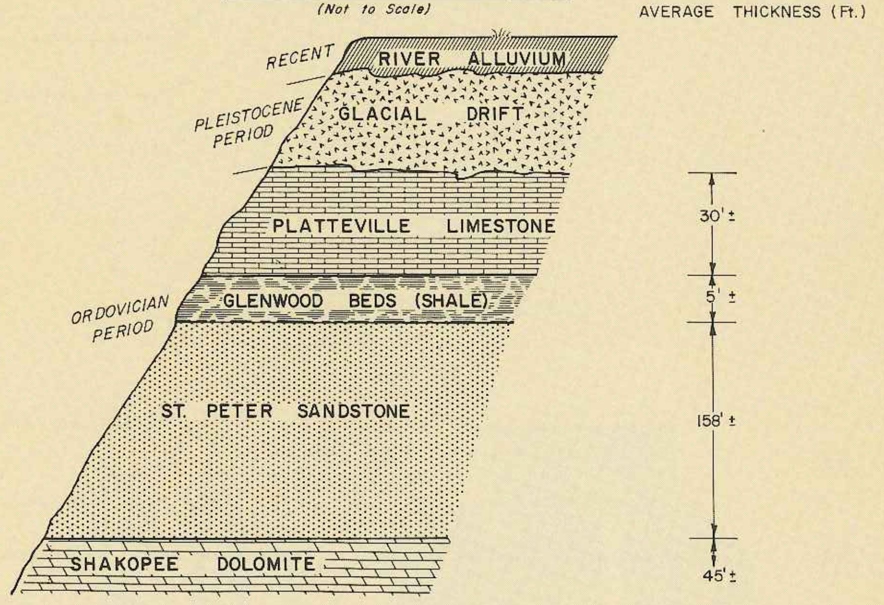
- ⊕ — AS-DRILLED BORING LOCATION (1968)
- ✧ — AS-DRILLED BORING LOCATION (1913) *
- ⌖ — AS-DRILLED BORING LOCATION (1958) **
- | — BORING LOCATION IN PROFILE (1968)
- ¦ — BORING LOCATION IN PROFILE (1913) *
- ¦ — BORING LOCATION IN PROFILE (1958)

* *Selected Boring from Drawing No. 77, Env. No. 5, City Engineers Office, Bridge Department, Minneapolis, Minnesota, dated 11-10-39.*

** *Selected Boring from drawings M-L STA 10/504 & 10/506, U.S. Army Engineer District, St. Paul, Corps of Engineers, St. Paul, Minnesota, dated March, 1959.*

STRATIGRAPHIC SECTION

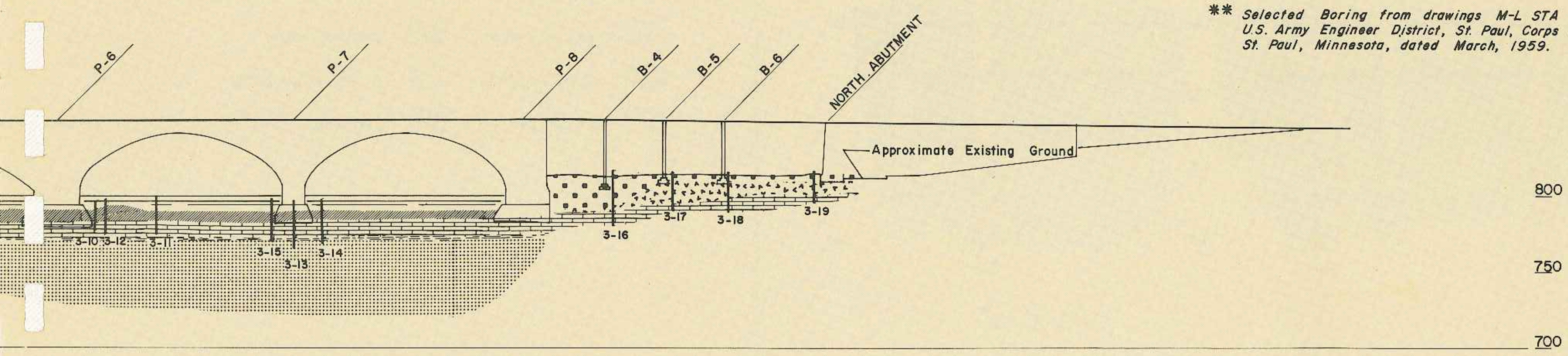
(Not to Scale)

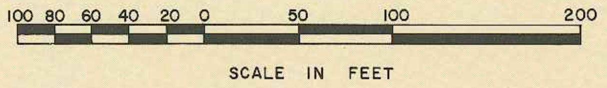


SUPPLEMENTAL STRATIGRAPHIC LEGEND

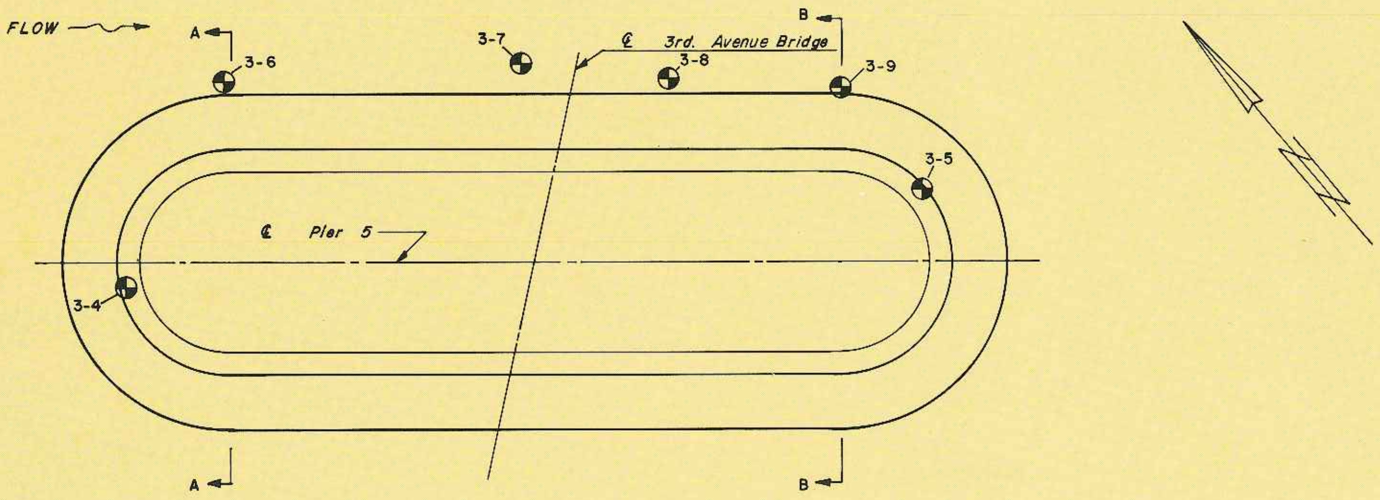
(No Scale)

- ⁘ MISCELLANEOUS FILL MATERIAL
- ▥ LIMESTONE FLOAT
- ▨ CONCRETE

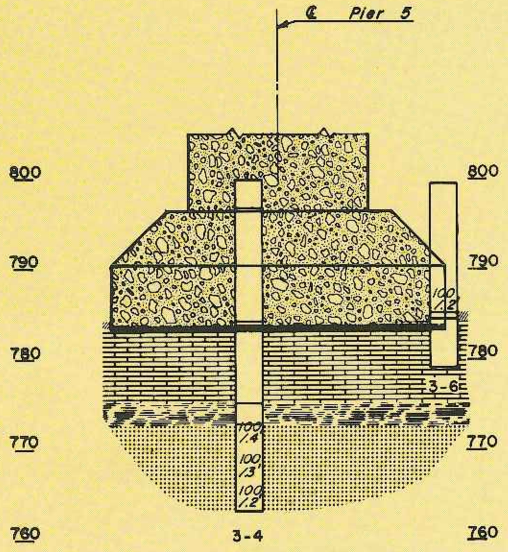




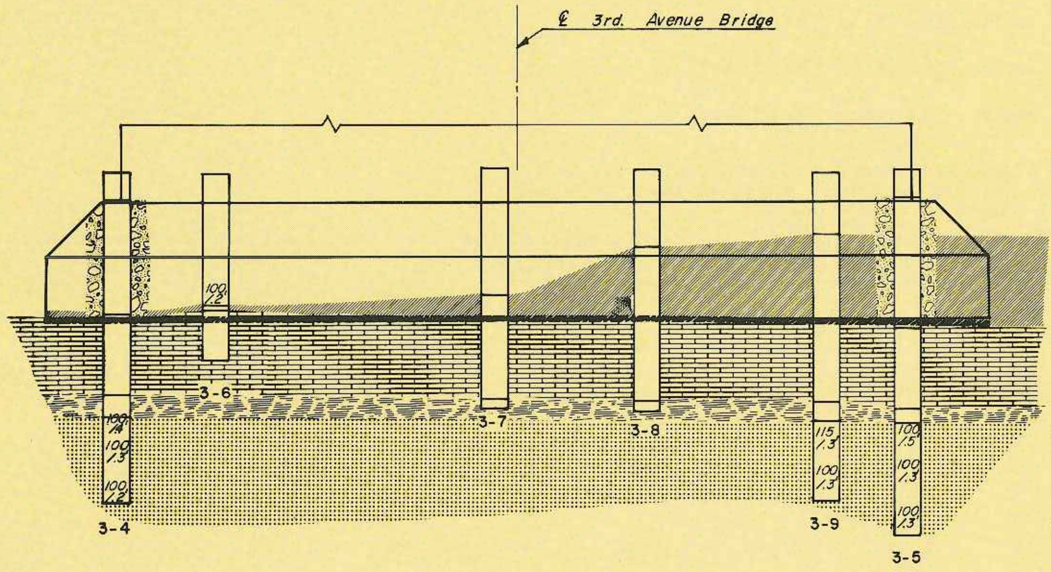
GEOLOGICAL PROFILE



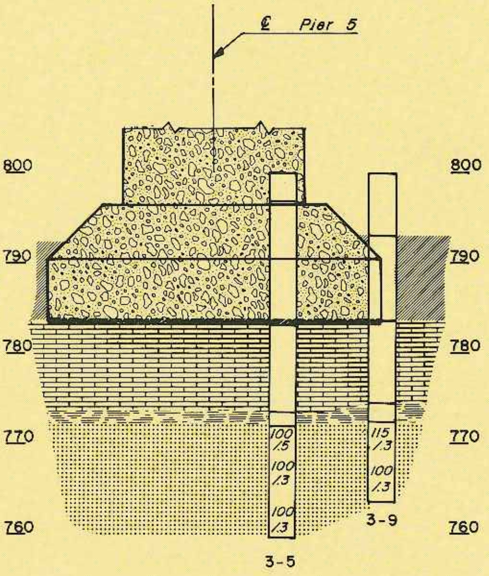
PLAN



SECTION A-A



ELEVATION



SECTION B-B

PIER 5

GENERAL NOTES

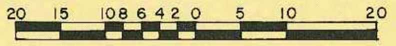
1. The numbers shown in the borings indicate the number of blows of a 140 pound hammer falling 30 inches required to drive a 2 inch O. D. Split Barrel Sampler 12 inches. Penetration of less than 12" inches is indicated as 100/.5' (Blows per Penetration in tenths of feet).

2. For plan view of structure and materials legend see Exhibit .

3. Approximate base of pier is shown as ▬.

MATERIALS LEGEND

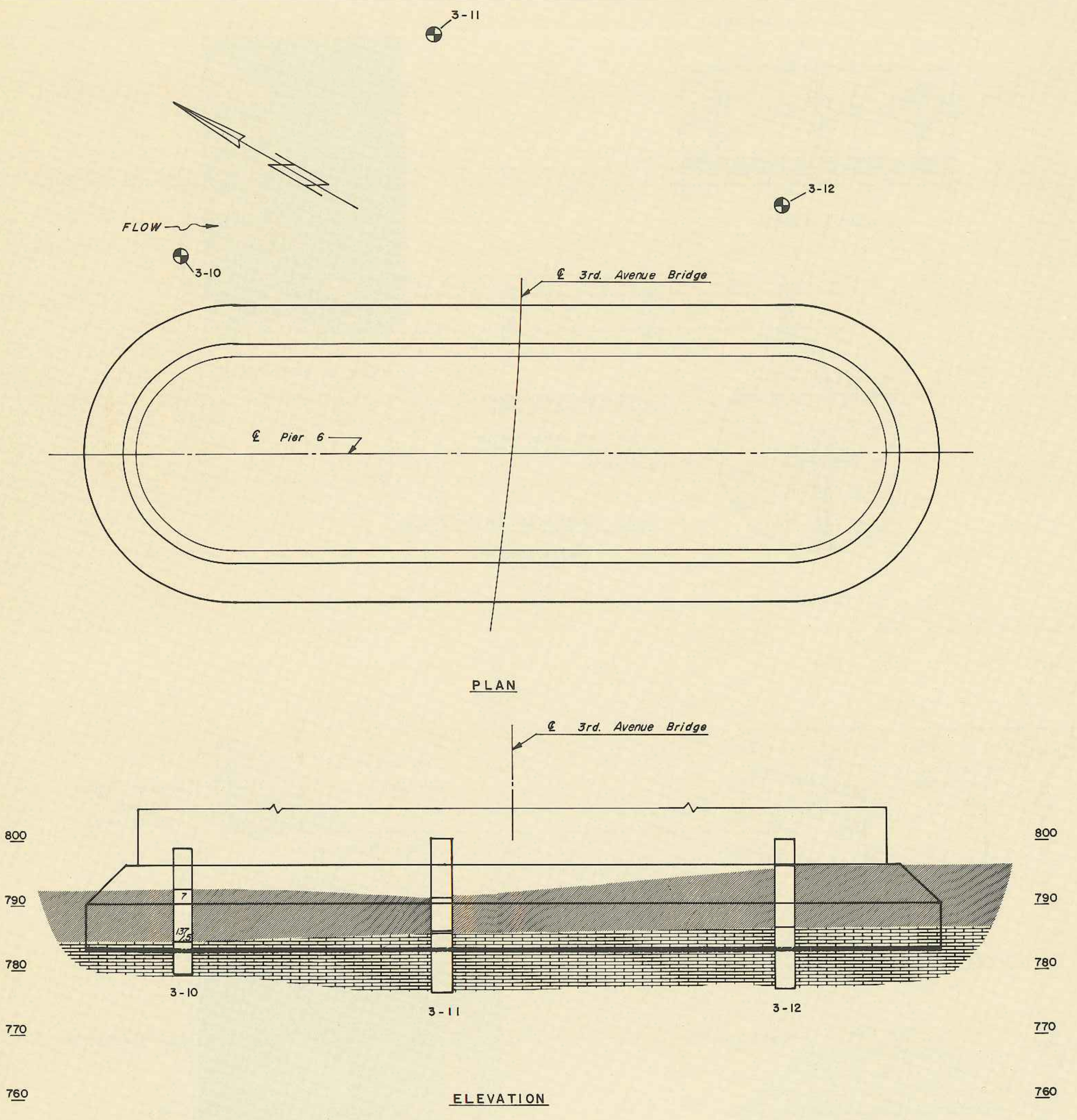
▩	CONCRETE	▤	PLATTEVILLE FORMATION
⁘	MISC. FILL MAT'L.	▩	GLENWOOD BEDS
▨	RIVER ALLUVIUM	░	ST. PETER SANDSTONE
⁙	GLACIAL DRIFT	▥	LIMESTONE FLOAT



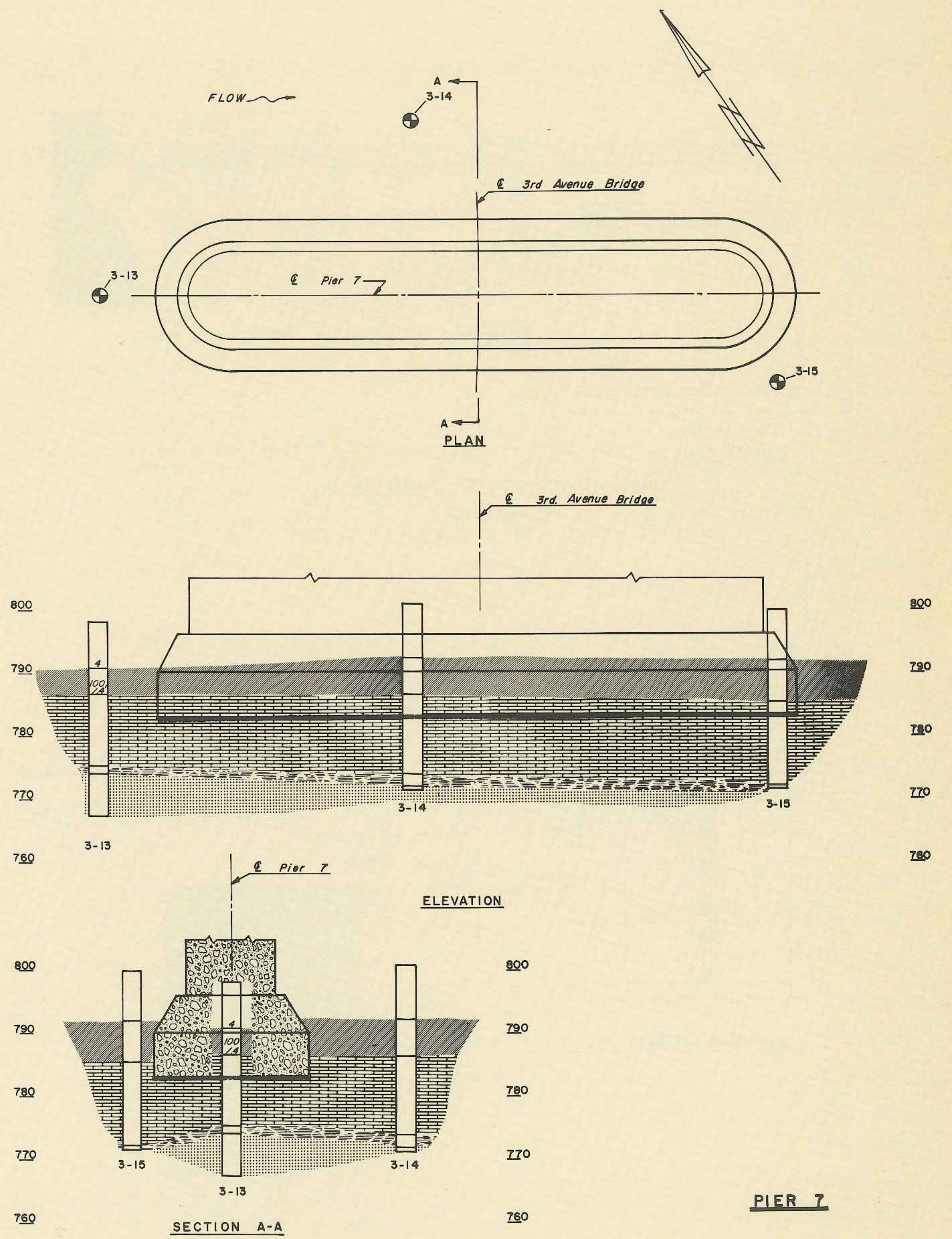
SCALE IN FEET

GEOLOGICAL PROFILES OF PIERS 5, 6, AND 7

EXHIBIT 6



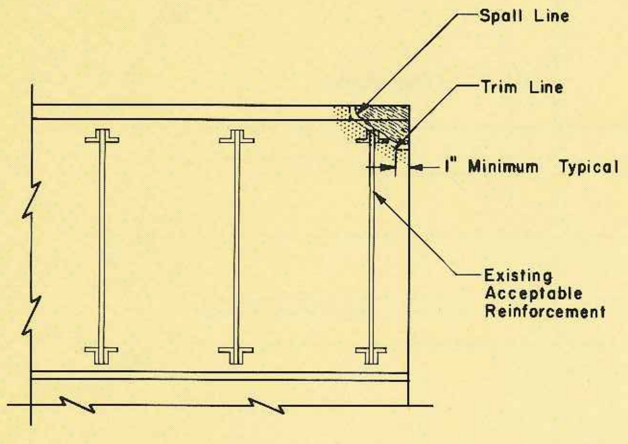
PIER 6



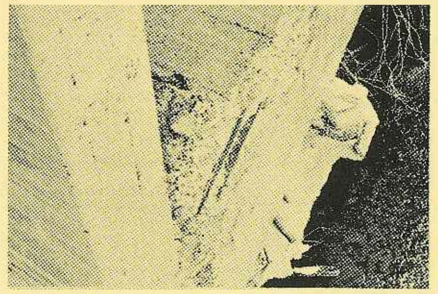
PIER 7

EXHIBIT 7

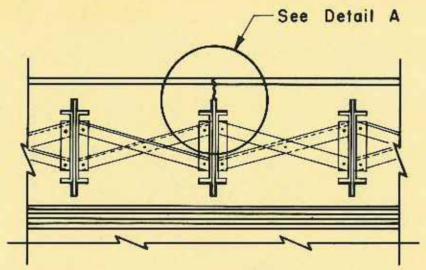
REPAIR PROCEDURES



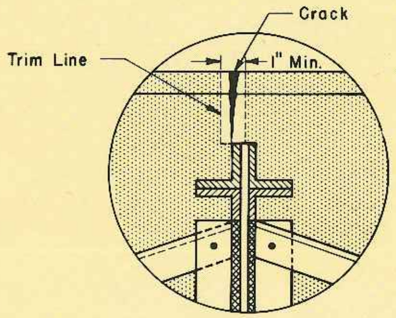
CORNER SPALL



SPALL AT BARREL ARCH



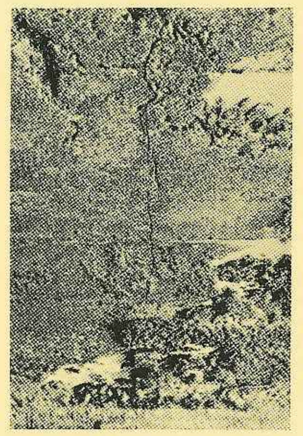
SURFACE CRACK



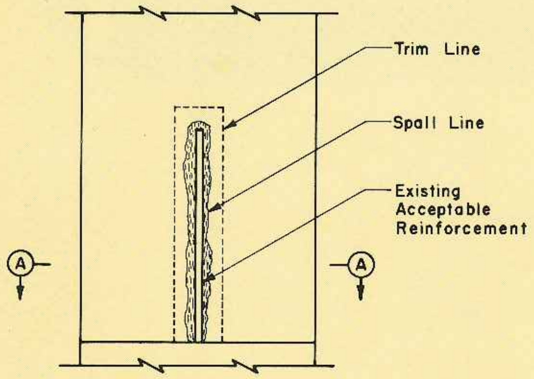
DETAIL A

If crack is narrow and concrete is sound, fill crack with epoxy.

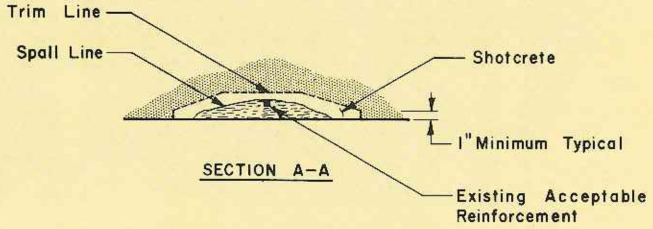
If crack is wide and or concrete is soft, chip out and fill with epoxy mortar.



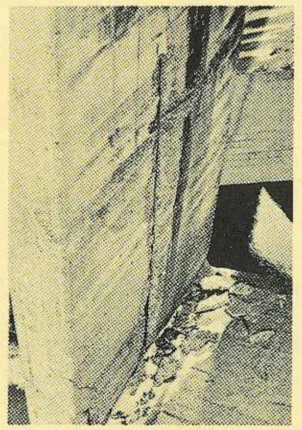
CRACK IN BARREL ARCH



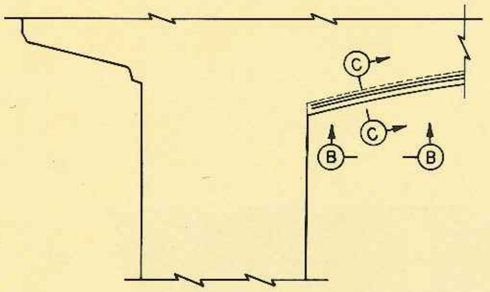
SURFACE SPALL



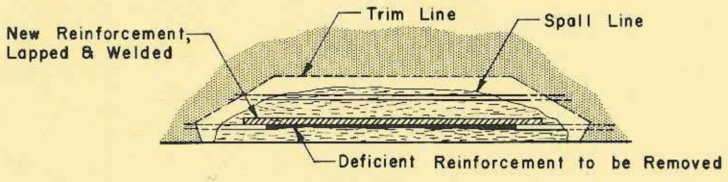
SECTION A–A



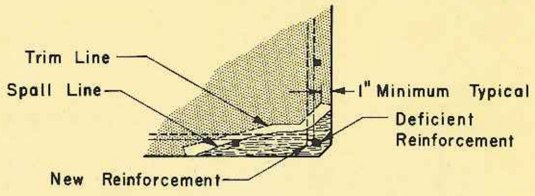
SPALL IN SPANDREL COLUMN



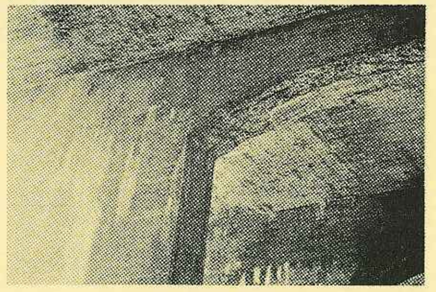
SPALL
(DEFICIENT REINFORCEMENT)



SECTION B–B



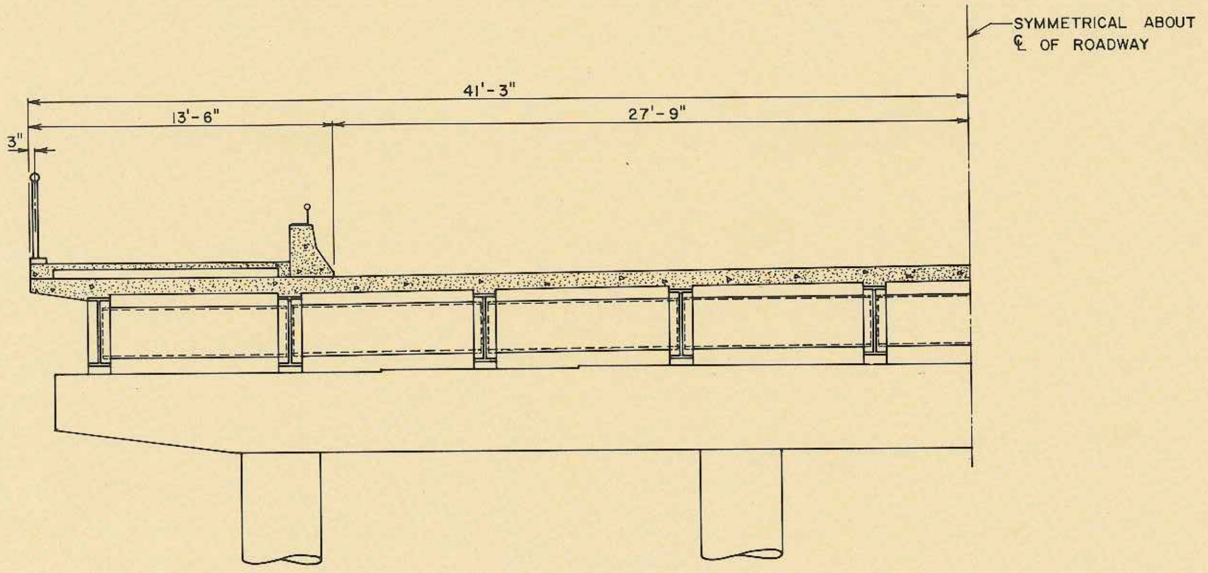
SECTION C–C



DEFICIENT REINFORCEMENT
IN BEAM OF SPANDREL

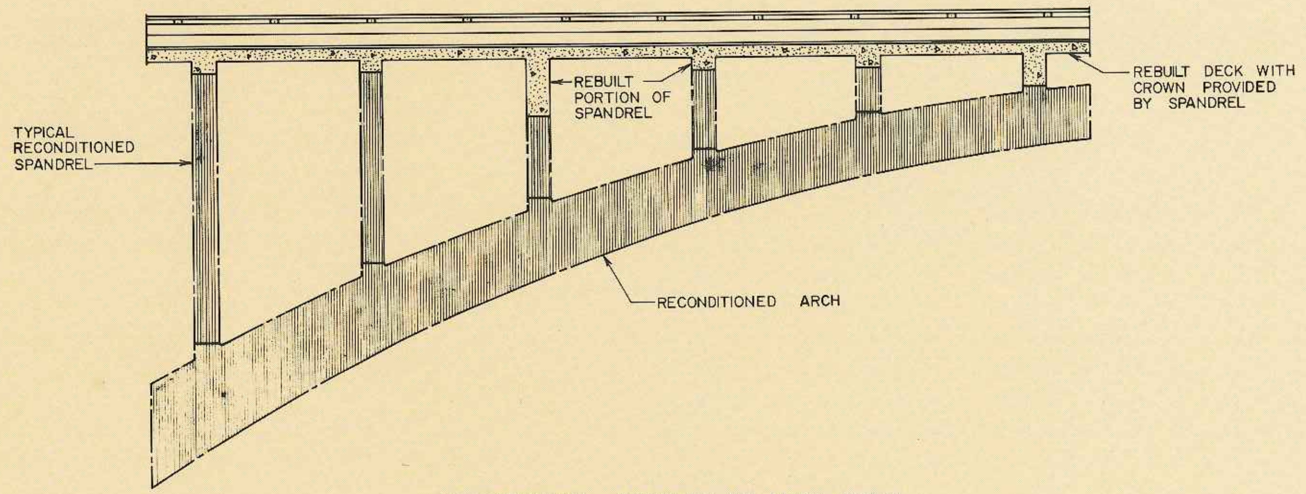
PROPOSED RECONSTRUCTION

EXHIBIT 8



SECTION AT APPROACH SPAN

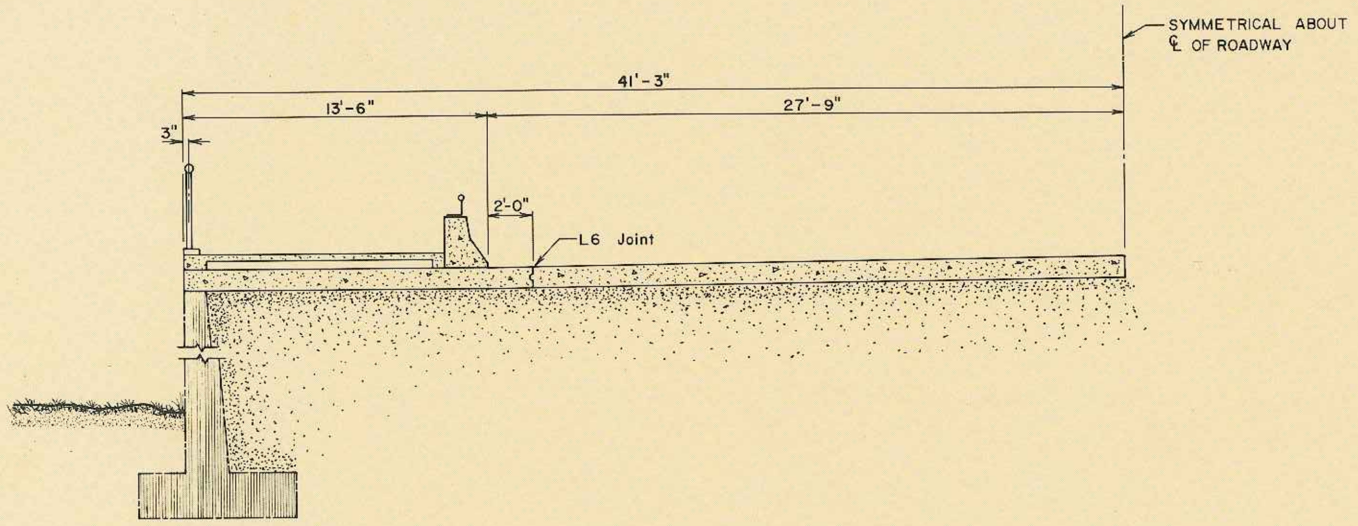
0 5
SCALE IN FEET



LONGITUDINAL SECTION AT ARCH SPAN

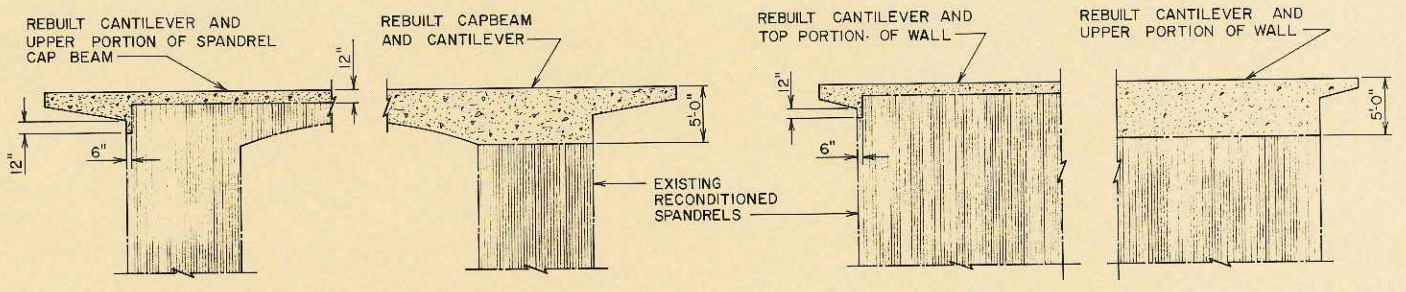
0 5
SCALE IN FEET

LINES SHOWN THUS ———— INDICATE
REUSED COMPONENTS OF EXISTING STRUCTURE



SECTION AT RETAINING WALL

0 5
SCALE IN FEET



SPANDREL COLUMN ABOVE RIBBED ARCH

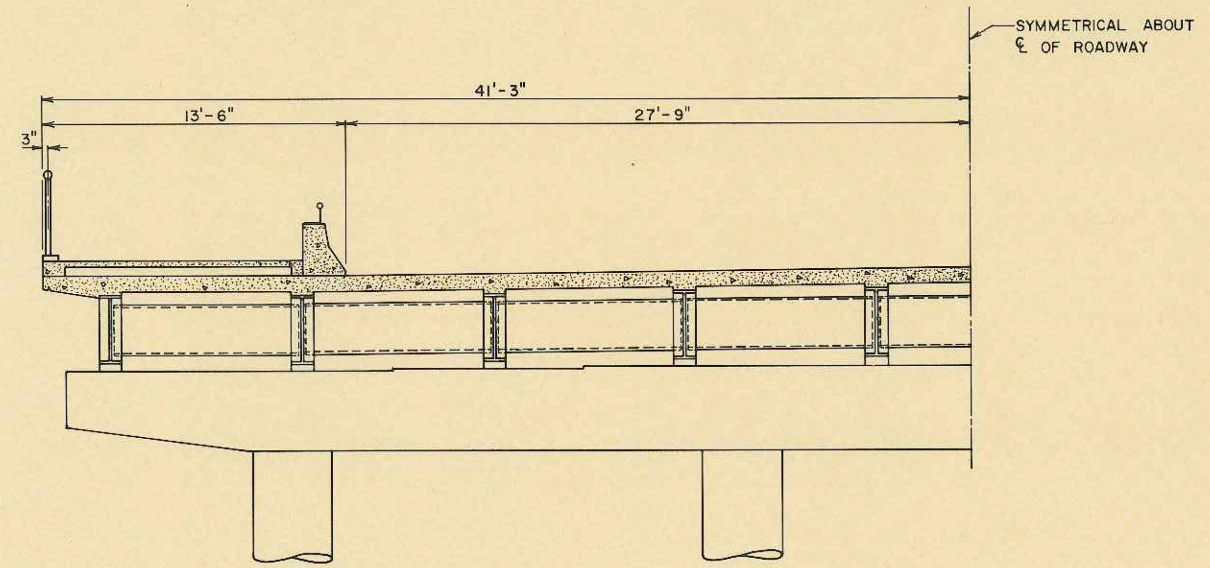
SPANDREL WALL ABOVE BARREL ARCH

PROPOSED TYPES OF RECONSTRUCTION OF SPANDREL COLUMNS AND WALLS

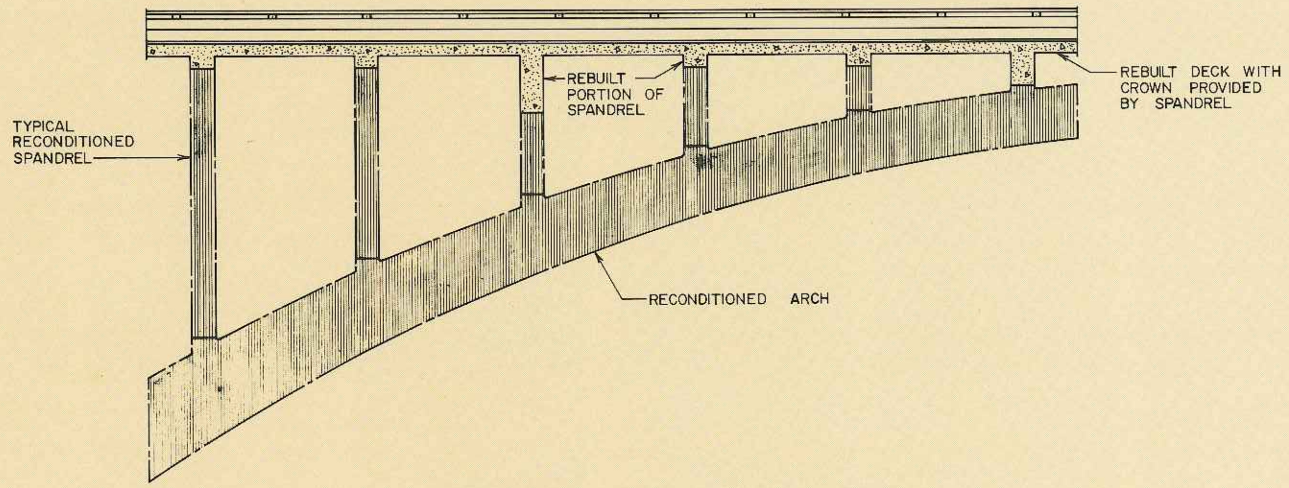
0 5
SCALE IN FEET

PROPOSED RECONSTRUCTION

EXHIBIT 8

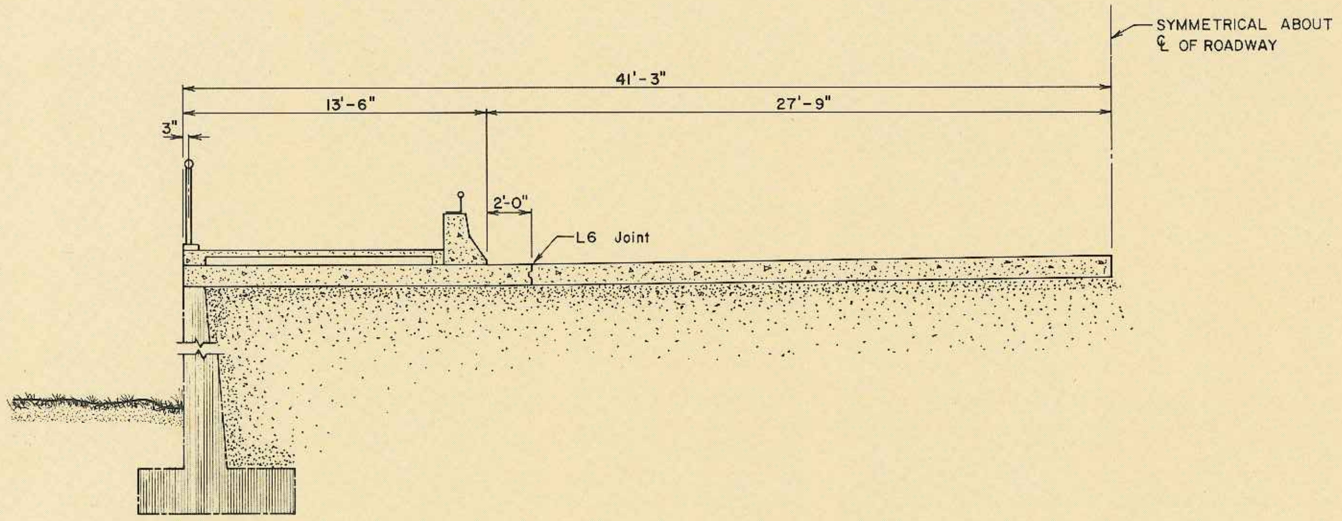


SECTION AT APPROACH SPAN

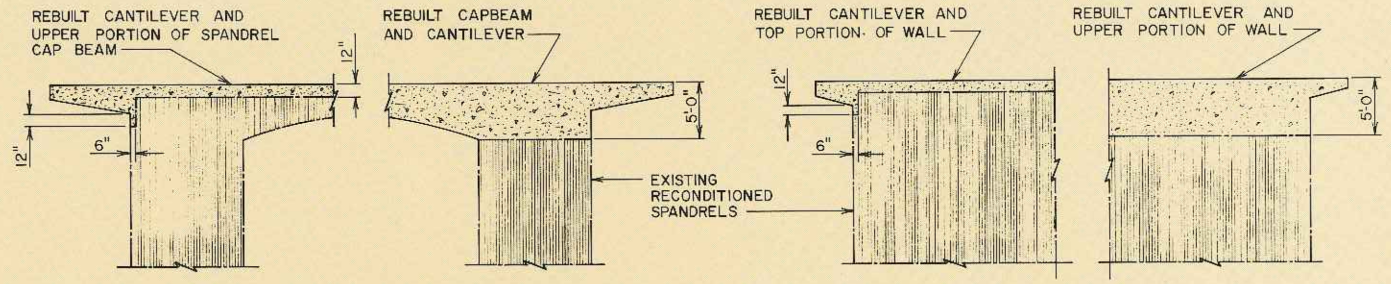


LONGITUDINAL SECTION AT ARCH SPAN

LINES SHOWN THUS ———— INDICATE
REUSED COMPONENTS OF EXISTING STRUCTURE



SECTION AT RETAINING WALL



SPANDREL COLUMN ABOVE RIBBED ARCH

SPANDREL WALL ABOVE BARREL ARCH

PROPOSED TYPES OF RECONSTRUCTION OF SPANDREL COLUMNS AND WALLS

APPENDIX B

LOG OF BORINGS

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: R. Kwilinski
Inspector: D. Wadley

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge, Minneapolis, Minn.
BORING NO: ST-3-1
SHEET 1 **OF** 2

DATE OF BORING
STARTED: 7/22/68
COMPLETED: 7/23/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
___ **HOURS AFTER DRILLING:** *
___ **HOURS AFTER DRILLING:** ___
___ **HOURS AFTER DRILLING:** ___

LOCATION
STATION: ___
OFFSET: ___
South Abutment

*boring on sidewalk -- backfilled immediately

SAMPLER TYPE AND DATA
☒ **SPLIT BARREL** 1 3/8" I.D., 2" O.D.
U **UNDISTURBED SAMPLE** ___
C **ROCK CORE** AX - Diamond Bit
A **AUGER** 2¼" ID, 6" O.D. Hollow-stem
☐ **OTHER** ___

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE. X = Blows 1st 6", Y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER
5	☒	34A	6/7	8"			
10	☒	35A	2/2	13"			
15	☒	36A	8/8	18"			
20	☒	37A	12/9	15"			
25	☒	38A	39/15	18"			
30	☒	39A	114 in 0.5 100	14"			
	☒						
35	C			18" = 30%			
40	C			11" = 18%			
	C		**	23" = 139%			

SOIL DESCRIPTION AND REMARKS
CLASSIFICATION SYSTEM U.S. Bureau of Soils
ELEV. 842.9

SOIL DESCRIPTION AND REMARKS	Depth	ELEV.
(3" Concrete Sidewalk) Fill, black to brown, loose, Fine to Medium Sand, with Fine to Medium Gravel, moist	19	823.9
Brown, stiff to rather stiff, Fine Loamy Sand, with some Fine Gravel, moist	24	818.9
Grey, very stiff, shale or Clay, with some igneous Gravel, dry to slightly moist *	31	811.9
Rock Fill, Grey to buff, Platteville Formation Limestone, with lenses of weathered Limestone	36	806.9
Rock Fill, Buff to grey, Platteville Formation, weathered Limestone	41	801.9
Rock Fill, Grey to buff, Platteville Formation Limestone, top***	42.5	800.4

*Refusal to hollow-stem auger at 31.0.

**Apparently recovered some of core lost from second run.

***very weathered

Note: Boring amended in Addendum of October 4, 1968, to show "Rock Fill."

FORM NO. 173-363-3

HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: R. Kwilinski
P. Anderson
Inspector: D. Wadley

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge, Minneapolis, Minn.
BORING NO: ST-3-3
SHEET 1 **OF** 1

DATE OF BORING
STARTED: 7/18/68
COMPLETED: 7/19/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
1 **HOURS AFTER DRILLING:** 10.5
___ **HOURS AFTER DRILLING:** ___
___ **HOURS AFTER DRILLING:** ___

LOCATION
STATION: ___
OFFSET: ___
Pier B-3

SAMPLER TYPE AND DATA
☒ **SPLIT BARREL** 1 3/8" I.D., 2" O.D.
U **UNDISTURBED SAMPLE** ___
C **ROCK CORE** AX-Tungsten Carbide & Diamond Bits
A **AUGER** 2¼" I.D. 6" O.D. Hollow-stem
☐ **OTHER** ___

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE. X = Blows 1st 6", Y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER
5	☒	30A	10/9	8"		Jetting not used during drilling.	2¼" I.D., 6" O.D. Hollow-stem auger
10	☒	31A	10/5	2"			
15	☒	32A	5/2	4"			
20	☒	33A	5/100 in 0.1'	10"			
25	C			59½" = 99%			

SOIL DESCRIPTION AND REMARKS
CLASSIFICATION SYSTEM U.S. Bureau of Soils
ELEV. 808.2

SOIL DESCRIPTION AND REMARKS	Depth	ELEV.
Fill, black to brown, loose to very dense, Fine to Medium Sand, with Medium to Coarse Gravel, and some limestone fragments, moist to wet	22.5	785.7
Grey with lenses of dark grey, Platteville Formation Limestone, upper 20" badly fractured and hard to core	27.5	780.7

FORM NO. 173-363-3

HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: P. H. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota
BORING NO: ST-3-4
SHEET 1 **OF** 2

DATE OF BORING
STARTED: 4/11/68
COMPLETED: 4/16/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
___ **HOURS AFTER DRILLING:** ___
___ **HOURS AFTER DRILLING:** ___
___ **HOURS AFTER DRILLING:** ___

LOCATION
STATION: ___
OFFSET: ___
Pier 5

SAMPLER TYPE AND DATA
☒ **SPLIT BARREL** 1 3/8" I.D., 2" O.D.
U **UNDISTURBED SAMPLE** ___
C **ROCK CORE** NX Diamond Bit
A **AUGER** ___
☐ **OTHER** ___

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE. X = Blows 1st 6", Y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER
2.5						Water used as drilling fluid	NX Casing initially seated to 6.0' (1st core run begun inside casing), lost water at 15.5' when wood was then noted in split barrell sampler, reamed hole with 3 3/4" drag bit to 16' advanced casing to 16.3' to continue coring
5.0	C			60" = 100%			
7.5							
10.0	C			60" = 100%			
12.5							
15.0	C			18½"			
	☒	6					
17.5	C			58½" = 97%			
20.0							
22.5	C			32" = 100%			
25.0		26"		27" = 98%			
27.5	C			26" = 94%			
	☒	7	100 in 0.4'				
32.5	☒		100 in 0.3'	0"			
35.0	☒		100 in 0.2'	0"			

SOIL DESCRIPTION AND REMARKS
CLASSIFICATION SYSTEM ___
ELEV. 799.8

SOIL DESCRIPTION AND REMARKS	Depth	ELEV.
Water	3'	796.8
Concrete Footing	15.5'	784.3
Decayed Wood	16'	783.8
Light grey mottled with dark grey Platteville Formation. (lost water at 23.5' -- two short core runs taken)	24.7'	775.1
Bluish grey to yellowish brown Glenwood Formation	27'	772.8
*Classification below 27' on	28.7	771.1
Very dense, greenish grey St. Peter Sandstone (upper portions well cemented -- retained in core barrell)	36.7	763.1

FORM NO. 173-363-3

HOWARD, NEEDLES, TAMMEN & BERGENDOFF

HOWARD, NEEDLES, TAMMEN & BERGENDOFF

LOG OF BORINGS

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: P. H. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota
BORING NO: ST-3-5
SHEET 1 OF 2

DATE OF BORING
STARTED: 4/16/68
COMPLETED: 4/19/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____

LOCATION
STATION: ____
OFFSET: ____
Pier 5

SAMPLER TYPE AND DATA
- ☒ SPLIT BARREL 1 3/8" I.D., 2" O.D.
- U UNDISTURBED SAMPLE ____
- C ROCK CORE NX Diamond bit
- A AUGER ____
- ☐ OTHER ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE.	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER	SOIL DESCRIPTION AND REMARKS CLASSIFICATION SYSTEM ____	ELEV. 800.0
						Water used as drilling fluid	Drilled pilot hole to 3.5' with drag bit to seat casing, NX casing advanced to 5.4' -- 1st NX core run begun in casing.	Water	
2.5								3'	797.0
								Concrete Footing (lower portions badly fractured and easier drilling -- possibly leaner concrete)	
5.0	C			60" = 100%					
7.5									
10.0	C			57½" = 96%					
12.5									
15.0	C			47" = 78%					
17.5								17.5'	782.5
								Light grey mottled with dark grey Platteville Formation	
20.0	C			60" = 100%					
22.5									
25.0	C			57" = 95%				26.5'	773.5
27.5								Bluish grey to yellowish brown Glenwood Formation 28'	772.0
								Very dense, brownish grey St. Peter Sandstone	
30.0	☒	8	100 in 0.5'						
32.5									
35.0	☒	9W	100 in 0.3'	0'				(W indicates sample obtained from wash water)	
37.5									
40.0	☒	10W	100 in 0.3'	0"				40.3'	759.7

FORM NO. 173-363-3 HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: P. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota
BORING NO: ST-3-6
SHEET 1 OF 1

DATE OF BORING
STARTED: 4/11/68
COMPLETED: 4/11/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____

LOCATION
STATION: ____
OFFSET: ____
Pier 5

SAMPLER TYPE AND DATA
- ☒ SPLIT BARREL 1 3/8" I.D., 2" O.D.
- U UNDISTURBED SAMPLE ____
- C ROCK CORE NX Diamond Bit
- A AUGER ____
- ☐ OTHER ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE.	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER	SOIL DESCRIPTION AND REMARKS CLASSIFICATION SYSTEM U.S. Bureau of Soils	ELEV. 799.6
						Water used as drilling fluid	NX casing advanced by rotation	Water	
2.5									
5.0									
7.5								*Loose, brown, Medium to Coarse Sand and Fine Gravel, wet	
10.0									
12.5								14.3	785.3
15.0	☒	5	2 100in	0.2'				* 15'	784.6
								Light grey mottled with dark grey Platteville Formation	
17.5	C			62" = 100%					
20.0								20.4	779.2

FORM NO. 173-363-3 HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: P.H. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota
BORING NO: ST-3-7
SHEET 1 OF 1

DATE OF BORING
STARTED: 4/30/68
COMPLETED: 5/1/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____

LOCATION
STATION: ____
OFFSET: Pier 5

SAMPLER TYPE AND DATA
- ☒ SPLIT BARREL ____
- U UNDISTURBED SAMPLE ____
- C ROCK CORE NX Diamond Bit
- A AUGER ____
- ☐ OTHER ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE.	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER	SOIL DESCRIPTION AND REMARKS CLASSIFICATION SYSTEM ____	ELEV. 800.1
						Water used as drilling fluid	NX casing seated by rotation to 17.0' -- first core run begun in casing	Water	
2.5									
5.0									
7.5									
10.0									
12.5								14.0'	786.1
15.0'								Sand -- not sampled	
								16.5'	783.6
17.5	C			60" = 100%				Light grey mottled with dark grey Platteville Limestone Formation (upper 1' fragmented)	
20.0									
22.5	C			53" = 92%					
25.0								25.3'	774.8
								Bluish grey Glenwood Formation 26.5'	773.6

FORM NO. 173-363-3 HOWARD, NEEDLES, TAMMEN & BERGENDOFF

THIRD AVENUE BRIDGE

LOG OF BORINGS

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: P.H. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota
BORING NO: ST-3-8
SHEET 1 OF 1

DATE OF BORING
STARTED: 5/1/68
COMPLETED: 5/1/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____

LOCATION
STATION: ____
OFFSET: ____
Pier 5

SAMPLER TYPE AND DATA
☒ SPLIT BARREL ____
U UNDISTURBED SAMPLE ____
C ROCK CORE NX Diamond Bit
A AUGER ____
☐ OTHER ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE, X = Blows 1st 6", Y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER
2.5					
5.0					
7.5					
10.0					
12.5					
15.0					
17.5				60" = 100%	
20.0	C				
22.5				60" = 100%	
25.0	C				

DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID: Water used as drilling fluid

CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER: NX Casing seated by rotation to 17.0' depth -- first core run begun in casing.

SOIL DESCRIPTION AND REMARKS
CLASSIFICATION SYSTEM ____

SOIL DESCRIPTION AND REMARKS		ELEV.
		800.0
Water	8.5'	791.5
Sand -- not sampled	16.6'	783.4
Light grey mottled with dark grey Platteville Formation	25.6'	774.4
*	26.6'	773.4
*Bluish grey Glenwood Formation		

FORM NO. 173-363-3
HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: P.H. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota
BORING NO: ST-3-9
SHEET 1 OF 2

DATE OF BORING
STARTED: 4/19/68
COMPLETED: 4/22/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____

LOCATION
STATION: ____
OFFSET: ____
Pier 5

SAMPLER TYPE AND DATA
☒ SPLIT BARREL 1 3/8" I.D. 2" O.D.
U UNDISTURBED SAMPLE ____
C ROCK CORE NX Diamond Bit
A AUGER ____
☐ OTHER ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE, X = Blows 1st 6", Y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER
2.5					
5.0					
7.5					
10.0					
12.5					
15.0					
17.5				58" = 97%	
20.0	C				
22.5				59½" = 100%	
25.0	C				
27.5	C			14" = 49%	
30.0	☒	10	115 in 0.3'		
32.5					
35.5	☒		110 in 0.3'	0"	

DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID: Water used as drilling fluid

CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER: NX casing seated by rotation to 18.5' -- first core run began in casing

SOIL DESCRIPTION AND REMARKS
CLASSIFICATION SYSTEM ____

SOIL DESCRIPTION AND REMARKS		ELEV.
		799.6
Water	7'	792.6
Sand - not sampled	16.5'	783.1
Light grey mottled with dark grey Platteville Formation (upper 1-foot fragmented)	25.6'	774.0
Bluish grey to yellowish brown Glenwood Formation	*27.7±	771.9±
Very dense, light grey mottled with brown, St. Peter Sandstone *burned up NX Diamond Bit on last run due to soft shale plugging bit	36.3'	763.6

FORM NO. 173-363-3
HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged by: P. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota
BORING NO: ST-3-10
SHEET 1 OF 1

DATE OF BORING
STARTED: ____
COMPLETED: ____

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____
____ HOURS AFTER DRILLING: ____

LOCATION
STATION: ____
OFFSET: ____
Pier 6

SAMPLER TYPE AND DATA
☒ SPLIT BARREL 1 3/8" I.D., 2" O.D.
U UNDISTURBED SAMPLE ____
C ROCK CORE NX Diamond Bit
A AUGER ____
☐ OTHER ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE, X = Blows 1st 6", Y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER
2.5					
5.0					
7.5	☒		4/4/3	0"	
10.0					
12.5					
15.0	☒	4	137 in 6"		
17.5	C			60" = 100%	

DRILLING-DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID: Water used as drilling fluid

CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER: NX casing advanced by rotation

SOIL DESCRIPTION AND REMARKS
CLASSIFICATION SYSTEM U. S. Bureau of Soils

SOIL DESCRIPTION AND REMARKS		ELEV.
		798.9
Water	6.5'	792.4
Loose, brown, Medium to Coarse Sand and Fine Gravel, with some small boulders and a few pieces of wood	14.5'	784.4
Light grey mottled with dark grey Platteville Formation	19.5'	779.4

FORM NO. 173-363-3
HOWARD, NEEDLES, TAMMEN & BERGENDOFF

HOWARD, NEEDLES, TAMMEN & BERGENDOFF

LOG OF BORINGS

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota

Logged By: P.H. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota

BORING NO: ST-3-11
SHEET 1 **OF** 1

DATE OF BORING
STARTED: 4/29/68
COMPLETED: 4/30/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
____ **HOURS AFTER DRILLING:** ____
____ **HOURS AFTER DRILLING:** ____
____ **HOURS AFTER DRILLING:** ____

LOCATION
STATION: ____
OFFSET: ____
Pier 6

SAMPLER TYPE AND DATA
☒ SPLIT BARREL ____
U UNDISTURBED SAMPLE ____
C ROCK CORE NX Diamond Bit
A AUGER ____
☐ OTHER ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE. x = Blows 1st 6", y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER	SOIL DESCRIPTION AND REMARKS CLASSIFICATION SYSTEM ____	ELEV. 800.2
						Water used as drilling fluid	NX casing seated by rotation to 16.5' depth -- first core run begun in casing	Water	
2.5									
5.0									
7.5								9.3'	790.9
10.0								Sand -- not sampled	
12.5								14.5'	785.7
								Wood 14.8'	785.4
15.0				54" = 100%				Light grey mottled with dark grey Platteville Formation	
17.5	C								
20.0				60" = 100%					
22.5	C							24'	776.2

FORM NO. 173-363-3 HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota

Logged By: P.H. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota

BORING NO: ST-3-12
SHEET 1 **OF** 1

DATE OF BORING
STARTED: 4/24/68
COMPLETED: 4/25/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
____ **HOURS AFTER DRILLING:** ____
____ **HOURS AFTER DRILLING:** ____
____ **HOURS AFTER DRILLING:** ____

LOCATION
STATION: ____
OFFSET: ____
Pier 6

SAMPLER TYPE AND DATA
☒ SPLIT BARREL ____
U UNDISTURBED SAMPLE ____
C ROCK CORE NX Diamond Bit
A AUGER ____
☐ OTHER ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE. x = Blows 1st 6", y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER	SOIL DESCRIPTION AND REMARKS CLASSIFICATION SYSTEM ____	ELEV. 800.1
						Water used as drilling fluid	NX casing seated by rotation to 16' -- first core run begun inside casing	Water	
								3'	797.1
2.5								Sand -- not sampled	
5.0									
7.5									
10.0									
12.5								13.5'	786.6
								Light grey mottled with dark grey Platteville Formation (badly fragmented in upper 24")	
15.0	C			58" = 97%					
17.5									
20.0	C			42" = 87%					
22.5								23'	777.1

FORM NO. 173-363-3 HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Borings By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota

Logged By: P. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota

BORING NO: ST-3-13
SHEET 1 **OF** 1

DATE OF BORING
STARTED: 4/8/68
COMPLETED: 4/8/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
____ **HOURS AFTER DRILLING:** ____
____ **HOURS AFTER DRILLING:** ____
____ **HOURS AFTER DRILLING:** ____

LOCATION
STATION: ____
OFFSET: ____
Pier 7

SAMPLER TYPE AND DATA
☒ SPLIT BARREL 1 3/8" I.D., 2" O.D.
U UNDISTURBED SAMPLE ____
C ROCK CORE NX diamond bit
A AUGER ____
☐ OTHER ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE. x = Blows 1st 6", y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER	SOIL DESCRIPTION AND REMARKS CLASSIFICATION SYSTEM U.S. Bureau of Soils	ELEV. 798.2
						Water used as drilling fluid	NX casing advanced by rotation to 13.0'	Water	
2.5									
5.0								7.3'	790.9
7.5	☒	1	1/2/2					Loose, dark grey and black Medium Sand, organic, with pieces of wood, wet	
10.0	☒	2	2/100 in 0.2'					11.5'	786.7
12.5								Light grey mottled with dark grey Platteville Formation	
15.0	C			63" = 100%					
17.5									
20.0	C			57" = 100%					
22.5								22.8'	775.4
				12" = 20%				* 24'	774.2
25.0	C							Very dense, light brown to brown St. Peter Sandstone	
27.5									
30.0	☒	3	100 in 0.4'					30.4'	767.8
								*Bluish grey Glenwood Formation	

FORM NO. 173-363-3 HOWARD, NEEDLES, TAMMEN & BERGENDOFF

LOG OF BORINGS

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: P.H. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota
BORING NO: ST-3-14
SHEET 1 **OF** 1

DATE OF BORING
STARTED: 4/26/68
COMPLETED: 4/29/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
____ **HOURS AFTER DRILLING:** ____
____ **HOURS AFTER DRILLING:** ____
____ **HOURS AFTER DRILLING:** ____

LOCATION
STATION: ____
OFFSET: ____
Pier 7

SAMPLER TYPE AND DATA
☒ **SPLIT BARREL** ____
U **UNDISTURBED SAMPLE** ____
C **ROCK CORE** NX Diamond Bit
A **AUGER** ____
☐ **OTHER** ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE	RECOVERY, in.	SOIL DESCRIPTION AND REMARKS CLASSIFICATION SYSTEM ____	ELEV.
						801.2
2.5					Water	
5.0						
7.5					8.5'	792.7
10.0					Sand -- not sampled	
12.5					14.5'	786.7
15.0				60" = 100%	Light grey mottled with dark grey Platteville Formation (upper 2' fragmented)	
17.5	C					
20.0				58½" = 98%		
22.5	C					
25.0				58" = 97%	27.2'	774.0
27.5	C				Bluish grey to yellowish brown Glenwood Formation 29.0'	772.1
		w			* 29.5'	771.7
					*Bluish grey to brownish grey St. Peter Sandstone	

DRILLING DATA: Water used as drilling fluid

CASING SIZE: NX casing seated by rotation to 16.5' depth -- first core run begun in casing

FORM NO. 173-363-3 w = washed sample HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: P.H. Anderson
Inspector: J. Murphy

LOG OF BORING

PROJECT: 68-80 3rd Avenue Bridge
Minneapolis, Minnesota
BORING NO: ST-3-15
SHEET 1 **OF** 1

DATE OF BORING
STARTED: 4/25/68
COMPLETED: 4/26/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
____ **HOURS AFTER DRILLING:** ____
____ **HOURS AFTER DRILLING:** ____
____ **HOURS AFTER DRILLING:** ____

LOCATION
STATION: ____
OFFSET: ____
Pier 7

SAMPLER TYPE AND DATA
☒ **SPLIT BARREL** ____
U **UNDISTURBED SAMPLE** ____
C **ROCK CORE** NX Diamond Bit
A **AUGER** ____
☐ **OTHER** ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE	RECOVERY, in.	SOIL DESCRIPTION AND REMARKS CLASSIFICATION SYSTEM ____	ELEV.
						800.0
2.5					Water	
5.0						
7.5					8'	792.0
10.0					Sand -- not sampled	
12.5					14.5'	785.5
15.0				60" = 100%	Light grey mottled with dark grey Platteville Formation (upper 26" fragmented)	
17.5	C					
20.0				48" = 80%*		
22.5	C					
25.0				55½" = 125%*		
27.5	C				** 27.9'	772.1
					28.2'	771.8
					**Bluish grey Glenwood Formation	

*12" of 2nd run dropped in withdrawing barrel and recovered in 3rd run

FORM NO. 173-363-3 HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: R. Kwilinski
Inspector: D. Wadley

LOG OF BORING

PROJECT: 68-80
3rd Avenue Bridge, Minneapolis, Minn.
BORING NO: ST-3-16
SHEET 1 **OF** 2

DATE OF BORING
STARTED: 7/9/68
COMPLETED: 7/10/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
1/4 **HOURS AFTER DRILLING:** 17.0
____ **HOURS AFTER DRILLING:** ____
____ **HOURS AFTER DRILLING:** ____

LOCATION
STATION: ____
OFFSET: ____
Pier B-4

SAMPLER TYPE AND DATA
☒ **SPLIT BARREL** 1 3/8" I.D., 2" O.D.
U **UNDISTURBED SAMPLE** ____
C **ROCK CORE** AX-Tungsten Carbide Bit
A **AUGER** 2¼" I.D., 6" O.D. Hollow-stem
☐ **OTHER** ____

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE	RECOVERY, in.	SOIL DESCRIPTION AND REMARKS CLASSIFICATION SYSTEM U.S. Bureau of Soils	ELEV.
						817.8
					Fill, loose to medium, black to dark brown, Fine to Medium Sand, with some Fine Gravel and glass and wood, lenses of Sandy Loam, Sandy Clay Loam, and with some Limestone fragments, moist	
5	☒	11A	5 3	7"		
10	☒	12A	2 2	7"		
15	☒	13A	3 5	5"		
20	☒	14A	9 11	5"	24	793.8
25	☒	15A	100 in 0.3	5"	Very dense, grey, with a little yellow, limestone * 26.5	791.3
	C			35" = 73%	Buff and grey Platteville Formation limestone, gragmented in lower portions 30.5	787.3
30						
35	C			63½" = 88%	Grey, Platteville Formation limestone, with thin interbedding of shale, fragmented 36.5	781.3
					*Refusal to auger at 26.5.	

DRILLING DATA: Jetting water not used during sampling and auger boring. Lost a lot of drilling fluid (water) during coring both runs. Runs were smooth.

CASING SIZE: 2¼" I.D., 6" O.D. Hollow-stem augers

FORM NO. 173-363-3 HOWARD, NEEDLES, TAMMEN & BERGENDOFF

HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: R. Kwilinski
Inspector: D. Wadley

LOG OF BORING

PROJECT: 68-80
3rd Avenue Bridge, Minneapolis, Minn.
BORING NO: ST-3-17
SHEET 1 OF 1

DATE OF BORING
STARTED: 7/9/68
COMPLETED: 7/9/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
1/4 HOURS AFTER DRILLING: 14.0
___ HOURS AFTER DRILLING: ___
___ HOURS AFTER DRILLING: ___

LOCATION
STATION: ___
OFFSET: ___
Pier B-5

SAMPLER TYPE AND DATA
☒ SPLIT BARREL 1 3/8" I.D., 2" O.D.
U UNDISTURBED SAMPLE ___
C ROCK CORE AX-Tungsten Carbide Bit
A AUGER 2½" I.D., 6" O.D. Hollow-stem
☐ OTHER ___

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE x = Blows 1st 6", y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER
5	☒	7A	3/10	12"		Jetting water not used during sampling and auger boring. Smooth coring run but lost a lot of drilling fluid (water), 260 fallons.	2¼" I.D. 6" O.D. Hollow-stem auger
10	☒	8A	30/15	14"			
15	☒	9A	6/7	12"			
20	☒	10A	100 in 0.3'	8"			
	C			36" = 60%			
25							

SOIL DESCRIPTION AND REMARKS		ELEV.
CLASSIFICATION SYSTEM U.S. Bureau of Soils		816.25
Fill, black, Fine to Medium Sandy Loam, non plastic,with some Fine Gravel, moist	3	813.2
Loose to Medium, brown, Fine to Medium Sand, with some Fine Gravel, moist (probably fill) (Refusal to auger at 20' depth)	20	796.2
Buff to grey, Platteville Formation, fragmented	25	791.2

FORM NO. 173-363-3
HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: R. Kwilinski
Inspector: D. Wadley

LOG OF BORING

PROJECT: 68-80
3rd Avenue Bridge, Minneapolis, Minn.
BORING NO: ST-3-18
SHEET 1 OF 1

DATE OF BORING
STARTED: 7/5/68
COMPLETED: 7/8/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
1/4 HOURS AFTER DRILLING: NE
___ HOURS AFTER DRILLING: ___
___ HOURS AFTER DRILLING: ___

LOCATION
STATION: ___
OFFSET: ___
Pier B-6

*NE = water not encountered in probing

SAMPLER TYPE AND DATA
☒ SPLIT BARREL 1 3/8" I.D., 2" O.D.
U UNDISTURBED SAMPLE ___
C ROCK CORE AX-Tungsten Carbide Bit
A AUGER 2¼" I.D., 6" O.D. Hollow-stem
☐ OTHER ___

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE x = Blows 1st 6", y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER
5	☒	1A	2/4	11"		Jetting not used in drilling and sampling. Lost a lot of water (200 gal/foot) in coring. Runs were smooth.	2¼" I.D., 6" O.D. Hollow-stem auger
10	☒	2A	15/25	14"			
15		3A	12/100 in 0.5'	11"			
20	C			39½" = 66%			
25	C			52½" = 87%			

SOIL DESCRIPTION AND REMARKS		ELEV.
CLASSIFICATION SYSTEM U.S. Bureau of Soils		816.30
Fill, black, Loamy Sand with some cinders, moist	2	814.3
Loose, dark brown to brown, Loamy Sand to Sand, moist	6	810.3
Medium brown, Fine to Medium Gravel, with some Medium to Coarse Gravel, with some Medium to Coarse Sand, moist	13	803.3
Very dense, brown to grey, weathered Limestone	16	800.3
Grey to bluish-grey, with some buff Platteville Formation Limestone, fragmented in lower 2/3 of sample	23	793.3
Bluish-grey to greyish-blue Platteville Formation Limestone, with some buff in 1st foot, *	26	790.3
*igneous pebbles in bottom foot		

FORM NO. 173-363-3
HOWARD, NEEDLES, TAMMEN & BERGENDOFF

Boring By: SOIL ENGINEERING SERVICES, INC.
Minneapolis, Minnesota
Logged By: R. Kwilinski
Inspector: D. Wadley

LOG OF BORING

PROJECT: 68-80
3rd Avenue Bridge, Minneapolis, Minnesota
BORING NO: ST-3-19
SHEET 1 OF 1

DATE OF BORING
STARTED: 7/8/68
COMPLETED: 7/8/68

BORING TYPE
Rotary
CME 55 Rig

GROUND WATER
1/4 HOURS AFTER DRILLING: NE
___ HOURS AFTER DRILLING: ___
___ HOURS AFTER DRILLING: ___

LOCATION
STATION: ___
OFFSET: ___
North Abutment

NE = water not encountered in probing

SAMPLER TYPE AND DATA
☒ SPLIT BARREL 1 3/8" I.D., 2" O.D.
U UNDISTURBED SAMPLE ___
C ROCK CORE AX-Tungsten Carbide Bit
A AUGER 2¼" I.D., 6" O.D. Hollow-stem
☐ OTHER ___

DEPTH-FEET	SAMPLE & TYPE	SAMPLE NO.	STD. PENETRATION RESISTANCE x = Blows 1st 6", y = Blows 2nd 6"	RECOVERY, in.	COMPRESSIVE STRENGTH TSF, POCKET PENETROMETER	DRILLING DATA: BIT SIZE, TYPE, etc; LOSS OR GAIN OF CIRCULATION; TYPE OF DRILLING FLUID	CASING SIZE, TYPE, etc., BLOWS PER FOOT, WEIGHT OF HAMMER
5	☒	4A	6/4	11"		Jetting not used in drilling or sampling. Little water loss during coring. Smooth run.	2¼" I.D., 6" O.D. hollow-stem auger
10	☒	5A	6/11	8"			
15	☒	6A	16/100 in 0.2'	18"			
	C			58½" = 98%			
20							

SOIL DESCRIPTION AND REMARKS		ELEV.
CLASSIFICATION SYSTEM U.S. Bureau of Soils		816.90
Concrete - 2" Fill, loose, brown to black, Fine to Medium Sand, with Medium Gravel, moist	8	808.9
Fill, stiff, brown and dark brown, plastic Sandy Loam, with some Limestone fragments, moist	12	804.9
Fill, very stiff, brown to dark brown, Sandy Clay Loam, with Fine to Medium Granite fragments, moist *	15.5	801.4
Buff to grey Platteville Formation Limestone, fragmented	20.5	796.4
*Refusal to hollow-stem auger at 15.5.		

FORM NO. 173-363-3
HOWARD, NEEDLES, TAMMEN & BERGENDOFF

C

Appendix C

Minnesota Historic Property Record

MINNESOTA HISTORIC PROPERTY RECORD

PART I. PROPERTY IDENTIFICATION AND GENERAL INFORMATION

Common Name:	Third Avenue Bridge
Bridge Number:	2440
Identification Number:	HE-MPC-0165

Location:

Feature Carried:	TH 65 (Third Avenue S.)
Feature Crossed:	Mississippi River, railroad, and city streets
Descriptive Location:	0.3 Miles Northeast of Jct. TH 952A
Town, Range, Section:	29N-24W-23
Town or City:	Minneapolis
County:	Hennepin

UTM:

Zone:	15
Easting:	4981072
Northing:	479448

Quad:

Minneapolis
7.5 Minute Series
1983

Present Owner:

State

Present Use:

Mainline

Significance Statement:

The Third Avenue Bridge is individually eligible under Criterion C for its engineering significance and under Criterion A as a contributing element to the St. Anthony Falls Industrial Historic District.

The Third Avenue Bridge is an example of Melan arch construction. In 1894, Viennese engineer Josef Melan received an American patent for his innovative reinforcing system. It consisted "of a number of steel I-beams bent approximately to the shape of the arch axis and laid in a parallel series near the undersurface of the arch. The resulting structure might be regarded as a combination of the steel-rib arch and the concrete barrel, the concrete serving a protective as much as a structural purpose" (Frame 1988:3). The first American bridge to embody the Melan system reportedly was a small highway span designed by German-born engineer Fritz von Emperger and built by William S. Hewett at Rock Rapids, Iowa, the same year as the patent. Several small but early Melan bridges were built and designed by Hewett in Minneapolis and Saint Paul for the Twin Cities Rapid Transit and survive today as park structures (Frame 1988:3). The

Third Avenue Bridge is significant because it reflects the design and engineering of Josef Melan's reinforcing system.

In 1912, Minneapolis planners solicited designs for a concrete-arch bridge from a New York-based company, the Concrete-Steel Engineering Co. The Third Avenue Bridge was to be constructed just above the St. Anthony Falls, originally planned to be to the north of the final location. The proposal, which called for sinking piers into the weak stratum that had caused the collapse of the Eastman Tunnel in the 1860s, was not well received by the public or the power companies (since a collapse of the falls would impact its power capabilities).

Frederick W. Capellen, Minneapolis city engineer, devised a solution by altering the bridge location and leapfrogging the bridge arches over the dangerous limestone breaks (Westbrook 1983:18). As described by A. M. Richter in an Engineering News article from 1915 (pp. 1269-1270):

"While bridge engineer for the city in previous years, Capellen had built six bridges across the Mississippi River and acquired a thorough knowledge of river conditions. He refused to approve the proposed location. The City Council then rejected the plans and instructed him to design a steel bridge that could be constructed without endangering the falls or affecting water-power-rights.

"His proposed location is shown on the plan, and his design included one span of 434 feet to clear entirely the area of the limestone breaks. The trusses were to be of the parabolic through-truss type. In the face of many objections (based mainly on aesthetic considerations), the City Council approved the plans and directed the engineer to proceed with construction."

At this time, however, Mr. Cappelen conceived the idea that by adopting a curved location for the line of the bridge, a design satisfactory to all parties might be worked out. On investigation it was found that at one point the limestone break could be spanned by a concrete arch of 211-foot clear-span. A revised plan for the desired ornamental structure was then presented. This proved satisfactory to all parties and was finally adopted."

Construction began on the Third Avenue Bridge in 1914, and the total project cost was $862,254.00.

PART II. HISTORICAL INFORMATION

Date of Construction:

1917

Contractor and/or Designer (if known):

Contractor: Unknown
Designer: Frederick W. Capellen

Historic Context:

Reinforced-Concrete Highway Bridges in Minnesota, 1900-1945

National Register Criterion:

A, C

PART III. DESCRIPTIVE INFORMATION

Descriptive Information:

The Third Avenue Bridge is the last major reinforced-concrete bridge constructed in the Twin Cities using Melan ribs (Westbrook 1983:18). As explained by Condit (1982:174-175):

"In the Melan system, the reinforcing consisted of a number of steel I-beams bent approximately to the shape of the arch axis and laid in a parallel series near the undersurface of the arch. The resulting structure might be regarded as a combination of the steel-rib arch and the concrete barrel, the concrete serving a protective as much as a structural purpose."

A detailed bridge description was presented in a 1915 article in Engineering News:

"There are five 211-ft. concrete arch spans with piers 20-ft. wide at the springing line and two 131-ft. spans with an intermediate pier 13.79-ft. wide. The two end, or abutment, piers and the pier between the 211-ft. and 134-ft. spans are 30-ft. wide. The approaches are steel girder spans on thin piers. All the river piers are skew to the center line. The 211-ft. spans are on the tangent of the 4? curves and the 134-ft. spans are on the 10? curves.

"Each of the 211-ft. spans is carried by three arched ribs of 36-ft. rise. The outside ribs are 12-ft. wide in the two end spans and 10 ft. in the intermediate spans, while all center ribs are 16 ft. wide. The reinforcing is of the Melan type, consisting of ribs of 4 x 4 x ½-in. angles laced with 3 x 3 x 5/16-in. angles (at haunches) and 2½ x -in. bars. There are six of these ribs in each 16-ft. arch rib, five in the 12-ft. and four in the 10-ft. ribs. They are braced every 30 ft. with 3 x 3 x 5/16-in. angles.

"The two 134-ft. spans over the east channel are full-barrel arches with Melan ribs of 3 x 3 x 5/16-in. angles laced with 2½ x ¼-in. bars. These are spaced 34 in. center to center and cross-braced every 30 ft. with 3 x 3 x 3/8-in. angles.

"Carrying the floor system from the ribs are transverse walls and girders supporting the floor slab and brackets supporting the sidewalk slabs and parapet-wall beam.

"The piers were constructed in open coffer-dams of Lackawanna steel sheeting, some of the sheeting being used three and four times. The coffer-dam dimensions were as follows: Pier No. 2, 46 x 121-ft.; Nos. 3 to 6, inclusive, 37 x 113-ft.; No. 8, 24 x 101.5-ft.; No. 7 (between the larger and smaller arches), 46 x 131-ft.; east abutment pier, 42 x 110-ft.

"The construction of pier No. 2 is described in what follows and is typical of all the work. After placing the underbracing for the coffer-dam, the sheetpiling was driven. On this pier (also No. 3) it was necessary at the upstream end of the coffer-dam, because of the strong current, to anchor 15-in. I-beam sills to the rock bottom with 2-in. rods to hold the lower end of the sheeting in place.

"The steel sheeting was very tight and was made entirely water-tight by a filling of coal dust and fine cinders. Sandbags were placed around the bottom of the sheeting and then pumping was started. If water came in through fissures in the rock, pumping was stopped and the bottom curse of the concrete, 5 to 6 ft. think, was placed under water. After this had set, the coffer-dam was pumped out and the remainder of the work placed dry. This was done on piers Nos. 2, 6 and 8 and partly on No. 3. Excavating for piers Nos. 6 and 8 was done entirely with orange-peel buckets. The rock in those coffer-dams was cleaned by divers with water jets. The other foundations were place dry, but always in sections, and generally four sections to each coffer-dam.

"After the footings were completed, the piers were concreted in forms which were used over and over again. The first section above the footing was carried above water level, generally leaving a center space considerable below water level to receive the ends of the steel ribs. Finally this part of the pier containing the ribs was cast in one continuous pouring. This amounted to about 7,000 yd. on piers Nos. 3, 4, 5, and 6; 1,266 yd. on Nos. 7 and 9; and 750 yd. on pier No. 8. The record run was 1,000 yd. in 22 hr.

"Pier construction was carried on through the winter except when the temperature was below zero, special precautions being taken against freezing. The forms were entirely inclosed [sic] with tarpaulins and heated with coke stoves. The sand and rock bins were supplied with heaters, and when necessary the cableway buckets for handling concrete were dipped in hot-water tanks on shore. Careful records were kept of temperatures of materials at deposit points. As a result, there was no trouble from frozen concrete.

"Concrete deposited under the water was 1:2:4 mixture. All other concrete in the piers was 1:3:6. It was mixed in batches of about 1yd. (24 ft. of stone, 12 of sand and 4 sacks of cement), two batches to each bucket. The stone was mostly traprock from Dresser Junction, Wis., crushed to a maximum size of 3 ½ in. The sand was a Minnesota product. A timber tower about 50 ft. high, with crib bottom for anchorage, was placed adjacent to the pier, standing on the river bottom. The tower had a hopper near the top, with a chute to the forms. The cableway buckets delivered concrete to the hopper, where a man regulated the discharge to the chute. The towers were picked up bodily by the cableway and moved from place to place.

"The first coffer-dam (pier No. 2) was begun Aug. 2, 1914, and the pier work was finished June 28, 1915. The river froze solid early in December, and the ice left the west channel in March and the east channel in April. Between the dates mention, 27,000 yd. of concrete was laid in pier construction.

"Falsework for the arches was begun Apr. 19, after the ice was out. One set of falsework was designed for the center ribs for the five 211-ft. spans. It was made in seven sections per span, supported by 24-in. 70-lb. I-beams, 28 ft. long on the inside sections and 26 ft. on the two end sections. The I-beams were supported on cribs made of eight 10 x 10-in. posts braced and capped and having open plank bottoms for loading with sandbags to sink them into place. These cribs were placed 28 ft. 11 in. c. to c.

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"The first arch rib, between piers Nos. 2 and 3, was poured July 8, 1915; 240 yd. of concrete was handled on one cableway in 11 hr. over the center section of the rib. The steel ribs were then riveted at the haunches during the next night and the two end sections poured simultaneously the following day, both cableways being used for 9 hr. to handle 340 yd. of concrete. The last upstream rib was poured Aug. 5. Two days later the centering was struck under the first rib and the falsework rolled over by means of a crab on pier No. 2, with block and tackle hitched to each section. The whole centering for one span was thus moved in one day.

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"The program for the rest of the work provided for pouring one rib a week until all 15 were completed. The cribs for the upstream ribs were moved and used again for the third ribs on the downstream side. The centering of the last rib was moved over into place in 2 hr. 40 min.

"In October, 1915, the timber for the first three 211-ft. spans was moved over to the 134-ft. spans in order to finished the arches before cold weather sets in. The transverse walls are being put in, and only the floor proper will remain to be put in next spring. It is expected that the new bridge will be opened to travel not later than June1, 1916.

"The alignment of the bridge and skew of the piers necessitated an elaborate system of location. The triangulation had for its base the center tangent line of the bridge. A series of large triangles was laid out on either side of this base line, regard being given to prominent points as targets for the apices of the triangles.

"A secondary triangulation system was calculated, with proper attention to balancing errors for the location of the instrument platforms. Upon this the intersection points of pier, transverse center lines and base line of platforms were accurately established. These intersections were established with ordinary transits reading to 30 sec. Seconds were interpolated on the platforms by means of thread intersections; the minute next great and that next smaller to the actual triangle calculated to the nearest second were ready by the instrument man and recorded on the platform. Actual measurements show a maximum error of ¼-in. in 211 ft."

The bridge had ornamental railing installed in 1939, and was remodeled in 1979-1980. The rehabilitation consisted of complete deck removal; new light standards; raising of the spandrel columns; raising of the roadway grade by 5 feet; new approach pads; removal, cleaning and reinstallation of the 1939 railing; and pier repair.

PART IV. SOURCES OF INFORMATION

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PART V. PROJECT INFORMATION

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MHPR NO. HE-MPC-0165

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Appendix D

Engineering News Article

A 2,223-Ft. Concrete-Arch Bridge Built on Reverse Curve

By A. M. Richter*

SYNOPSIS—A long bridge, curved in plan, with arches of two types; 211-ft. spans, with three arch ribs carrying crosswalls, and 134-ft. spans, with barrel arches. All are reinforced by steel truss ribs. Concrete was placed by a cableway of 2,038-ft. span, handling drop-bottom buckets. The entire construction is being done by the city on the day-labor system.

The construction of the Third Ave. reinforced-concrete arch bridge across the Mississippi River at Minneapolis, Minn., has involved unusual engineering difficulties and presents features of interest in both design and construction. The bridge is notable for its size, because it is curved in plan and because it is being built on the day-labor system, which has been employed for some time in the city engineering department. Some stages of the work under construction are shown in Fig. 1, and the general plan is shown in Fig. 2.

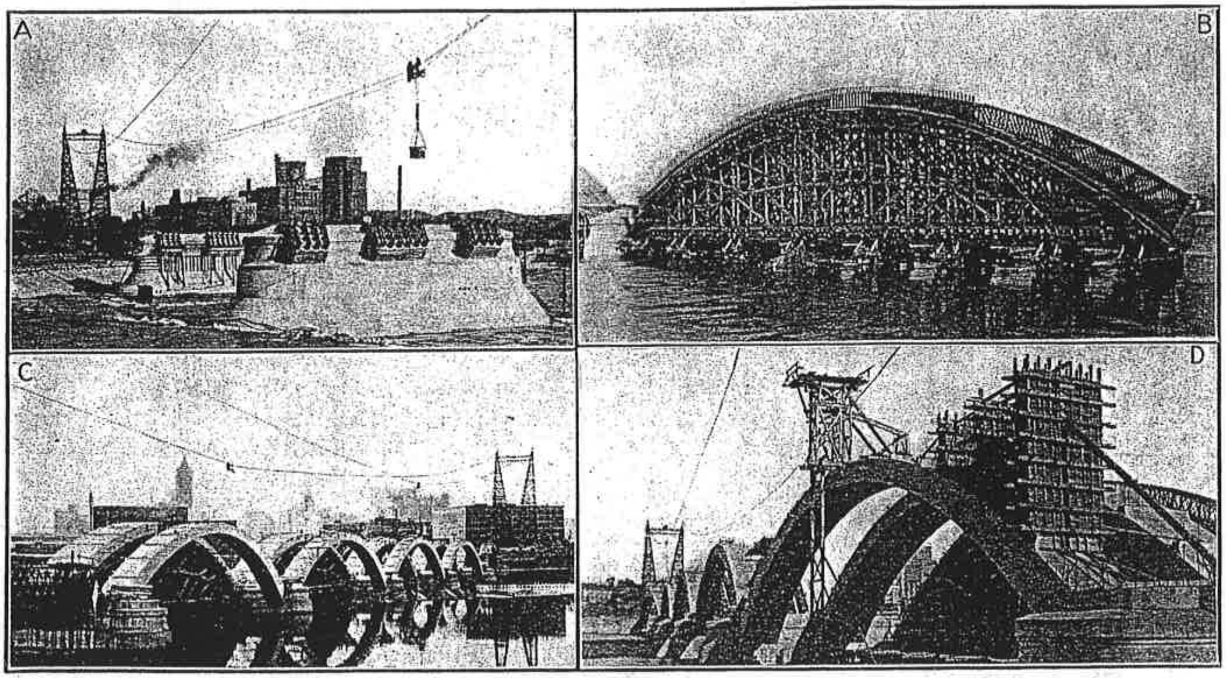

FIG. 1. CONSTRUCTION OF REINFORCED-CONCRETE ARCH BRIDGE OVER MISSISSIPPI RIVER AT THIRD AVE., MINNEAPOLIS, MINN.

A—Piers 4 and 5, showing ends of steel arch ribs, and tail tower of the cableway (Nov. 20, 1914). B—Falsework and steel reinforcing ribs for first 211-ft. arch (July 2, 1915). C—Arch ribs of the five 211-ft. spans; the third line of ribs (at the left) not built (Oct. 12, 1915). D—Fifth 211-ft. span, with two ribs completed and falsework and steel reinforcement in place for third rib. On the outer rib are forms for the cross walls, and a tower for the concreting chute

TROUBLESOME GEOLOGICAL CONDITIONS

Of special interest are the geological conditions which affect the foundation work and which account for the curved line of the bridge. The geological formation of the river bottom at the bridge site consists of a limestone bed about 15 ft. thick, which is practically bare in the west channel and covered only by a few feet of silt and sand in the east channel. The limestone ledge rests on the St. Peter sandstone, which is about 600 ft. in depth. This sandstone is readily excavated with picks and is easily eroded by the action of water, especially when under a head. The limestone extends upstream about 500 ft. from the bridge and downstream about 700 ft. to the crest of St. Anthony Falls.

In the early construction of works to utilize the power from St. Anthony Falls a peculiar advantage was afforded by the facility with which tunnels could be excavated in the soft sandstone. The water was led from the mill pond in a canal above the limestone, and the tunnels served as tailraces. In 1869, however, one of these tunnels had reached a point near the foot of Nicollet Island (2,000 ft. from the point of beginning), when water poured in from a break in the overlying bed of limestone. The project had to be abandoned, and the United States Government made extensive repairs to close the break in order to insure continuance of the water-power and restore the original conditions as far as possible. Another break occurred in 1876. The locations of these breaks in the river bed and their relation to the bridge projects are shown on the plan.

With the growth of the city, there has been strong demand for a bridge in the neighborhood of Third Ave. South. It was desired that this should be of handsome appearance, and a concrete arch structure was considered

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*"Minneapolis Journal," Minneapolis, Minn.

the best to meet the requirements of the situation. In 1912 the City Council commissioned the Concrete-Steel Engineering Co., of New York, to prepare designs for a reinforced-concrete arch bridge between Third Ave. South and First Ave. Southeast. The location is indicated by a dotted line on the plan.

The design was subjected to a public hearing before the engineers of the United States War Department in 1913. The water-power companies had not favored any bridge project and announced that, if necessary, they would resort to litigation to oppose any work threatening danger to the falls.

Shortly after affairs had reached this stage, Frederick W. Cappelen was elected city engineer. While bridge engineer for the city in previous years he had built six bridges across the Mississippi River and acquired a thorough knowledge of river conditions. He refused to approve the proposed location. The City Council then rejected the plans and instructed him to design a steel bridge that could be constructed without endangering the falls or affecting water-power rights.

His proposed location is shown on the plan, and his design included one span of 434 ft. to clear entirely the area of the limestone breaks. The trusses were to be of the parabolic through-truss type. In the face of objections (based mainly on æsthetic considerations) the City Council approved the plans and directed the engineer to proceed with construction.

At this time, however, Mr. Cappelen conceived the idea that by adopting a curved location for the line of the bridge, a design satisfactory to all parties might be worked out. On investigation it was found that at one point the limestone break could be cleared by a concrete arch of 211-ft. clear span. A revised

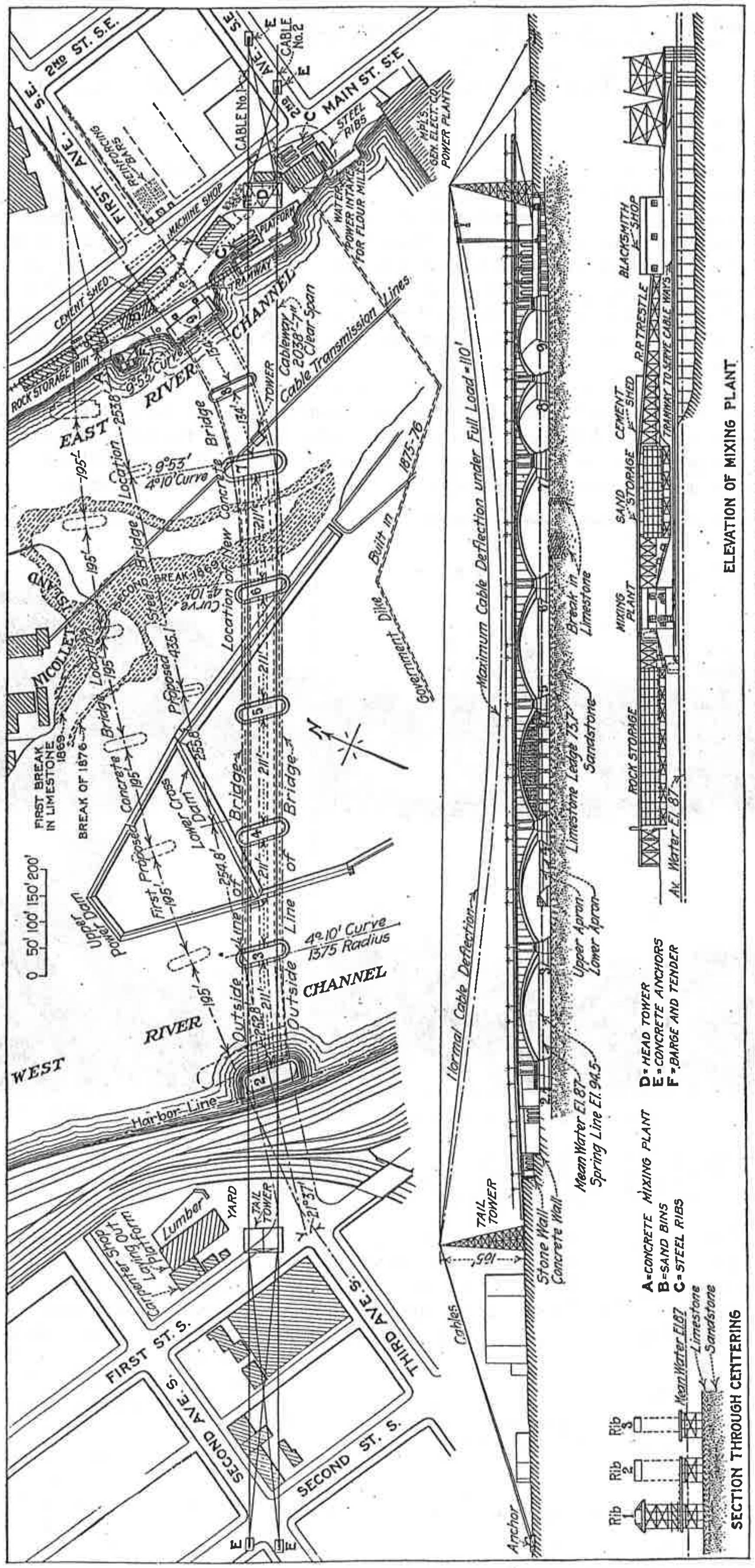


FIG. 2. PLAN AND ELEVATION OF THE THIRD AVE. REINFORCED-CONCRETE ARCH BRIDGE OVER THE MISSISSIPPI RIVER AT MINNEAPOLIS, MINN.

plan for the desired ornamental structure was then prepared. This proved satisfactory to all parties and was finally adopted.

The bridge is 2,223 ft. long and consists of seven main river spans. It has a 54-ft. roadway (with double-track street railway) and two 12-ft. sidewalks. The loading provides for two 40-ton cars and 100 lb. per sq.ft. uniform load. The floor system is designed to carry a 24-ton road roller on a space of 12x18½ ft. The center line starts at the intersection of Third Ave. South and First St. at an angle of 21° 39′, and is on a tangent for 151 ft. to a 4° curve 330.2 ft. long. A tangent 719 ft. long continues to a curve consisting of a 4° compounded into a 10° curve in a distance of 526.83 ft., bringing the center line of the bridge to that of First Ave. Southeast. The bridge is level, with a grade of 0.9% on the east approach and 3.4% on the west approach.

There are five 211-ft. spans with piers 20 ft. wide at springing line and two 134-ft. spans with an intermediate pier 13.79 ft. wide. The two end, or abutment, piers and the pier between the 211-ft. and 134-ft. spans are 30 ft. wide. The approaches are steel girder spans on thin piers. All the river piers are skew to the center line. The 211-ft. spans are on the tangent of the 4° curves and the 134-ft. spans are on the 10° curves.

Each of the 211-ft. spans is carried by three arched ribs of 36-ft. rise, as shown in the cross-section, Fig. 3. The outside ribs are 12 ft. wide in the two end spans and 10 ft. in the intermediate spans, while all center ribs are 16 ft. wide. The reinforcing is of the Melan type, consisting of ribs of 4x4x½-in. angles laced with 3x3x$\frac{5}{16}$-in. angles (at haunches) and 2½x⅜-in. bars. There are six of these ribs in each 16-ft. arch rib, five in the 12-ft. and four in the 10-ft. ribs. They are braced every 30 ft. with 3x3x$\frac{5}{16}$-in. angles.

The two 134-ft. spans over the east channel are full-barrel arches (Fig. 3) with Melan ribs of 3x3x$\frac{5}{16}$-in. angles laced with 2½x¼-in. bars. These are spaced 34 in. c. to c. and cross-braced every 30 ft. with 3x3x⅜-in. angles.

Carrying the floor system from the ribs are transverse walls and girders supporting the floor slab and brackets supporting the sidewalk slabs and parapet-wall beam. These are shown in Figs. 3 and 4.

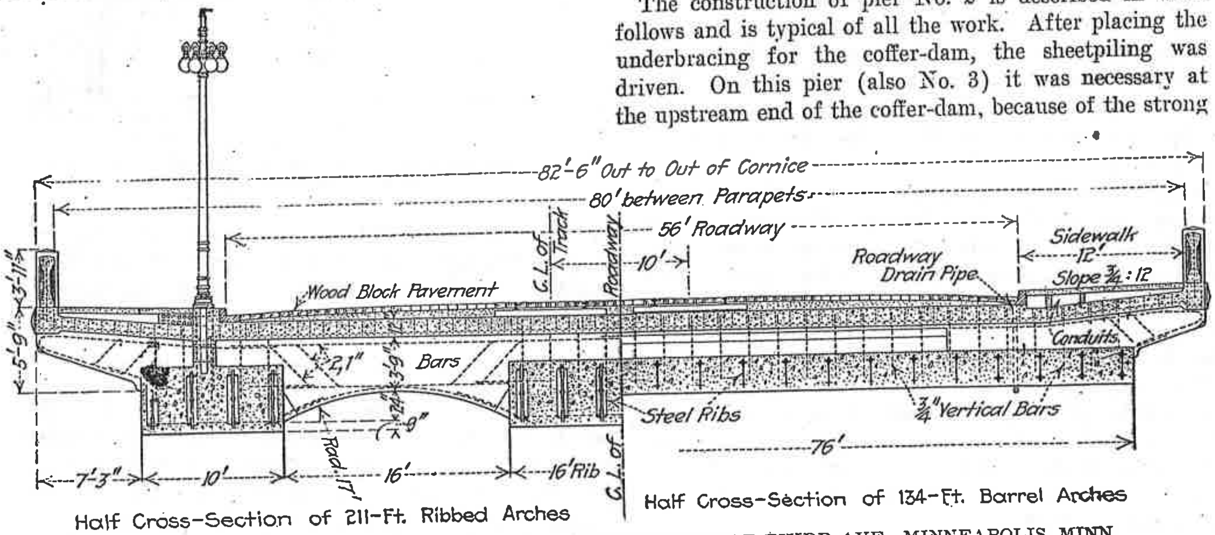
The piers were constructed in open coffer-dams of Lackawanna steel sheeting, some of the sheeting being used three and four times. The coffer-dam dimensions were as follows: Pier No. 2, 46x121 ft.; Nos. 3 to 6, inclusive, 37x113 ft.; No. 8, 24x101.5 ft.; No. 7 (between the larger and smaller arches), 46x131 ft.; east abutment pier, 42x110 ft.

Practically no silt was found on top of the ledge at piers Nos. 2, 4 and 5, but there were from 6 to 12 in. at No. 3, 5 ft. at the downstream and 7 ft. at the upstream ends of No. 6, 9 and 10 ft. at No. 7, 8 ft. originally and scouring out to 3 ft. minimum at No. 8 and 8 ft. at pier No. 9. The depth of water was 16 ft. at piers Nos. 7, 8 and 9, 12 ft. at Nos. 2, 3, 6 and 12 and 5 ft. at Nos. 4 and 5.

The construction of pier No. 2 is described in what follows and is typical of all the work. After placing the underbracing for the coffer-dam, the sheetpiling was driven. On this pier (also No. 3) it was necessary at the upstream end of the coffer-dam, because of the strong current, to anchor 15-in. I-beam sills to the rock bottom with 2-in. rods to hold the lower end of the sheeting in place. This is shown in Fig. 5.

The steel sheeting was very tight and was made entirely water-tight by a filling of coal dust and fine cinders. Sandbags were placed around the bottom of the sheeting and then pumping was started. If water came in through fissures in the rock, pumping was stopped and the bottom course of concrete, 5 to 6 ft. thick, was placed under water. After this had set, the coffer-dam was pumped out and the remainder of the work placed dry. This was done on piers Nos. 2, 6 and 8 and partly on No. 3. Excavating for piers Nos. 6 and 8 was done entirely with orange-peel buckets. The rock in these coffer-dams was cleaned by divers with water jets. The other foundations were placed dry, but always in sections, and generally four sections to each coffer-dam.

The silt on top of the bedrock was full of old water-soaked logs that caused trouble in the excavation. Two slight breaks occurred in coffer-dam No. 3. Both occurred during night shifts, and might have been prevented had there been an intimation of a break during the day.



Half Cross-Section of 211-Ft. Ribbed Arches

Half Cross-Section of 134-Ft. Barrel Arches

FIG. 3. CROSS-SECTION OF THE CONCRETE ARCH BRIDGE AT THIRD AVE., MINNEAPOLIS, MINN.

CONSTRUCTION OF THE PIERS

After the footings were completed, the piers were concreted in forms which were used over and over again (Fig. 6). The first section above the footing was carried above water level, generally leaving a center space considerably below water level to receive the ends of the steel ribs. Finally this part of the pier containing the ribs was cast in one continuous pouring. This amounted to about 1,000 yd. on piers Nos. 3, 4, 5 and 6, 1,266 yd. on Nos. 7 and 9 and 750 yd. on pier No. 8. The record run was 1,000 yd. in 22 hr.

Pier construction was carried on through the winter except when the temperature was below zero, special precautions being taken against freezing. The forms were entirely inclosed with tarpaulins (Fig. 6) and heated with coke stoves. The sand and rock bins were supplied with heaters, and when necessary the cableway buckets for handling concrete were dipped in hot-water tanks on shore. Careful records were kept of temperatures of materials at deposit points. As a result, there was no trouble from frozen concrete.

Concrete deposited under water was a 1 : 2 : 4 mixture. All other concrete in the piers was 1 : 3 : 6. It was mixed in batches of about 1 yd. (24 ft. of stone, 12 of sand and 4 sacks of cement), two batches to each bucket. The stone was mostly traprock from Dresser Junction, Wis., crushed to a maximum size of 3½ in. The sand was a Minnesota product. A timber tower about 50 ft. high, with crib bottom for anchorage, was placed adjacent to the pier, standing on the river bottom. The tower had a hopper near the top, with a chute to the forms. The cableway buckets delivered concrete to the hopper, where a man regulated the discharge to the chute. The towers were picked up bodily by the cableway and moved from place to place.

The first coffer-dam (pier No. 2) was begun Aug. 2, 1914, and the pier work was finished June 28, 1915. The river froze solid early in December, and the ice left the west channel in March and the east channel in April. Between the dates mentioned, 27,000 yd. of concrete was laid in pier construction.

FALSEWORK FOR THE ARCH SPANS

Falsework for the arches was begun Apr. 19, after the ice was out. One set of falsework was designed for the center ribs for the five 211-ft. spans. It was made in seven sections per span, supported by 24-in. 70-lb. I-beams, 28 ft. long on the inside sections and 26 ft. on the two end sections. The I-beams were supported on cribs made of eight 10x10-in. posts braced and capped and having open plank bottoms for loading with sandbags to sink them into place. These cribs were placed 28 ft. 11 in. c. to c. (Fig. 1).

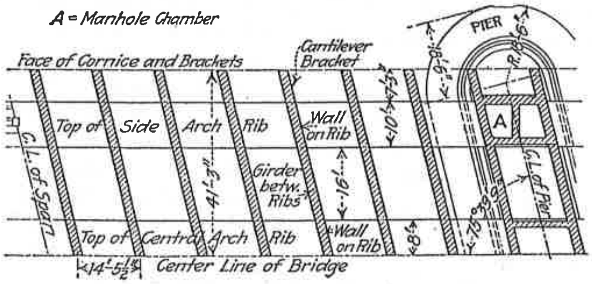


FIG. 4. PART PLAN OF 211-FT. ARCH SPAN

Showing the arch ribs and cross walls

The falsework to carry the ribs was of 8x8-in. posts braced with 2x10-in. planks. The bents were capped and furnished with wedges under caps supporting the joists which carried the lagging and the framework for the rib. The lagging and side forms were of 1-in. tongued-and-grooved plank, the forms being supported by 4x4-in. posts and 4x6-in. longitudinal timbers.

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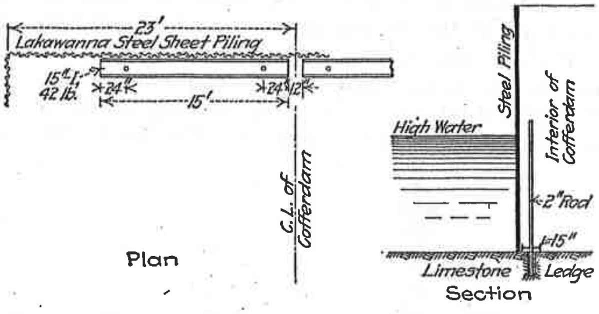

FIG. 5. SUPPORT FOR BOTTOM OF STEEL SHEETING FOR COFFER-DAM

CONCRETING THE ARCHES OF THE BRIDGE

The first arch rib, between piers Nos. 2 and 3, was poured July 8, 1915; 240 yd. of concrete was handled on one cableway in 11 hr. over the center section of the rib. The steel ribs were then riveted at the haunches during the next night and the two end sections poured simultaneously the following day, both cableways being used for 9 hr. to handle 340 yd. of concrete. The last upstream rib was poured Aug. 5. Two days later the centering was struck under the first rib and the falsework rolled over by means of a crab on pier No. 2, with block and tackle hitched to each section. The whole centering for one span was thus moved in one day.

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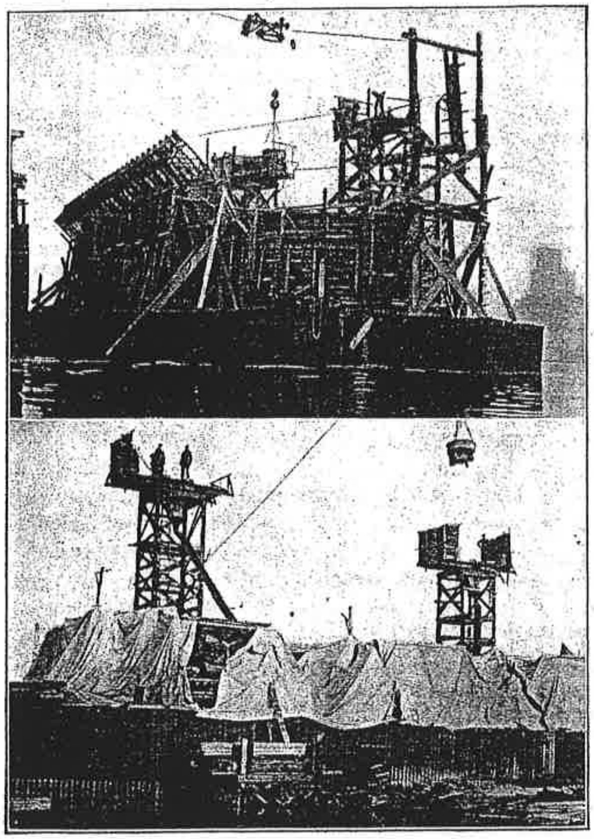

FIG. 6. CONCRETING THE PIERS FOR THE THIRD AVE. BRIDGE

Upper View—Work at pier No. 7 in April, 1915. Lower View—Concreting pier No. 3 during freezing weather in December, 1914

Handling Material by Cableways

The river conditions, as well as railway operations at the site of the bridge, led to the use of a Lidgerwood double cableway to handle the work. This has two 2½-in. steel main cables and is operated by two 75-hp. engines and a 150-hp. tubular boiler. The cables have a working span of 2,020 ft., giving a capacity of 6 tons each at a speed of 1,200 ft. per min. constant work and 10 tons for occasional demand. Fig. 2 shows the location of the cableway and the bridge.

The towers had to be 165 ft. high to insure clearance. The location of the anchors was a difficult problem, which was finally solved by utilizing space in the streets in such a way as to interfere but little with traffic. The towers are 39x76½ ft. at the base. They are built of Douglas-fir timbers, 12x14 to 10x10 in., and capped with oak headblocks, 20x20 in. and 7 ft. long.

The anchors of the head tower (east side) contain 122 yd. of concrete each and weigh 237 tons. They are 27x12½x10 ft. and entirely under the ground. The tail-tower anchors are 22x15x10 ft. each, containing 120 cu.yd. and weighing 230 tons. They are half buried in the street. Each tower is guyed by two 1½-in. lines. Difficulty was met in carrying one of the cable lines over a seven-story building about 100 ft. from the tail tower and in supporting the same line to afford clearance on the street at the anchor, as indicated in Fig. 2. High-water conditions, inability to get near the dam in rowboats and the necessity of precaution against dropping the heavy cable and cutting a power-transmission line, as well as danger of interference with trains on several tracks, were among the complications involved. The task was finally accomplished by using a rope line and then a 1⅛-in. messenger cable to trolley across the 2½-in. cables.

The cableways are located to serve all piers except No. 9, the abutment pier on the east side. This is near the mixing plant and was served direct with towers, elevators and derricks.

The towers were framed on the ground and then erected. The framing began Apr. 1, 1914, and was completed Aug. 1. It was done on the day-labor system, with a crew of 7 ironworkers, 20 carpenters and 15 laborers. The erection cost is estimated at $5,000 and the machinery and special equipment cost approximately $21,000.

Concrete-Mixing and Handling Plant

A difficult part of the work was that of arranging the working plant. There was no room except on Main St. (on the east side), and here some travel had to be provided for. The Great Northern R.R. maintains two industrial tracks on the street and owns the land to the river, which is leased to various concerns. The railway company, however, cancelled the leases and made it possible to establish the mixing plant at that point. Concrete, steel and machinery were located on the east side and lumber on the west side. The layout of the construction plant and supply yards is shown in Fig. 2.

The concrete-mixing plant has a capacity of 400 yd. in 8 hr. There is a 2,000-yd. rock-storage bin 125 ft. long and an 800-yd. sand bin 74 ft. long. The bins are 27½ ft. wide and 12 ft. deep, and are provided with boilers for heating. The material track reaches the top of the bins by a trestle 460 ft. long, with a grade of 4%. Stone and sand are delivered through bottom openings into cars of 24-cu.ft. capacity, which serve the mixing platform by a cable incline operated by hoisting engines. The cement shed, 20x200 ft., of 5,000 bbl. capacity, is served by a sidetrack.

There are two cube-type mixers, each of 1-yd. capacity. They are equipped with steam lines, and in freezing weather the water is run through a reheater. The water tanks are fitted with gages for measuring the supply to each batch. The sand and stone are dumped into the hopper at one level and the cement from a higher level, and the entire charge then spouted into the mixer.

The concrete is discharged into 2-yd. drop-bottom buckets. These are circular in shape, with conical bottoms, and have legs so that they stand upright on flat cars. These cars are hauled between the mixer plant and the cableway on a double-track cable tramway operated by American reversible hoisting engines.

At the city workhouse prisoners are making the ornamental concrete railing shown in Fig. 7. The concrete is made with mica-spar crystals (from Crown Point, N. Y.) and is cast in steel molds to obtain a smooth finish and polish. The cement workers' union protested against this plan, but finally withdrew its objections. This railing is estimated to cost $20,000.

Other Construction Plant

Space and trackage were very limited on the west side, but three lots at the street level were loaned by the Rock Island Lines. Here was erected a 48x135-ft. platform for framing timber and laying out the centering; also a small planing and ripping mill, tool house, men's house and engineers' field office. The timber is piled high on account of limited space. It is handled by a derrick with a 47-ft. boom and a 17-ft. mast.

The machine shop, 30x60 ft., is also on the east side. It has a blacksmith forge, bolt-threading machine, emery grinder, miscellaneous tools and one 4-ton and two 2-ton chain blocks. A track runs under the shop from the cableway tower, so that equipment may be run in for repairs. Two movable derricks in the same location have 65-ft. masts and 50-ft. booms. Completing the east-side layout are the steel storage yards, served by a derrick with 65-ft. mast and 75-ft. boom. Each Melan rib is in its own common pile, some running 35 sections high. The reinforcing steel and bar iron are in separate places bundled and labeled for length and location.

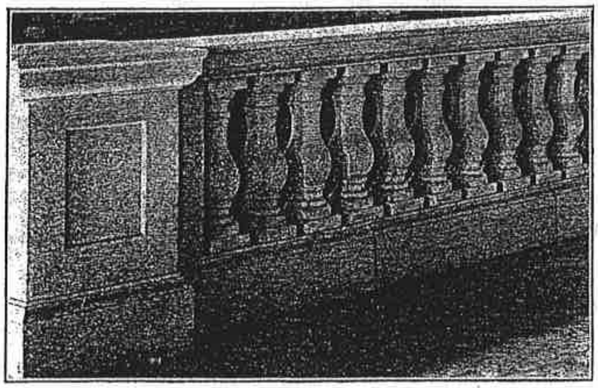

FIG. 7. CONCRETE RAILING FOR THE THIRD AVE. BRIDGE

Electric motors are used in the mill and machine shop and for operating the concrete mixers. Electric drills are employed in timber framing. All pumping was done with electric belt-driven centrifugal pumps. Compressed air was used for riveting.

The total cost was estimated at $650,000. The bridge will require 53,000 cu.yd. of concrete, 963 tons of structural steel for the arch ribs, 800 tons of reinforcing bars for piers and 1,500,000 ft. of lumber for centering. The prices for materials delivered at the site were as follows: Crushed traprock, $1.45 per cu.yd.; washed sand, 75c.; cement, $1.20 per bbl.; structural steel (including erection bolts and nuts), $53.50 per ton; reinforcing bars, $1.429 per 100 lb.; Lackawanna 7-in. steel sheetpiling, $1.63¾ per 100 lb.; coal (lump), $4.15 to $4.25 per ton; electric current for light and power, 2.9c. per kw.-hr.

The wages paid by the city (for an 8-hr. day) were as follows: Foremen, $4.50 to $6; ironworkers, $5; carpenters, hoisting enginemen and electricians, $4; handymen, $2.65; laborers, $2.50; water boys, $1.40; teams, $5.

The location was determined and the general design made by Frederick W. Cappelen, City Engineer (with whose aid and approval these notes were prepared). All construction was done under his direct supervision. He also designed the methods of construction, the working plant, falsework, etc. The assistant engineers, all employed in the city eigineering department, were as follows: K. Oustad, Bridge Engineer; William Elsberg, Superintendent of Construction; and John E. Lawton, Junior Engineer.

The construction foremen were N. Linstrom, for the forms, concreting and falsework, and J. F. McAuley, for the mechanical equipment. The Concrete-Steel Engineering Co. furnished the detail plans under its original commission of 1912. Its resident engineer on the work was Charles F. Bornefeld.

⁂

Large Water-Works Figures

Municipal ownership of water-works prevails in 155 of the 204 cities of the United States having an estimated population of 30,000 or more in 1915, according to a statement just issued by the United States Bureau of the Census. The total estimated value of these municipally owned works is $1,071,000,000. The distribution systems in the 155 cities comprise a total of 36,936 mi. of mains, 330,593 fire hydrants and 1,787,448 meters. The total water consumption in the 155 cities for the year covered by the report was 1,326,028,000,000 gal., supplied to 26,200,000 people, giving an average daily per capita consumption of 139 gal. On the range of water consumption and the effect of meters, the Bureau of the Census says:

> The greatest daily consumption of water per inhabitant, 430 gal., is reported for Tacoma, Wash., and the smallest, 34 gal., for Woonsocket, R. I. In the former city 8% of the water is metered and in the latter 98%. The tendency of meters to curtail greatly the use of water is strikingly shown by a comparison of the figures for the 26 cities in which the entire water-supply is metered with those for the 26 cities in which not more than 25% is metered. In the former group the average daily consumption per inhabitant ranges from 42 gal. in Brockton, Mass., to 179 gal. in Columbia, S. C., and in only 7 cities does it exceed 100 gal. In the latter group it varies from 43 gal. in Savannah, Ga., to 430 gal. in Tacoma, Wash., and in only 3 cities does it fall below 100 gal.

The number of cities with water-purification plants is not given. Instead the statement is made that in the 155 cities of over 30,000 population having municipal ownership there are in operation a total of 87 settling reservoirs, in which are treated 958,600,000 gal. a day; 54 coagulation plants, treating 492,100,000 gal.; 527 sand filters, treating 598,700,000 gal.; and 427 mechanical filters with a daily output of 468,200,000 gal. The surprising total of 1,972,900,000 gal. of water per day is treated by some disinfection process.

The range of cost of water treatment per 1,000,000 gal. is reported as from 4c. per 1,000,000 gal. in Chicago, Ill., for disinfection, to $17.46 in Columbus, Ohio, for "mechanical filtration and chemical sterilization."

More detailed information regarding both municipally owned water-works and various other works and operations of the larger cities of the United States will be published later on under the title "General Statistics of Cities, 1915," compiled under the direction of Starke M. Grogan, Chief Statistician for Statistics of Cities. Sam L. Rogers is Director of the Bureau of the Census.

E

Appendix E

AET Subsurface Exploration Report

AMERICAN
ENGINEERING
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CONSULTANTS
- ENVIRONMENTAL
- GEOTECHNICAL
- MATERIALS
- FORENSICS

REPORT OF SUBSURFACE EXPLORATION

Pier 5, 3rd Avenue Bridge

Minneapolis, Minnesota

Report No. 01-05995

Date:

April 14, 2014

Prepared for:

HDR Engineering, Inc.
701 Xenia Avenue South; Suite 600
Minneapolis, MN 55416

www.amengtest.com

AMERICAN
ENGINEERING
TESTING, INC.

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• MATERIALS
• FORENSICS

April 14, 2014

HDR Engineering, Inc.
701 Xenia Avenue South; Suite 600
Minneapolis, MN 55416

Attn: Jacob Bronder, P.E.

RE: Subsurface Exploration
Pier 5; 3rd Avenue Bridge
Minneapolis, Minnesota
Report No. 01-05995

Dear Mr. Bronder:

American Engineering Testing, Inc. (AET) is pleased to present the results of our geotechnical exploration services for the referenced project.

We are submitting three bound and one unbound copies of the report to you.

Unless notified to do otherwise, we routinely retain representative samples recovered from the test borings for a period of 30 days. Notify us if you want to retain the samples longer.

Please contact us if you have any questions about the report.

Sincerely,
American Engineering Testing, Inc.

James C. Rudd, P.E.
Vice President/Principal Engineer
Phone: (651) 659-1367
Fax: (651) 659-1347
jrudd@amengtest.com

SIGNATURE PAGE

Prepared for:

HDR Engineering
701 Xenia Ave. S.
Minneapolis, Minnesota 55416

Prepared by:

American Engineering Testing, Inc.
550 Cleveland Avenue North
St. Paul, Minnesota 55114
(651) 659-9001/www.amengtest.com

James C. Rudd, P.E.
Principal Engineer

I hereby certify that this plan, specification, or report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under Minnesota Statute Section 326.02 to 326.15

Date: 4/14/14 **License #: 13996**

TABLE OF CONTENTS

1.0 INTRODUCTION

This report presents the results of a subsurface test boring and temporary piezometer installation at Pier 5 of the 3rd Avenue Bridge in downtown Minneapolis.

2.0 SCOPE OF SERVICES

AET's services on this project were done in accordance with our proposal dated January 22, 2014. The work scope contained in this report consists of the following items:

- One test boring was drilled through Pier 5 and into the underlying bedrock formations until the St. Peter Sandstone was encountered. The final test boring depth was 85 feet below the bridge deck.
- A vibrating wire piezometer was installed within the St. Peter Sandstone formation to measure piezometric pressure. In our proposal, we had planned to install a standpipe piezometer instead of a transducer piezometer. We decided to switch in order to effectively isolate the piezometer from the overlying river head. Further discussion is given in Section 6.3 of this report. The piezometer was monitored periodically for a period of 5 days, and then removed.
- A petrographic analysis of a portion of the recovered concrete core was run.
- The boring was sealed with neat cement grout in accordance with Minnesota Department of Health regulations.

Our proposal also included hand probes of the river bottom, sediment sampling of river sediments, environmental testing, and concrete strength testing. These scope items have not yet been performed.

3.0 SUBSURFACE EXPLORATION AND TESTING

3.1 Location

The site location is shown on Figure 1, included in Appendix A. The location of the test boring was specified by HDR to be directly over an intermediate concrete pier ledge located about 57 feet below the bridge deck. The location of the test boring on the bridge deck is shown on Figure 2. An illustration of the location of the pier ledge is shown on Figure 3.

3.2 Test Boring Procedure and Results

The bridge deck was cored with a 6 inch diameter core barrel at the specified location. We then set a length of HW casing between the bridge deck and the intermediate pier ledge approximately 57 feet below the bridge deck.

The concrete was then cored using NQ wireline coring equipment starting at the intermediate pier ledge and extending down a distance of 12 feet. At the base of the concrete, we encountered a 1 foot thick void space between the bottom of concrete and the top of bedrock.

We then set a second casing (NW casing) inside the outer casing down to the top of bedrock. We then continued to core through the bedrock using NQ wireline coring equipment. The limestone and shale bedrock was cored until the sandstone was encountered at a depth of 80.5 feet below the bridge deck.

A log of the test boring is included in Appendix A. An illustration of the test boring is shown on Figure 3. Photographs of the concrete and rock core samples are included in Appendix A.

Sample types described as "HQ" are 2.5 inch diameter core samples collected with wire line diamond bit rock coring equipment. Core runs were 60 inches long. The core recovery length and percentage are shown on the logs. Descriptions of the rock classification terminology is given on the standard sheet included in this appendix.

A split barrel sample was collected from a depth of 80-81 feet depth. The retrieved sample showed that the shale/sandstone interface was located at a depth of 80.5 feet below the bridge deck.

In order to set the temporary piezometer, we then advanced the borehole using a tricone drill bit a distance of approximately 5 feet into the sandstone formation. Further discussion of the piezometer installation is given in Section 3.3, below.

Upon completion of the work, we removed the piezometer and grouted the bedrock with neat cement grout. We then set an inflatable plug at the base of the concrete pier, just above the 1 foot void that we had encountered. We then also grouted the borehole through the concrete pier with neat cement grout.

3.3 Piezometer Installation and Monitoring

We originally had planned to seat the NW casing in the Glenwood Shale formation, and then install a standpipe piezometer in the sandstone below the sealed casing. The shale formation was thinner than expected; therefore, we were unable to seal the NW casing since it was already through the shale. As an alternate method to measure the piezometric head in the sandstone formation, we installed a vibrating wire (VW) piezometer in the sandstone using the "fully grouted" method. A description of the piezometer specifications and installation method are given in the following sections.

3.3.1 VW Piezometer Specifications

Geokon Model 4500S, installed by "fully grouted method"
Cable length: 100 feet

Specifications	
Range	350 kPa (50 psi)
Over Range	2× rated pressure
Resolution	0.025% F.S.*
Accuracy	±0.1% F.S.*
Linearity	<0.5% F.S.*
Temperature Range	−20°C to +80°C
Length × Diameter	133 × 19.1 mm

3.3.2 VW Piezometer Installation & Monitoring

Prior to installing the vibrating wire sensor, as well as immediately after installation in the borehole, AET personnel recorded initial (baseline) sensor readings and sensor temperatures. The vibrating wire piezometer was installed by the "Fully Grouted Method" (identified in the Geokon manual for this sensor as "Installation Method C"). A summary of the piezometer readings are shown in Table A-1, Appendix A.

After completion of all piezometer readings, the special piezometer grout was drilled out and the piezometer was removed. The bedrock was then re-grouted with neat cement grout per Minnesota Department of Health regulations.

3.3.3 Piezometer Results

Based on the piezometer readings, the piezometric level within the sandstone formation is below the elevation of the VW piezometer (elev. 768.5 feet).

* FS = The value of x at full scale indicating the upper limit of the measurement range capability of the instrument or measurement system

3.4 Petrographic Analysis of Concrete

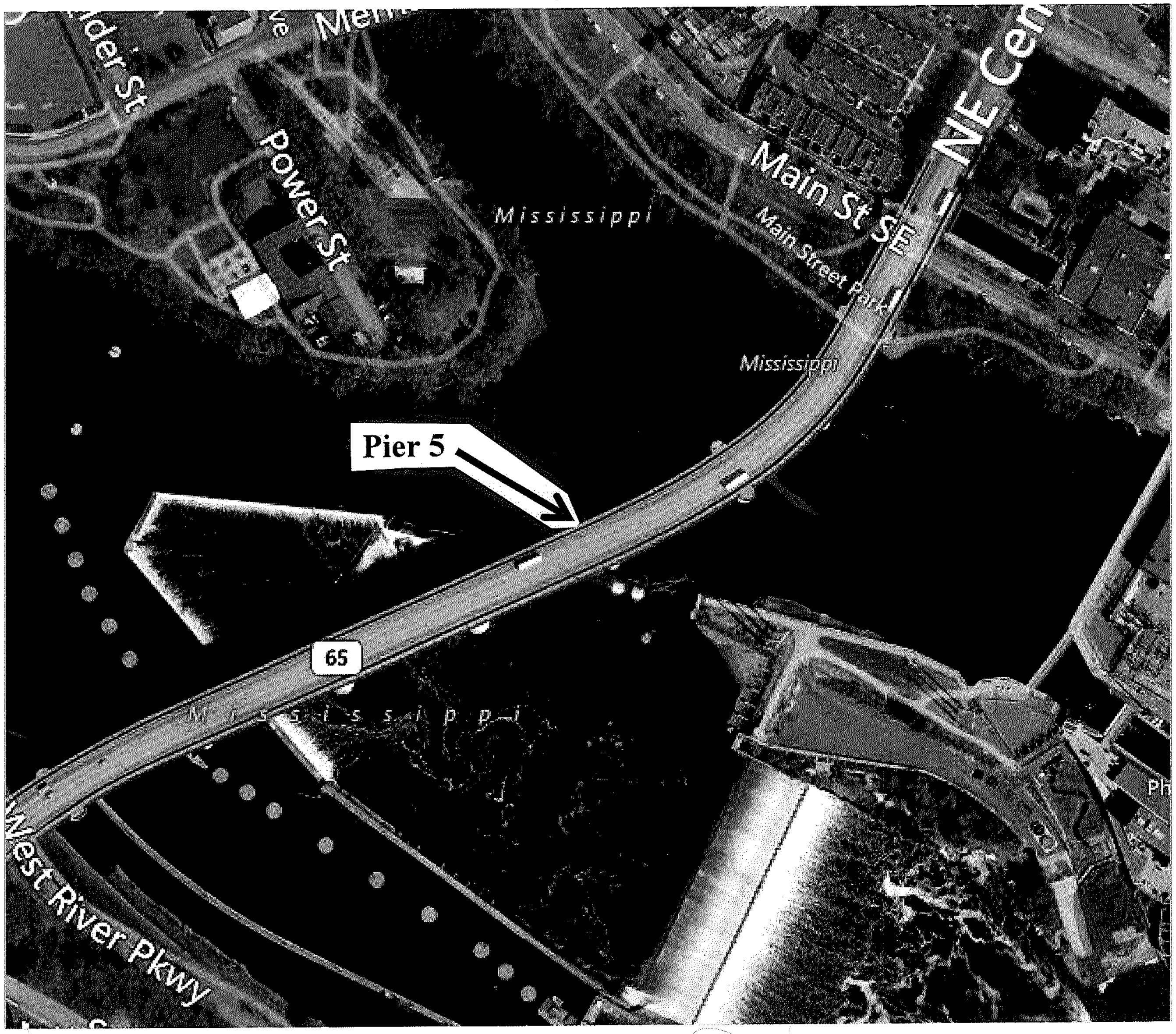
A sample of the concrete core from 60 to 61 foot depth interval was submitted to our petrographic laboratory for analysis. The report of the petrographic analysis is included in Appendix B.

4.0 LIMITATIONS

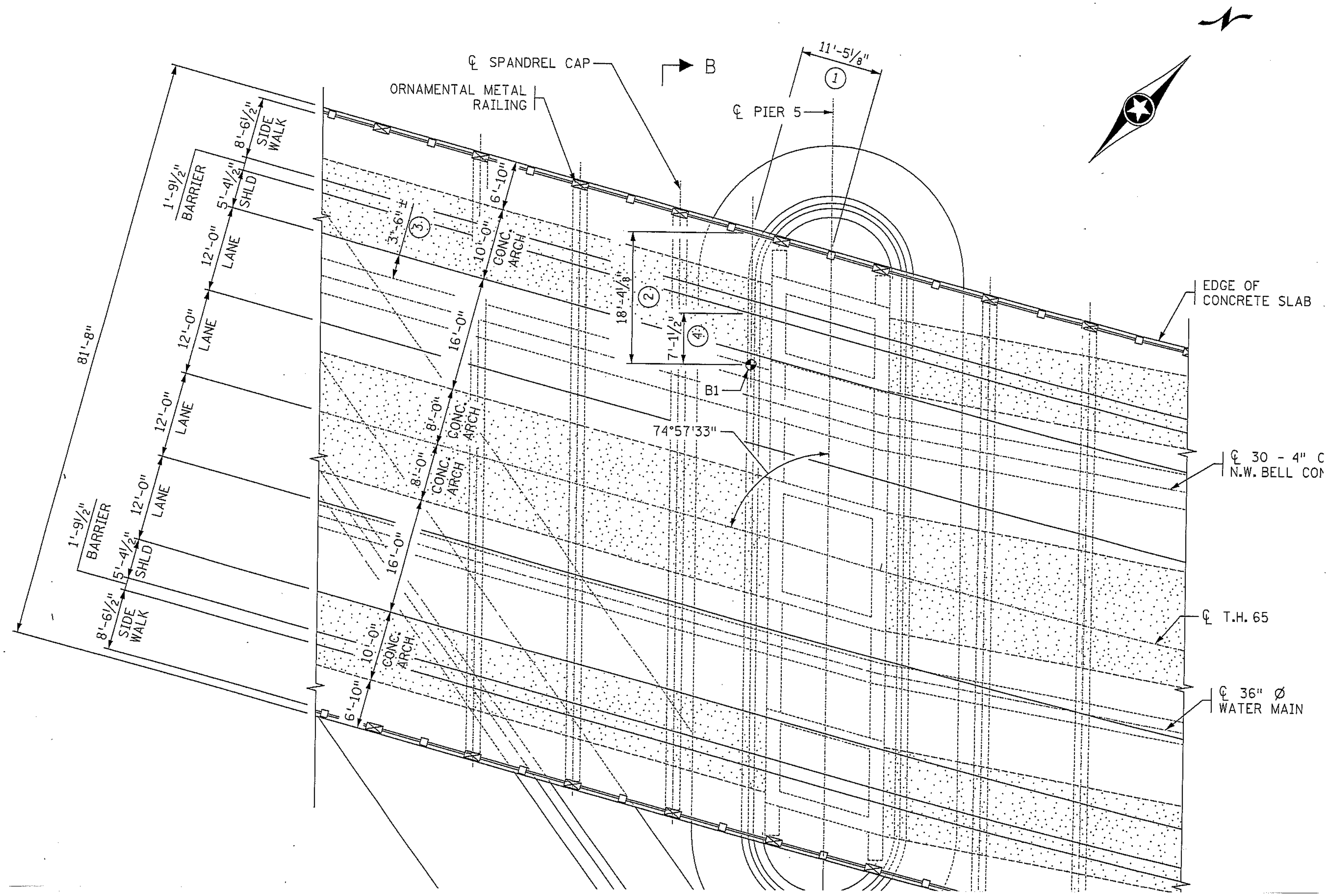
Within the limitations of scope, budget, and schedule, our services have been conducted according to generally accepted geotechnical engineering practices at this time and location. Other than this, no warranty, either expressed or implied, is intended.

Appendix A

Figure 1: Site Location
Figure 2: Test Boring Location
Figure 3: Test Boring Illustration
Test Boring Log
Boring Log Notes
Rock Description Terminology
Core Sample Photographs
Table A-1: Results of Piezometer Monitoring



AMERICAN ENGINEERING TESTING, INC.	PROJECT:	Pier 5; 3rd Avenue Bridge Minneapolis, Minnesota		REPORT NO. 01-05995
	SUBJECT	Site Location		DATE 4/8/14
	SCALE None	DRAWN BY JCR		**Figure 1**



AMERICAN ENGINEERING TESTING, INC.

PROJECT: Pier 5; 3rd Avenue Bridge
Minneapolis, Minnesota

SUBJECT Test Boring Location

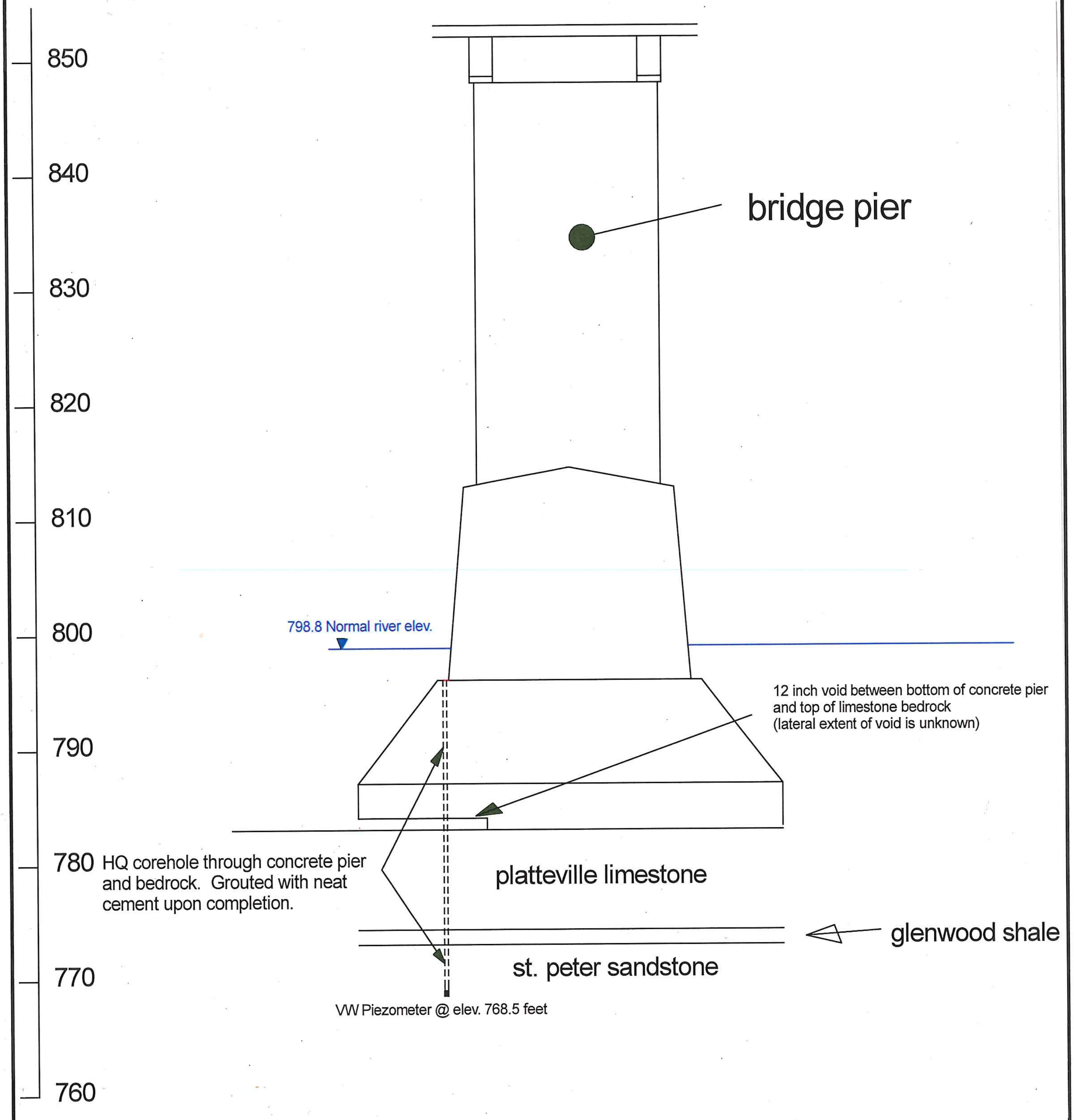
SCALE As Shown

DRAWN BY JCR

REPORT NO. 01-05995

DATE 4/8/14

Figure 2



AMERICAN ENGINEERING TESTING, INC.	**PROJECT:** **Pier 5; 3rd Avenue Bridge Minneapolis, Minnesota**		**AET REPORT NO.** **01-05995**
	SUBJECT: **Test Boring Illustration**		**DATE** **4/8/14**
	SCALE **None**	**DRAWN BY** **JCR**	**Figure 3**

AMERICAN
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SUBSURFACE BORING LOG

AET JOB NO: **01-05995**

LOG OF BORING NO. **B-1 (p. 1 of 3)**

PROJECT: **3rd Avenue Bridge; Minneapolis, MN**

DEPTH IN FEET	SURFACE ELEVATION: 853.0 MATERIAL DESCRIPTION		GEOLOGY	N	MC	SAMPLE TYPE	REC IN.	FIELD & LABORATORY TESTS				
								WC	REC %	RQD IN.	RQD %	%-#200
1	0-57.1' Set HW casing between bridge deck and ledge on bridge pier											
2												
3												
4												
5												
6												
7												
8												
9												
10												
11												
12												
13												
14												
15												
16												
17												
18												
19												
20												
21												
22												
23												
24												
25												
26												
27												
28												
29												
30												
31												

DEPTH:	DRILLING METHOD
0-57.1'	**Set HW Casing**
57.1-80'	**HQ Coring**
80-85	**RD w/DM**

BORING COMPLETED: **4/10/14**

DR: **SS** LG: **TK** Rig: **41**

WATER LEVEL MEASUREMENTS

DATE	TIME	SAMPLED DEPTH	CASING DEPTH	CAVE-IN DEPTH	DRILLING FLUID LEVEL	WATER LEVEL

NOTE: REFER TO THE ATTACHED SHEETS FOR AN EXPLANATION OF TERMINOLOGY ON THIS LOG

SUBSURFACE BORING LOG

AET JOB NO: **01-05995** LOG OF BORING NO. **B-1 (p. 2 of 3)**

PROJECT: **3rd Avenue Bridge; Minneapolis, MN**

DEPTH IN FEET	MATERIAL DESCRIPTION	GEOLOGY	N	MC	SAMPLE TYPE	REC IN.	FIELD & LABORATORY TESTS				
							WC	REC %	RQD IN.	RQD %	%-#200
33	0-57.1' Set HW casing between bridge deck and ledge on bridge pier *(continued)*										
34											
35											
36											
37											
38											
39											
40											
41											
42											
43											
44											
45											
46											
47											
48											
49											
50											
51											
52											
53											
54											
55											
56											
57	CONCRETE, horizontal cracks/weathering around 59.2', 59.4', 62.5', 63.6', 63.8', 64.2', 64.3', 64.4', 64.5', 64.6', 67.2', 67.3', 68.2', 68.4'	FILL									
58					HQ	35		101			
59											
60											
61											
62					HQ	60		100			
63											
64											
65											
66											
67					HQ	48		100			
68											
69	VOID										

AET_CORP 01-05995.GPJ AET+CPT+WELL.GDT 4/9/14

SUBSURFACE BORING LOG

AET JOB NO: **01-05995** LOG OF BORING NO. **B-1 (p. 3 of 3)**

PROJECT: **3rd Avenue Bridge; Minneapolis, MN**

DEPTH IN FEET	MATERIAL DESCRIPTION	GEOLOGY	N	MC	SAMPLE TYPE	REC IN.	FIELD & LABORATORY TESTS WC	REC %	RQD IN.	RQD %	%-#200
71 72 73 74 75 76 77	LIMESTONE, light gray and gray, crinkley bedded Weathering: Slightly weathered to fresh Fracturing: Slightly fractured Stratification: Very thinly bedded Hardness: Hard	PLATTEVILLE FORMATION MIFFLIN MEMBER			HQ	60		100	50	83	
					HQ	57		95	43.5	72	
78 79	Weathering: Fresh Fracturing: Slightly fractured Stratification: Thinly bedded Hardness: Hard	PLATTEVILLE FORMATION PECATONICA MEMBER									
80	SHALE, gray	GLENWOOD FORMATION			SS	12					
81 82 83 84	SANDSTONE, light gray, fine grained	ST. PETER FORMATION									
85	**END OF BORING** Set VW piezometer at 84.5 feet (elevation 768.5 feet)										

AET_CORP 01-05995.GPJ AET+CPT+WELL.GDT 4/9/14

BORING LOG NOTES

DRILLING AND SAMPLING SYMBOLS

Symbol	Definition
AR:	Sample of material obtained from cuttings blown out the top of the borehole during air rotary procedure.
B, H, N:	Size of flush-joint casing
CAS:	Pipe casing, number indicates nominal diameter in inches
COT:	Clean-out tube
DC:	Drive casing; number indicates diameter in inches
DM:	Drilling mud or bentonite slurry
DR:	Driller (initials)
DS:	Disturbed sample from auger flights
DP:	Direct push drilling; a 2.125 inch OD outer casing with an inner 1½ inch ID plastic tube is driven continuously into the ground.
FA:	Flight auger; number indicates outside diameter in inches
HA:	Hand auger; number indicates outside diameter
HSA:	Hollow stem auger; number indicates inside diameter in inches
LG:	Field logger (initials)
MC:	Column used to describe moisture condition of samples and for the ground water level symbols
N (BPF):	Standard penetration resistance (N-value) in blows per foot (see notes)
NQ:	NQ wireline core barrel
PQ:	PQ wireline core barrel
RDA:	Rotary drilling with compressed air and roller or drag bit.
RDF:	Rotary drilling with drilling fluid and roller or drag bit
REC:	In split-spoon (see notes), direct push and thin-walled tube sampling, the recovered length (in inches) of sample. In rock coring, the length of core recovered (expressed as percent of the total core run). Zero indicates no sample recovered.
SS:	Standard split-spoon sampler (steel; 1.5" is inside diameter; 2" outside diameter); unless indicated otherwise
SU	Spin-up sample from hollow stem auger
TW:	Thin-walled tube; number indicates inside diameter in inches
WASH:	Sample of material obtained by screening returning rotary drilling fluid or by which has collected inside the borehole after "falling" through drilling fluid
WH:	Sampler advanced by static weight of drill rod and hammer
WR:	Sampler advanced by static weight of drill rod
94mm:	94 millimeter wireline core barrel
▼:	Water level directly measured in boring
▽:	Estimated water level based solely on sample appearance

TEST SYMBOLS

Symbol	Definition
CONS:	One-dimensional consolidation test
DEN:	Dry density, pcf
DST:	Direct shear test
E:	Pressuremeter Modulus, tsf
HYD:	Hydrometer analysis
LL:	Liquid Limit, %
LP:	Pressuremeter Limit Pressure, tsf
OC:	Organic Content, %
PERM:	Coefficient of permeability (K) test; F - Field; L - Laboratory
PL:	Plastic Limit, %
q_p:	Pocket Penetrometer strength, tsf (approximate)
q_c:	Static cone bearing pressure, tsf
q_u:	Unconfined compressive strength, psf
R:	Electrical Resistivity, ohm-cms
RQD:	Rock Quality Designation of Rock Core, in percent (aggregate length of core pieces 4" or more in length as a percent of total core run)
SA:	Sieve analysis
TRX:	Triaxial compression test
VSR:	Vane shear strength, remolded (field), psf
VSU:	Vane shear strength, undisturbed (field), psf
WC:	Water content, as percent of dry weight
%-200:	Percent of material finer than #200 sieve

STANDARD PENETRATION TEST NOTES
(Calibrated Hammer Weight)

The standard penetration test consists of driving a split-spoon sampler with a drop hammer (calibrated weight varies to provide N_{60} values) and counting the number of blows applied in each of three 6" increments of penetration. If the sampler is driven less than 18" (usually in highly resistant material), permitted in ASTM: D1586, the blows for each complete 6" increment and for each partial increment is on the boring log. For partial increments, the number of blows is shown to the nearest 0.1' below the slash.

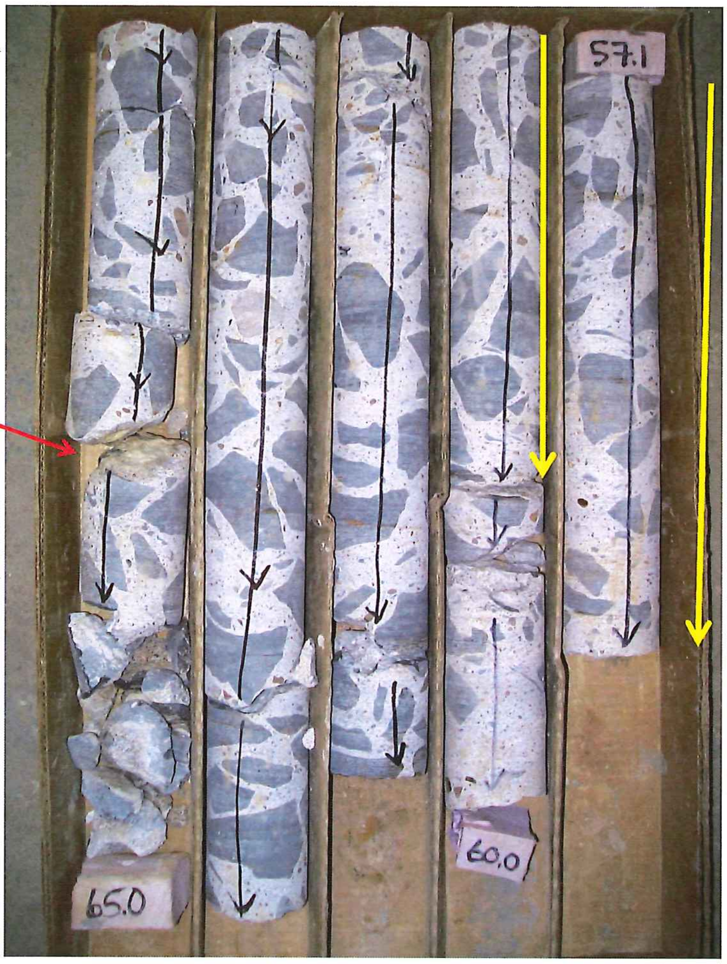
The length of sample recovered, as shown on the "REC" column, may be greater than the distance indicated in the N column. The disparity is because the N-value is recorded below the initial 6" set (unless partial penetration defined in ASTM: D1586 is encountered) whereas the length of sample recovered is for the entire sampler drive (which may even extend more than 18").

ROCK DESCRIPTION TERMINOLOGY

Rock Property	Descriptive Term	Visual or Physical Properties
Weathering	Highly Weathered	Almost complete rock disintegration and decomposition. Soil-like texture with some small inclusions of hard rock.
	Very Weathered	Abundant fractures coated with oxides, carbonates, sulfates, mud, etc., thorough discoloration, rock disintegration, and mineral decomposition.
	Moderately Weathered	Some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition
	Slightly Weathered	A few stained fractures, slight discoloration, little to no effect on cementation, no mineral decomposition.
	Fresh	Unaffected by weathering agents, no appreciable change with depth.
Fracturing	Intensely Fractured	Less than 1" spacing
	Very Fractured	1" to 6" spacing
	Moderately Fractured	6" to 12" spacing
	Slightly Fractured	12" to 36" spacing
	Solid	36" spacing or greater
Stratification	Thinly Laminated	Less than 1/10"
	Laminated	1/10" to ½"
	Very Thinly Bedded	½" to 2"
	Thinly Bedded	2" to 2'
	Thickly Bedded	More than 2'
Hardness	Soft	Can be dug by hand and crushed by fingers.
	Moderately Hard	Friable, can be gouged deeply with knife and will crumble readily under light hammer blows.
	Hard	Knife scratch leaves dust trace, will withstand a few hammer blows before breaking.
	Very Hard	Scratched with knife with difficulty, difficult to break with hammer blows.
RQD*	Very Poor	0 - 25 (%)
	Poor	25 - 50 (%)
	Fair	50 - 75 (%)
	Good	75 - 90 (%)
	Excellent	90 - 100 (%)

**Rock Quality Designation: Percent of core run consisting of the summation of hard, sound and unfractured rock core segments 4″ or greater in length.*

Core run 57.1 ft. to 65 ft.



Piece of sandstone

100% coring water return from 57.1 ft. to 59.2 ft.

0% coring water Return below 59.2 ft.

Core run 65ft. to 70 ft.

Void from 69 ft. to 70 ft.

Table A-1: Summary of Piezometer Readings

Linear

$P=G(R1-R0)+K(T1-T0)-(S1-S0)$

SN: 1405257 Date_time	Notes	Reading R1	R0	Temperature (degree C) T1	T0	Linear Gage Factor (psi/digit) G	Thermal Factor (psi/degree C) K	Pressure (psi) P
032014_1200	Pre Soaked	8756.9	8756	11.8	23	-0.01692	-0.01248	0.12
032014_1245	Pre Grouted	6568.8	8756.9	9.9	11.8	-0.01692	-0.01248	37.05
032014_1250	Pre Grouted	6575.0	8756.9	9.7	11.8	-0.01692	-0.01248	36.94
032014_1300	Pre Grouted	6583.1	8756.9	9.5	11.8	-0.01692	-0.01248	36.81
032014_1315	Post Grouted	6328.6	8756.9	11.4	11.8	-0.01692	-0.01248	41.09
032114_1400		8797.3	8756.9	9.5	11.8	-0.01692	-0.01248	-0.65
032514_1130		8788.9	8756.9	8.8	11.8	-0.01692	-0.01248	-0.50

Appendix B

Report of Petrographic Analysis

AMERICAN
ENGINEERING
TESTING, INC.

CONSULTANTS
▫ ENVIRONMENTAL
▫ GEOTECHNICAL
▫ MATERIALS
▫ FORENSICS

REPORT OF CONCRETE ANALYSIS

PROJECT:	**REPORTED TO:**
3RD AVENUE BRIDGE REPAIRS	HDR ENGINEERING, INC. 701 XENIA AVE S SUITE 600 MINNEAPOLIS, MN 55416
	ATTN: JACOB BRONDER
AET PROJECT NO: 01-05995	**DATE:** MARCH 21, 2014

INTRODUCTION

This report presents the results of laboratory work performed by our firm on one concrete core sample taken by representatives of AET at the 3rd Avenue Bridge in Minneapolis on February 19, 2014. We understand the concrete core was obtained from one of the bridge piers and was taken in a vertical orientation. The scope of our work was limited to performing petrographic analysis on the sample to document the general overall quality of the concrete.

CONCLUSIONS

Based on our observations and testing, we believe:

1. The concrete was in good condition. However, the concrete was not air entrained and contained several secondary crystalline deposits (ettringite) suggesting water movement through the concrete. The concrete appeared to be of considerable age, due primarily to the large nominal size of the coarse aggregate (up to 3") and the relatively coarse relict portland cement clinker particles. An apparent cold joint was present at approximately 80 mm (3-1/8") depth from the top surface. Segregation of coarse aggregate was observed up to 25 mm (1") on either side of the cold joint. The two concretes appeared to be well bonded but a thin layer of laitance was present along much of the cold joint.

2. In general, the coarse and fine aggregate was hard and durable. However, a few alkali-silica reactive (ASR) quartzite gravel particles were observed. ASR gel was observed partially filling a few air voids proximate to the particles. The ASR appeared to be innocuous and no extensive cracking or bulk expansion was observed. In addition, several shale fine aggregate particles exhibited internal microcracking and a few exhibiting microcracking propagating into the paste.

3. White, acicular ettringite was observed partially filling to filling several air voids throughout the paste. The secondary ettringite is innocuous and is consistent with water movement through the concrete.

SAMPLE IDENTIFICATION

Sample ID:	B1 (60.0’-61.0’)
Sample Type:	Hardened Concrete Core
Original Sample Dimensions:	63 mm (2-1/2") diameter 350 x mm (13-3/4") long

TEST RESULTS

Our complete petrographic analysis documentation appears on the attached sheet entitled 24-LAB-001 "Petrographic Examination of Hardened Concrete, ASTM C856." A brief summary of the general physical characteristics of the concrete is as follows:

1. The coarse aggregate was comprised of 76 mm (3") maximum sized crushed trap rock comprised of diabase or ophitic basalt. The fine aggregate was a natural glacial sand with some gravel particles up to 12 mm (1/2") in dimension.
2. The paste color was mottled very light gray to light gray. The paste was moderately hard (Mohs’ ≈ 3.5) with the paste/aggregate bond considered fair to good.
3. The top and bottom surfaces of the core were fractured. The depth of carbonation was negligible at both fractured surfaces.
4. The w/cm was estimated to be between 0.45 and 0.55 with approximately 4 to 6% residual portland cement clinker particles. No supplementary cementitious materials were observed in the concrete sample.

Air Content Testing

Sample ID	B1
Total Air Content (%)	0.2
"Entrained" Air (%) voids < 1mm (0.040")	0.1
"Entrapped" Air (%) voids > 1mm (0.040")	0.1
Spacing Factor, in.	0.044

TEST PROCEDURES

Laboratory testing was performed on March 18, 2014 and subsequent dates. Our procedures were as follows:

Petrographic Analysis

A petrographic analysis was performed in accordance with AET Standard Operating Procedure 24- LAB-001, "Petrographic Examination of Hardened Concrete," ASTM C856-latest revision. The petrographic analysis consisted of reviewing the cement paste and aggregate qualities on a whole basis on saw cut and lapped, and fractured sections. Reflected light microscopy was performed under an Olympus SZX-12 binocular stereozoom microscope at magnifications up to 160x. The depth of carbonation was documented using a phenolphthalein pH indicator solution applied on freshly saw cut and lapped surfaces of the concrete sample. The paste-coarse aggregate bond quality was determined by fracturing a sound section of the concrete in the laboratory with a rock hammer.

The water/cementitious of the concrete was estimated by viewing a thin section of the concrete under a Nikon E600 polarizing light microscope at magnifications of up to 600x. Thin section analysis was performed in accordance with Standard Operating Procedure 24-LAB-009, "Determining the Water/Cement of Portland Cement Concrete, AET Method." An additional, smaller, saw cut subdivision of the concrete sample is epoxy impregnated, highly polished, and then attached to a glass slide using an optically clear epoxy. Excess sample is saw cut from the glass and the thin slice remaining on the slide is lapped and polished until the concrete reaches 25 microns or less in thickness. Thin section analysis allows for the observation of portland cement morphology, including: phase identification, an estimate of the amount of residual material, and spatial relationships. Also, the presence and relative amounts of supplementary cementitious materials and pozzolans may be identified and estimated.

Air Content Testing

Air content testing was performed using Standard Operating Procedure 24-LAB-003, "Microscopical Determination of Air Void Content and Parameters of the Air Void System in Hardened Concrete, ASTM C457-latest revision." The linear traverse method was used. The concrete core was saw cut perpendicular with respect to the horizontal plane of the concrete as placed and then lapped prior to testing.

REMARKS

The test sample will be retained for a period of at least sixty days from the date of this report. Unless further instructions are received by that time, the sample may be discarded. Test results relate only to the items tested. No warranty, express or implied, is made.

Report Prepared By: American Engineering Testing, Inc.	Report Reviewed By: American Engineering Testing, Inc.
Blake Lemcke, PG Geologist/Petrographer MN License #50337 blemcke@amengtest.com	Gerard F. Moulzolf, PG Vice President/Principal Petrographer MN License #30023 gmoulzolf@amengtest.com

24-LAB-001 Petrographic Examination of Hardened Concrete
ASTM C856

Project No.	01-05995	Date:	March 18, 2014
Sample ID:	B1	Performed by:	B. Lemcke, G. Moulzolf

I. General Observations

1. Sample Dimensions: Our analysis was performed on two lapped profiles of a 332 mm (13-1/16") x 61 mm (2-3/8") x 24 mm (15/16"), a 345 mm (13-5/8") x 62 mm (2-7/16") x 32 mm (1-1/4") thick lapped sections and a 76mm (3") x 52mm (2") thin section that were sawcut and prepared from the original 63 mm (2-1/2") diameter x 350 mm (13-3/4") long core.

2. Surface Conditions:
   Top (60.0'): Rough, irregular, fractured surface
   Bottom (61.0'): Rough, irregular, fractured surface

3. Reinforcement: None observed.

4. General Physical Conditions: The concrete was well consolidated. A cold joint was observed at approximately 80 mm (3-1/8") depth from the top fractured surface. The cold joint was sub-horizontal and defined by a slight change in paste color and a fine horizontal 'ribbon' of white-colored laitance. A segregation of coarse aggregate was observed on approximately 25 mm of either side of the cold joint. A few fine fragments of cellulose/wood material were observed scattered throughout the paste. A sub-horizontal microcrack was observed at 10 mm depth from the top fractured surface. The microcrack proceeded from the cored edge of the sample through approximately one quarter of the core's diameter. It is likely a result of the coring of the sample. A few other random microcracks were observed at various depths and orientations throughout the paste. Several fine microcracks were observed within and proximate to (alkali-silica reactive?) shale fine aggregate particles. White, acicular ettringite was observed partially filling to filling several air voids throughout the paste. Alkali-silica gel was observed partially filling and filling air voids proximate to two reactive quartzite fine aggregate particles. The residual portland cement clinker was very coarse; a sign of significant age of the concrete.

II. Aggregate

1. Coarse: 76 mm (3") maximum sized crushed trap rock comprised of diabase or ophitic basalt. The crushed material was angular to sub-angular. Several particles of 12 mm (1/2") maximum sized naturally occurring gravel were also observed and were comprised of granite, basalt, greywacke and quartzite. The coarse aggregate appeared well graded and exhibited fair overall distribution. Alkali-silica reactive particles consist of quartzite.

2. Fine: Natural quartz, feldspar, and lithic sand with several carbonate and shale particles. The grains were mostly sub-rounded with many smaller sub-angular particles. The fine aggregate appeared fairly graded and exhibited good overall uniform distribution. Several shale particles exhibited internal microcracking, with some microcracks propagating into the surrounding paste.

III. Cementitious Properties

1. Air Content:	0.2% total
2. Depth of carbonation:	Carbonation was negligible at both ends of the core.
3. Pozzolan presence:	None observed.
4. Paste/aggregate bond:	Fair to good.
5. Paste color:	Mottled very light gray to light gray (Munsell® N8 to N7)
6. Paste hardness:	Moderately hard (Mohs' ≈ 3.5).
7. Microcracking:	A sub-horizontal microcrack was observed at 10 mm depth from the top fractured surface. The microcrack proceeded from the cored edge of the sample through approximately one quarter of the core's diameter. A few microcracks were observed at various depths and orientations throughout the paste. Several microcracks were observed within and proximate to (alkali-silica reactive?) shale fine aggregate particles.
8. Secondary deposits:	White, acicular ettringite was observed partially filling to filling several air voids throughout the paste. Alkali-silica gel was observed partially filling and filling air voids proximate to two reactive quartzite aggregate particles.
9. w/cm:	Estimated at between 0.45 and 0.55 with approximately 4 to 6% residual portland cement clinker particles.
10. Cement hydration:	Alites: Fully. Belites: Fully.

AMERICAN ENGINEERING TESTING, INC.

AIR VOID ANALYSIS

PROJECT:
3^{RD} AVE BRIDGE REPAIRS

REPORTED TO:
HDR ENGINEERING, INC.
701 XENIA AVE S
SUITE 600
MINNEAPOLIS, MN 55416

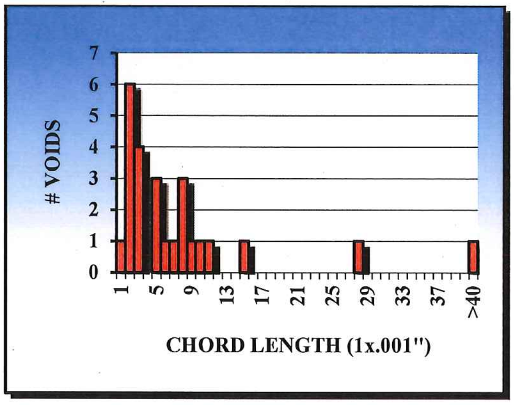
ATTN: JACOB BRONDER

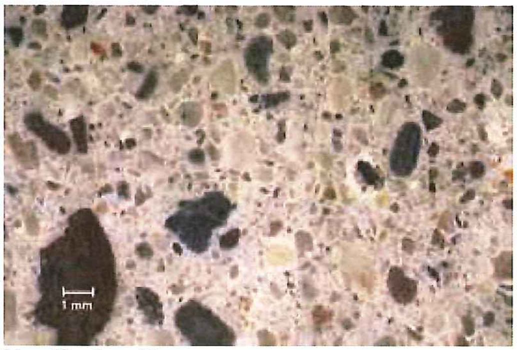
AET PROJECT NO: 01-05995

DATE: MARCH 19, 2014

Sample Number:	B1
Conformance:	The sample contains an air void system which is not consistent with current technology for freeze-thaw resistance.
Sample Data	
Description:	Hardened Concrete Core Section
Dimensions:	63 mm (2-1/2") diameter by 350 mm (13-3/4") long
Test Data:	By ASTM C457*
Air Void Content %	0.2
Entrained, % < 0.040"(1mm)	0.1
Entrapped, %> 0.040"(1mm)	0.1
Air Voids/inch	0.21
Specific Surface, in^2/in^3	420
Spacing Factor, inches	0.044
Paste Content, % estimated	26.0
Magnification	50x
Traverse Length, inches	120
Test Date	3/18/2014

*Sample surface are size did not meet the ASTM C457 minimum requirements.





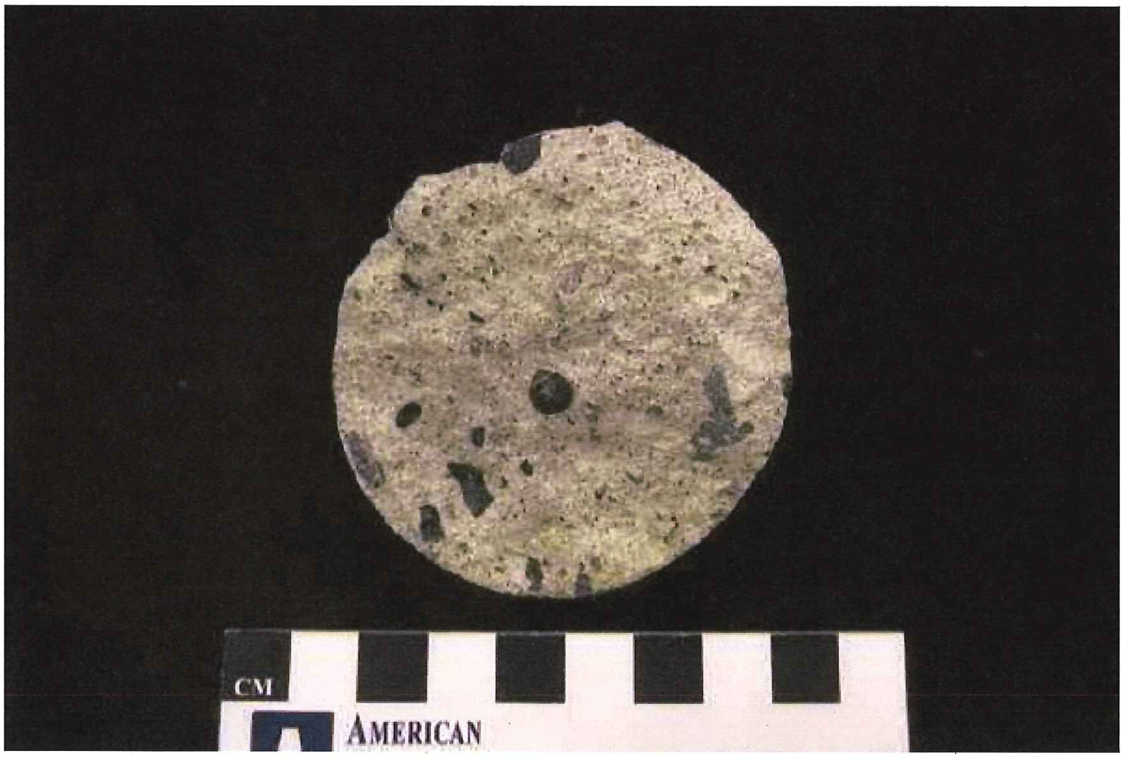
Magnification: 15x
Description: Hardened air void system.

AET PROJECT NO: 01-05995 **DATE:** MARCH 21, 2014
PROJECT: 3RD AVENUE BRIDGE REPAIRS
MINNEAPOLIS, MN

PHOTO: 1

SAMPLE ID: B1 **DESCRIPTION:** Profile view of the sample as received with the top surface to the left.

PHOTO: 2



SAMPLE ID: B1 **DESCRIPTION:** Fractured end surface of core sample.

AET PROJECT NO: 24-00783

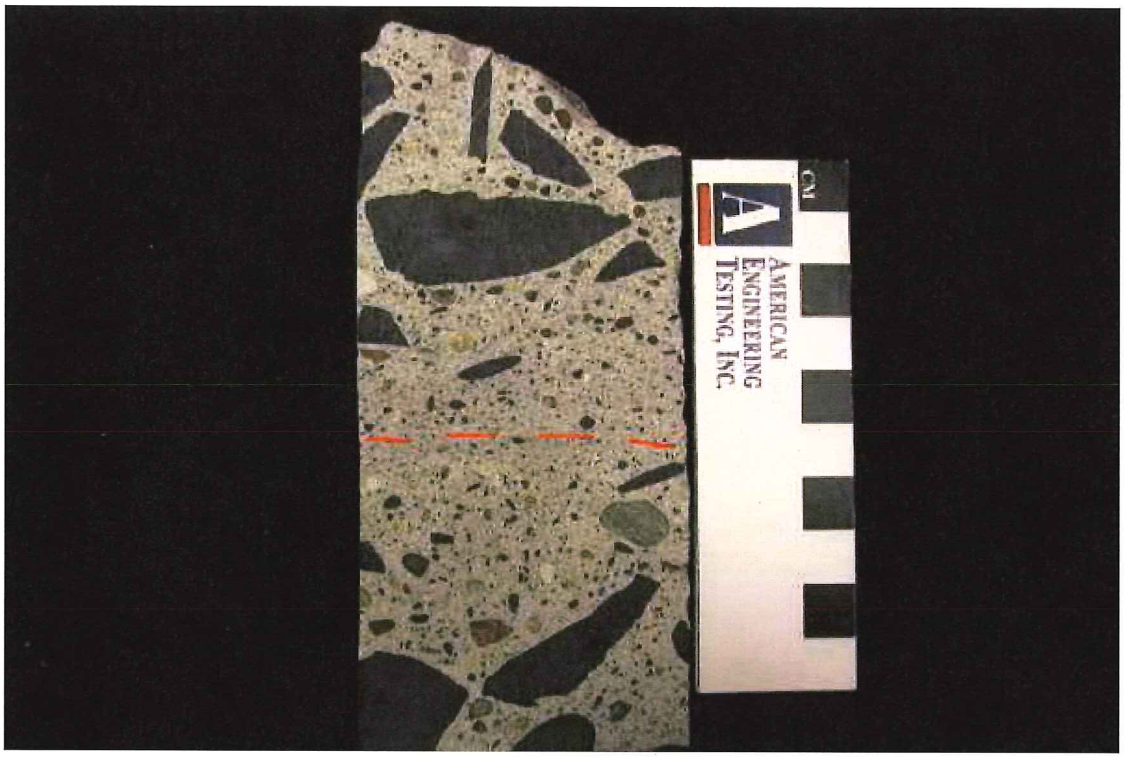
DATE: MARCH 21, 2014

PROJECT: 3RD AVENUE BRIDGE REPAIRS
MINNEAPOLIS, MN

PHOTO: 3

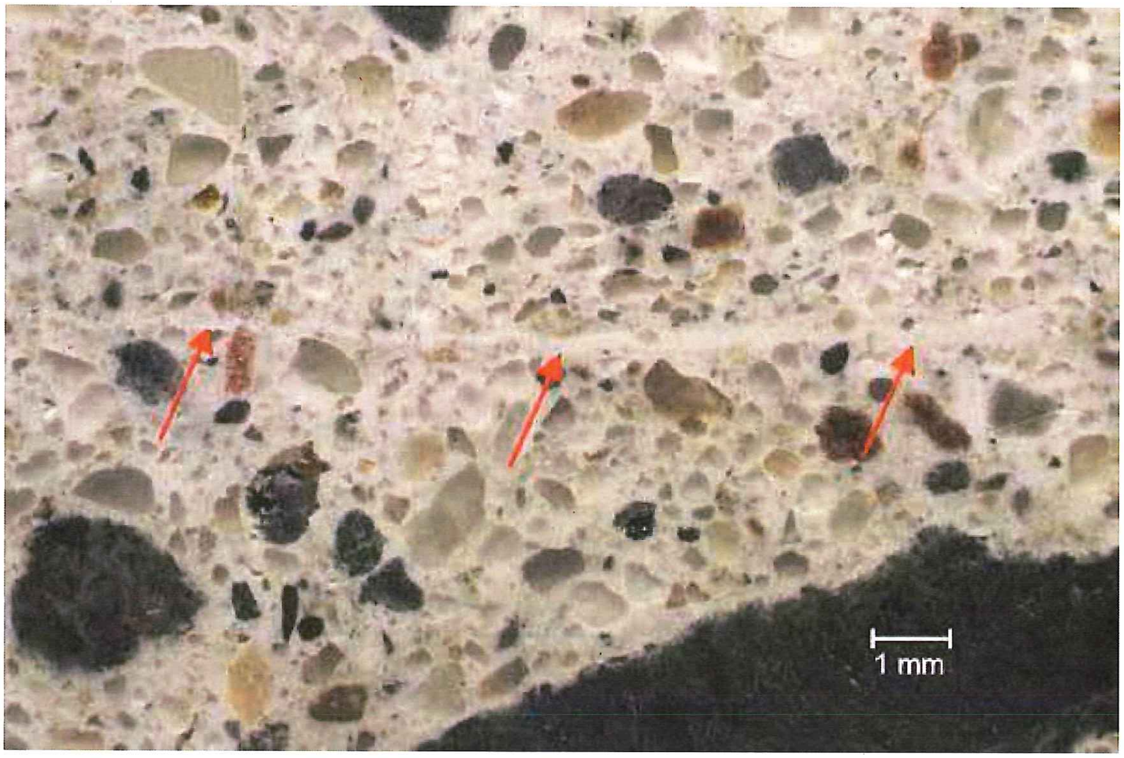
SAMPLE ID: B1 **DESCRIPTION:** Sawcut and lapped cross section of core. Note the relatively large crushed coarse aggregate and a zone of aggregate segregation proximate to the top of the core.

PHOTO: 4



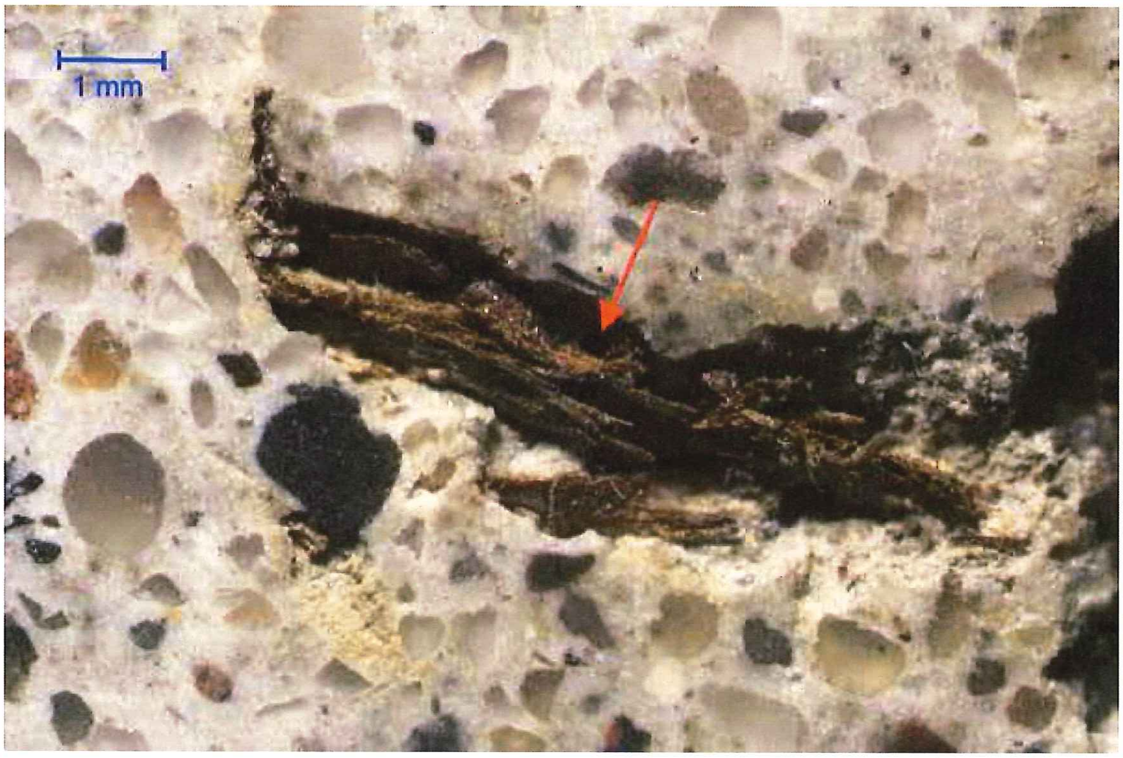
SAMPLE ID: B1 **DESCRIPTION:** Closer view of segregated zone with a cold joint marked with a red dashed line.

PHOTO: 5



SAMPLE ID: B1 **DESCRIPTION:** Magnified view of cold joint showing a white 'ribbon' of laitance (red arrows).
MAG: 15x

PHOTO: 6

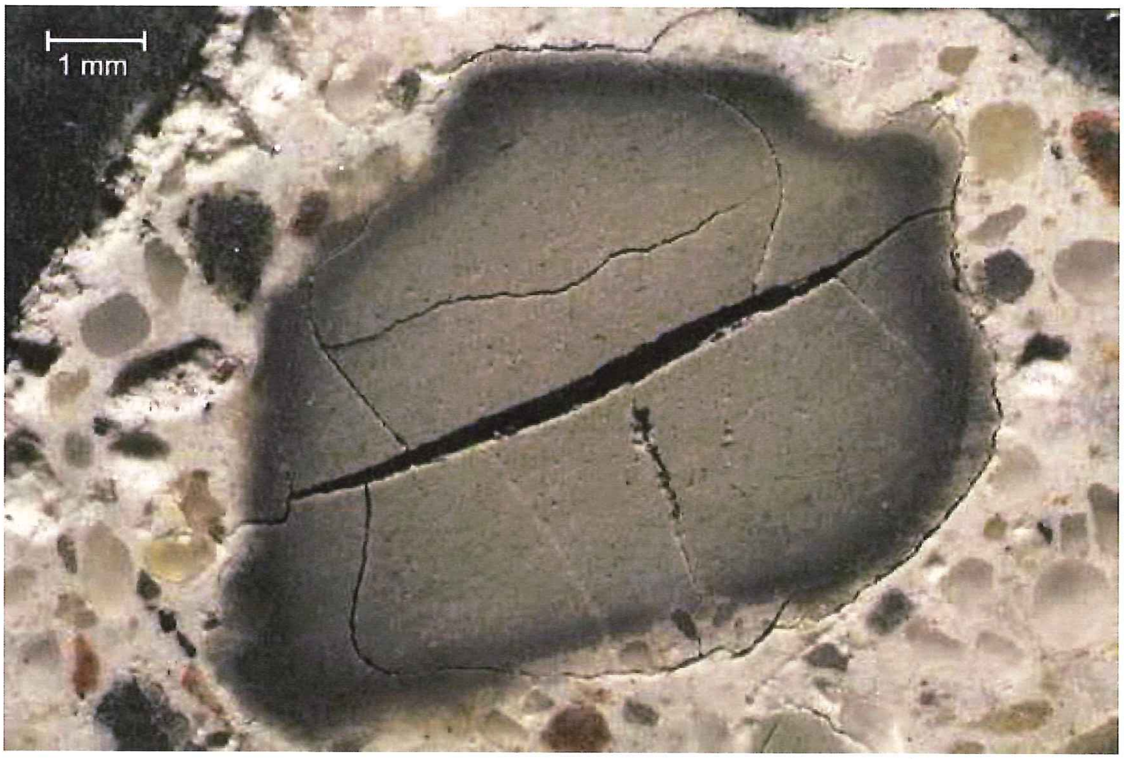


SAMPLE ID: B1 **DESCRIPTION:** Fragment of plant material or wood (red arrow) along the cored edge of the sample.
MAG: 30x

PHOTO: 7

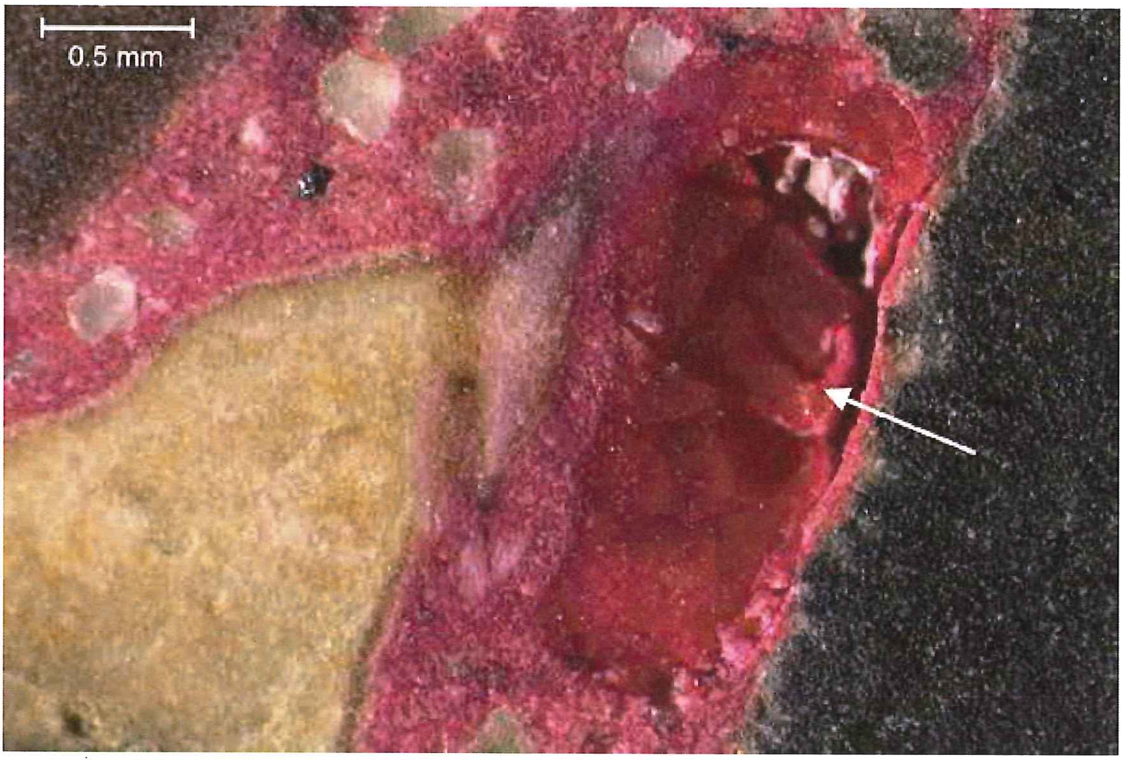
SAMPLE ID: B1 **DESCRIPTION:** White, acicular ettringite nearly filling an air void (red circle) within the paste.
MAG: 50x

PHOTO: 8



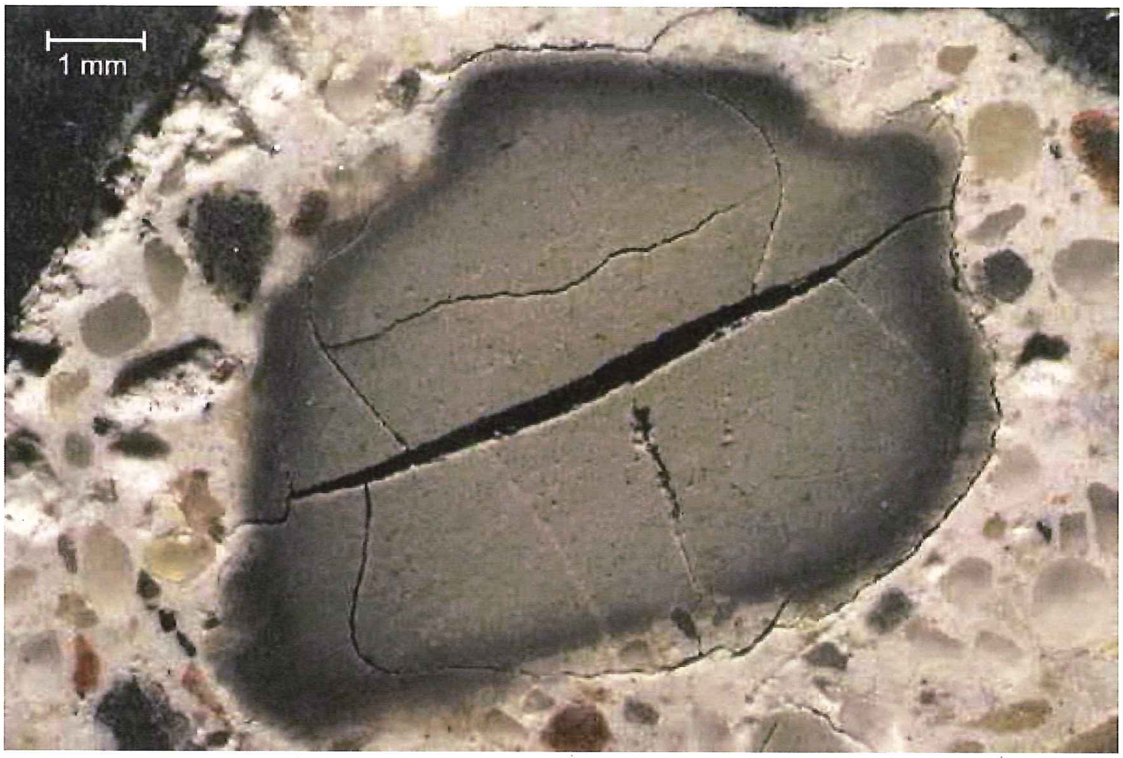
SAMPLE ID: B1 **DESCRIPTION:** A shale fine aggregate particle that exhibits internal cracking and cracking propagating into the paste.
MAG: 30x

PHOTO: 9



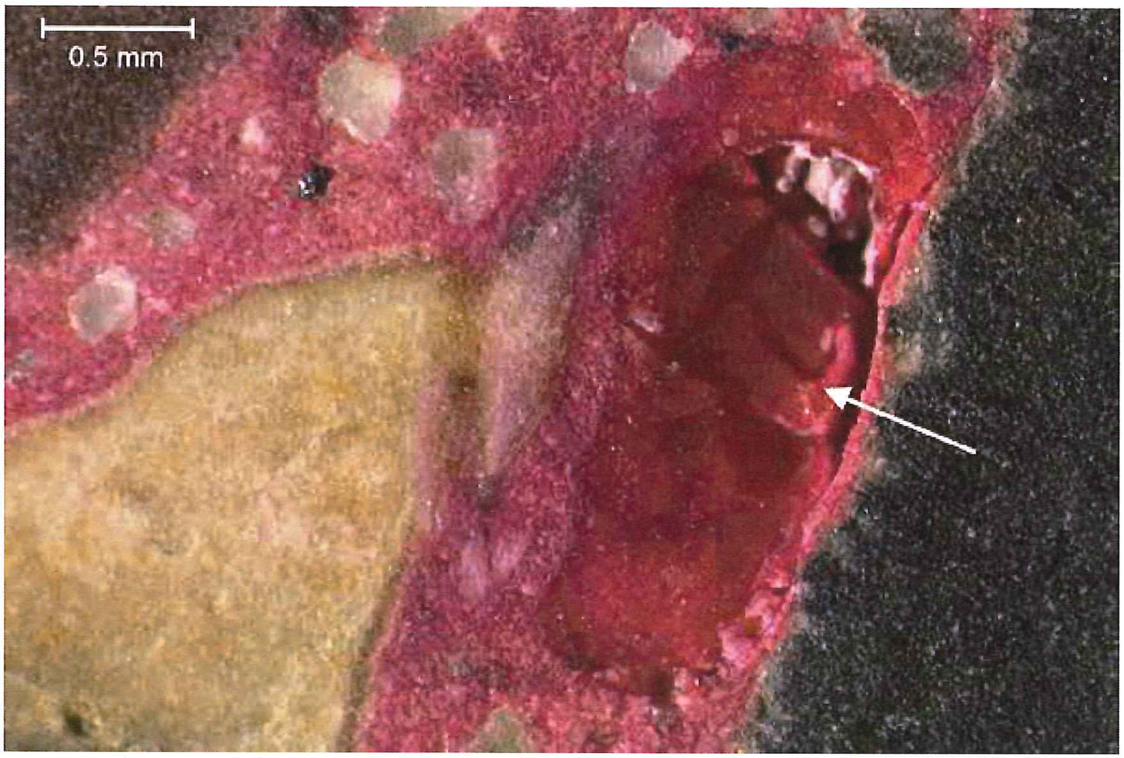
SAMPLE ID: B1 **DESCRIPTION:** Alkali-silica gel partially filling an air void (white arrow) proximate to a reactive quartzite gravel particle.
MAG: 75x

PHOTO: 8



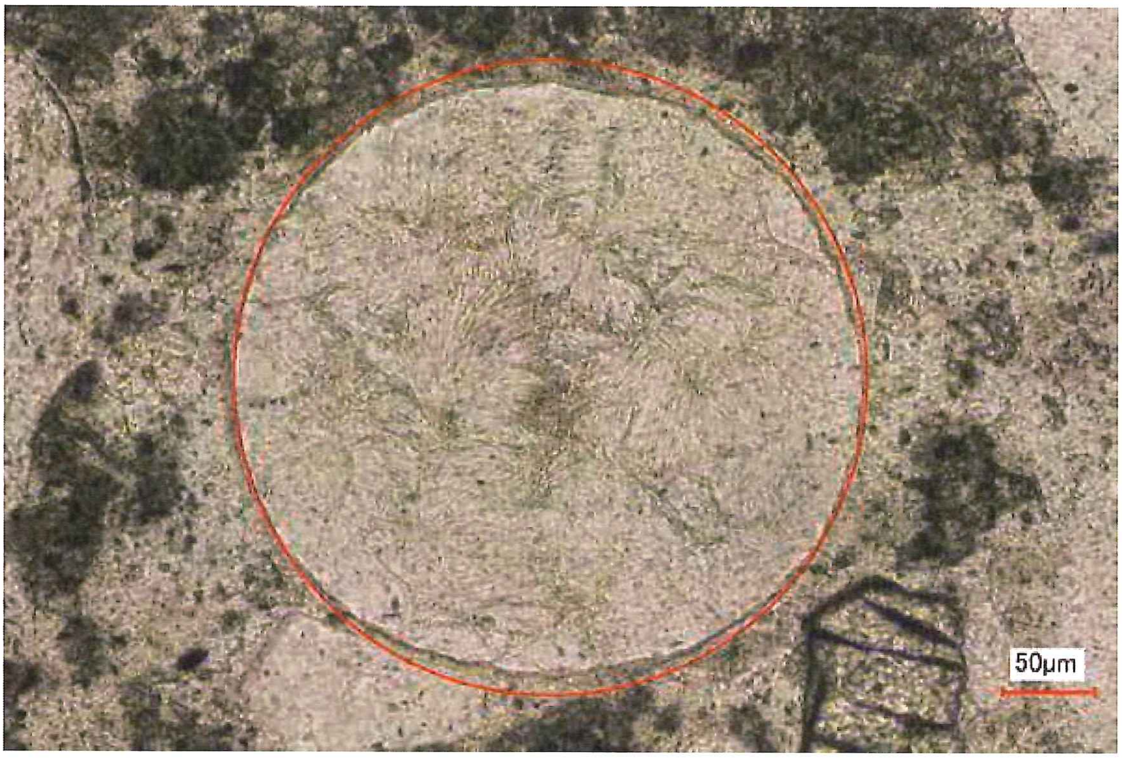
SAMPLE ID: B1
MAG: 30x
DESCRIPTION: A shale fine aggregate particle that exhibits internal cracking and cracking propagating into the paste.

PHOTO: 9



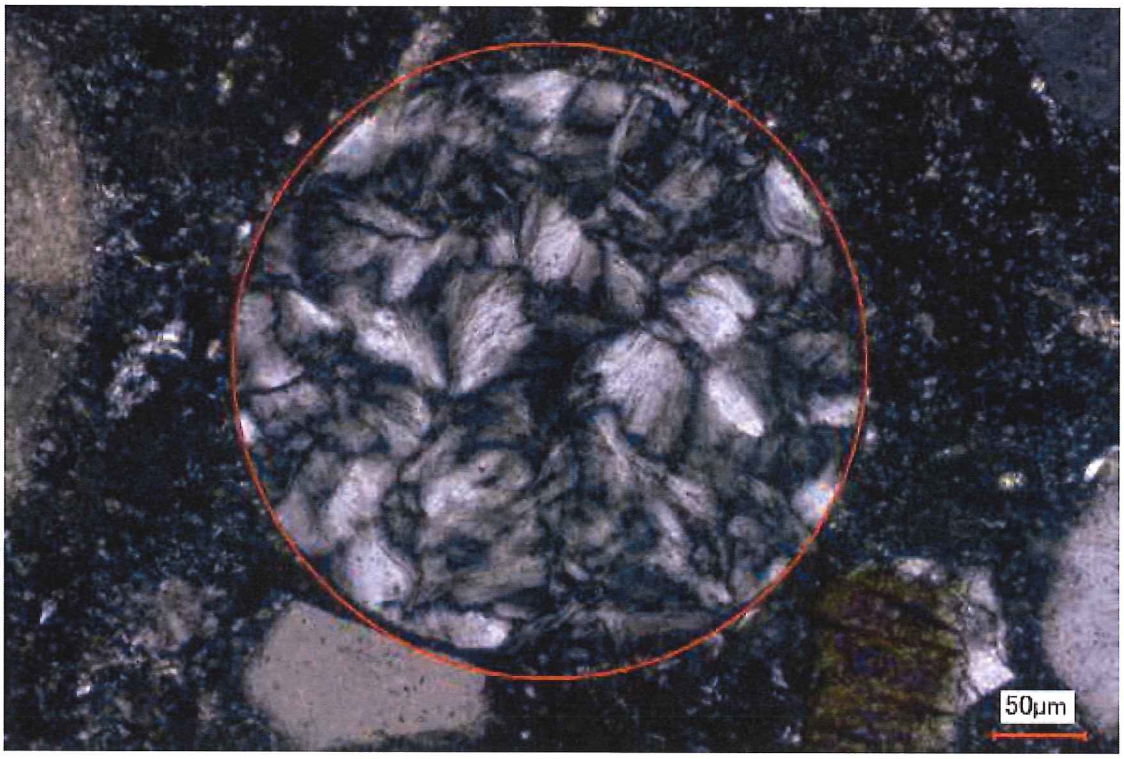
SAMPLE ID: B1
MAG: 75x
DESCRIPTION: Alkali-silica gel partially filling an air void (white arrow) proximate to a reactive quartzite gravel particle.

PHOTO: 10



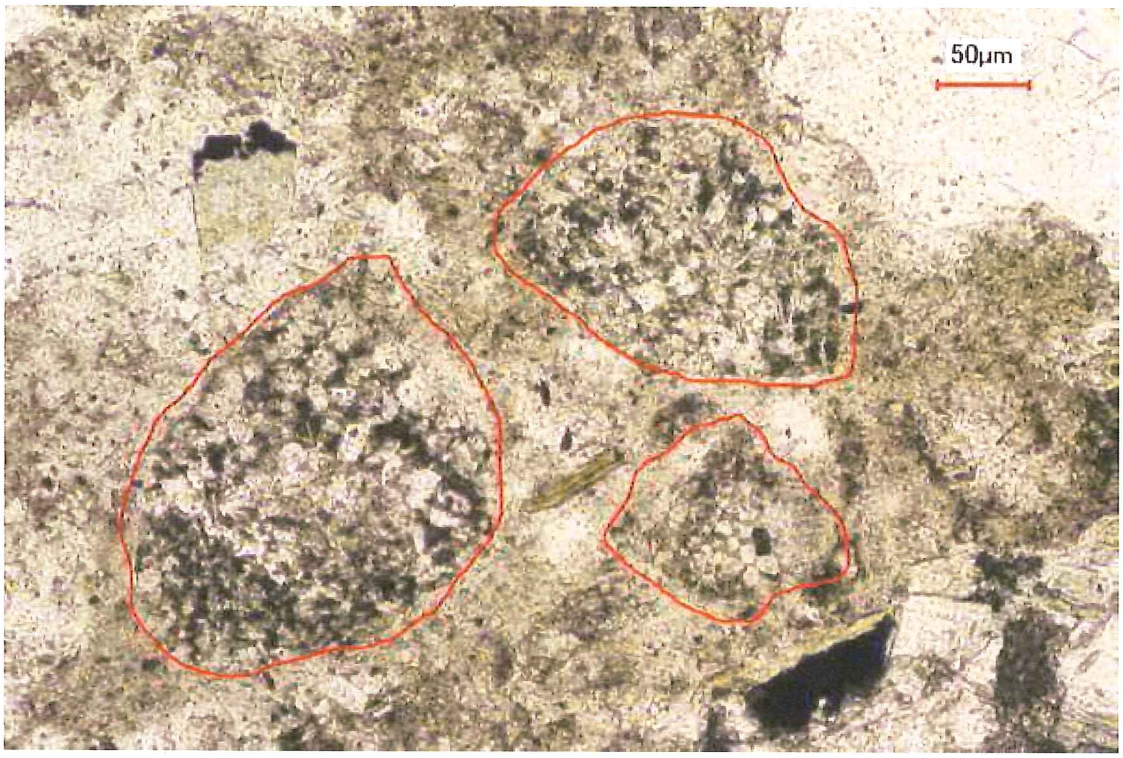
SAMPLE ID: B1
MAG: 200x
DESCRIPTION: Acicular ettringite filling an air void (red circle) in thin section of concrete under transmitted plane polarized light.

PHOTO: 11



SAMPLE ID: B1
MAG: 200x
DESCRIPTION: Same view as above under transmitted cross polarized light.

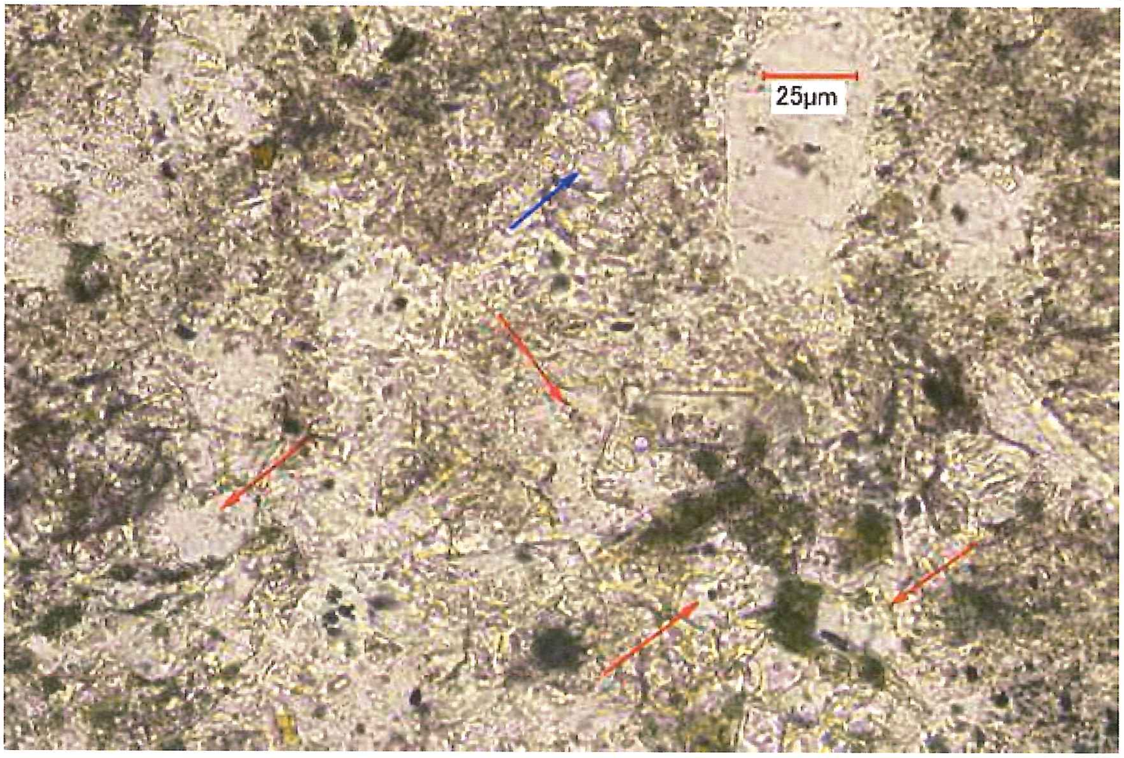
PHOTO: 12



SAMPLE ID: B1
MAG: 200x

DESCRIPTION: Relatively coarse, polycrystalline portland cement clinker particles (red outlines) in thin section of concrete under transmitted plane polarized light. Note these particles are fully hydrated.

PHOTO: 13



SAMPLE ID: B1
MAG: 200x

DESCRIPTION: Fully hydrated alite portland cement clinker particles (red arrows) and a cluster of fully hydrated belite portland cement clinker particles (blue arrow) in thin section of concrete under transmitted plane polarized light.

F

Appendix F

Blue View Screen Shots

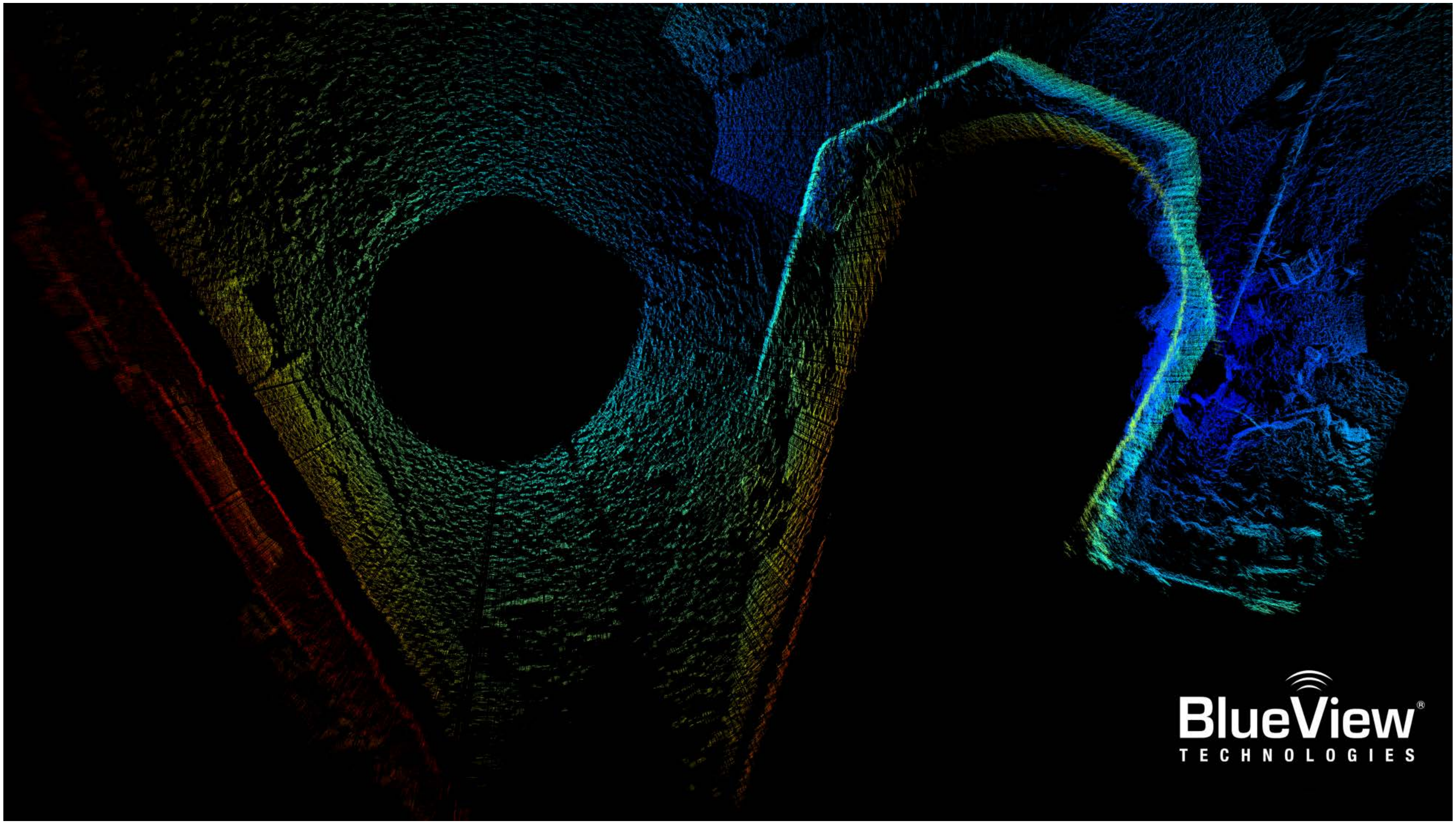


Figure 24: Overall view of Pier 5 footing showing horseshoe dam on the left.

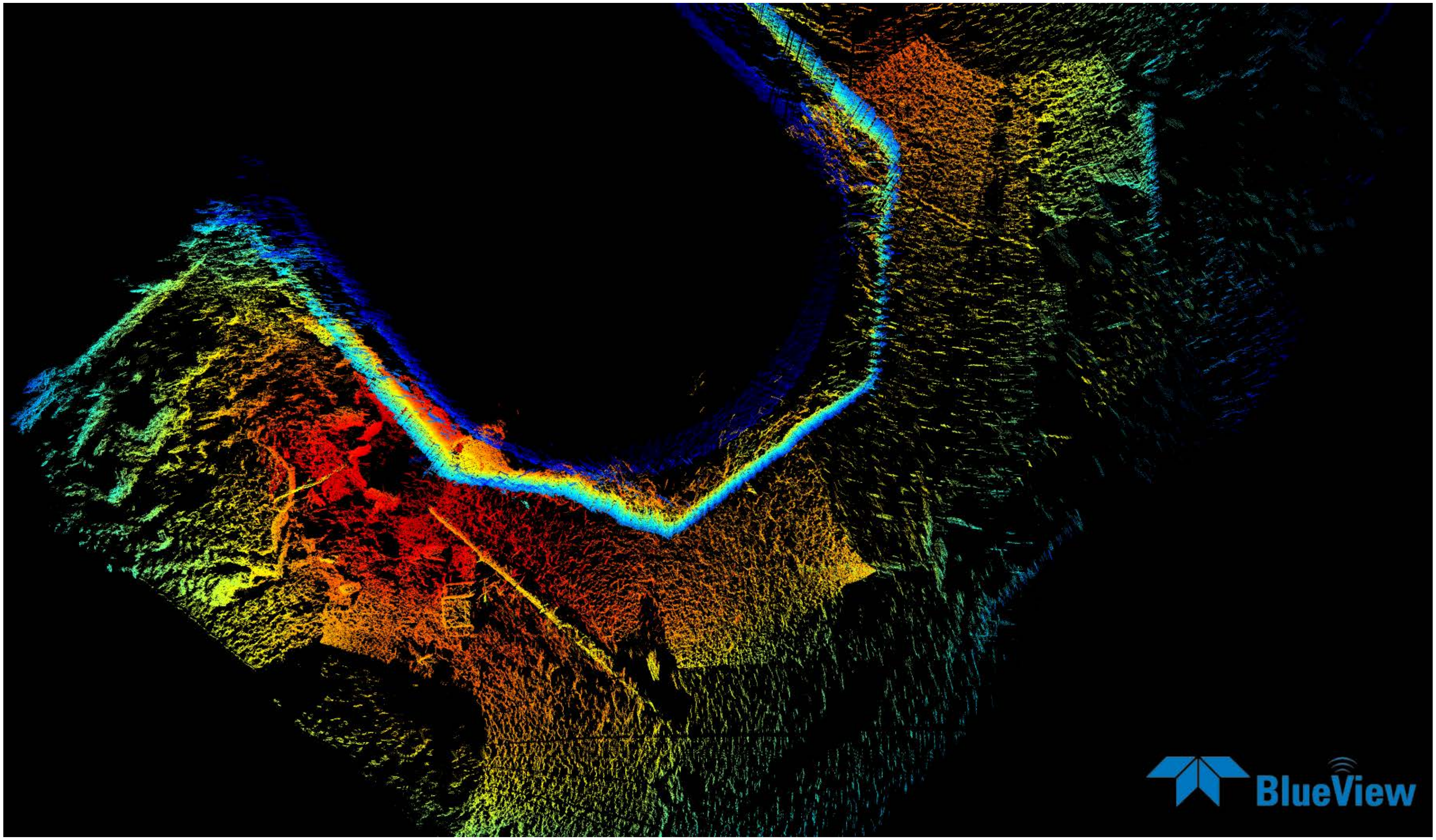


Figure 25: Concrete void in Pier 5.

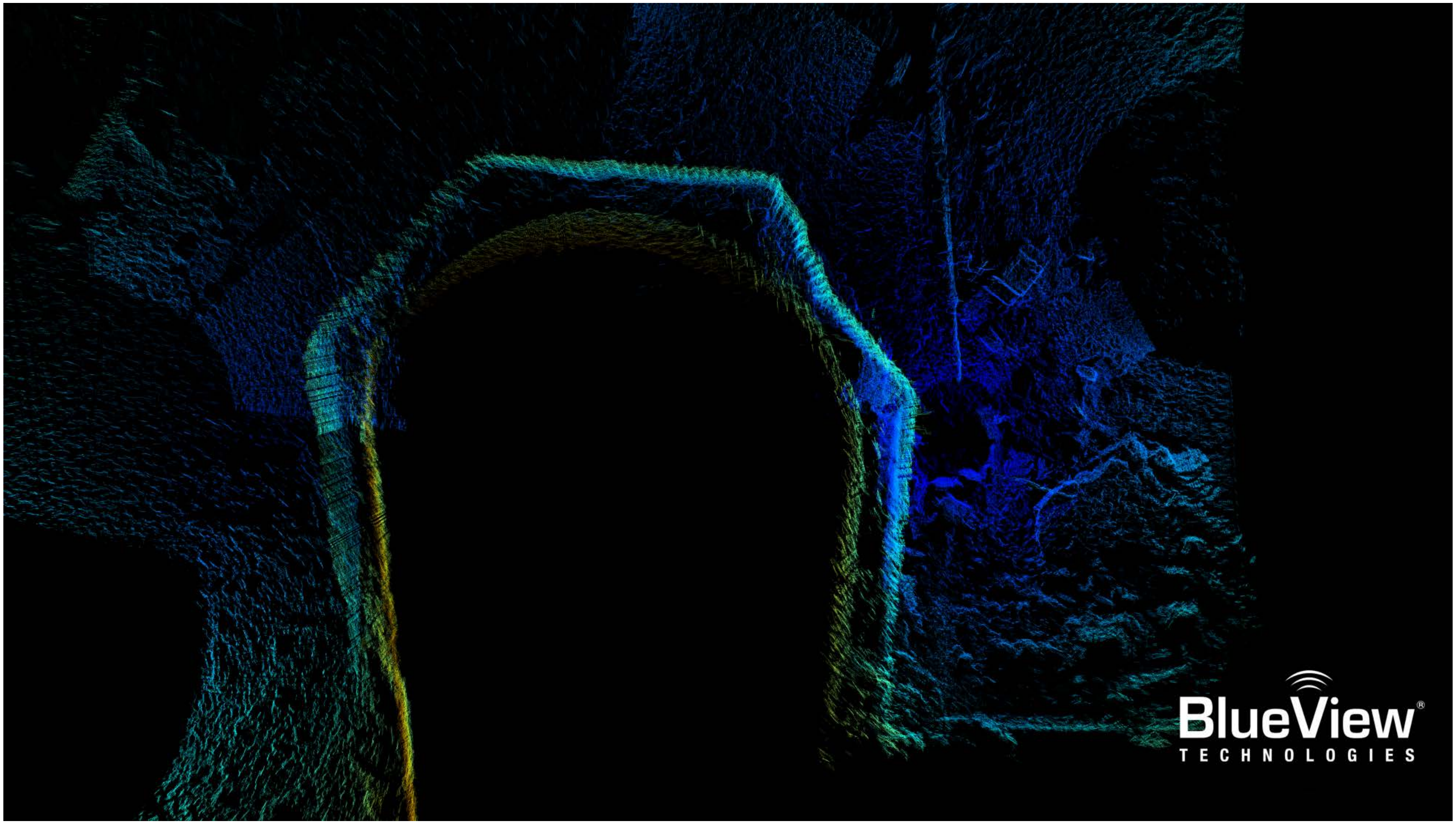


Figure 26: Void in Pier 5 footing.

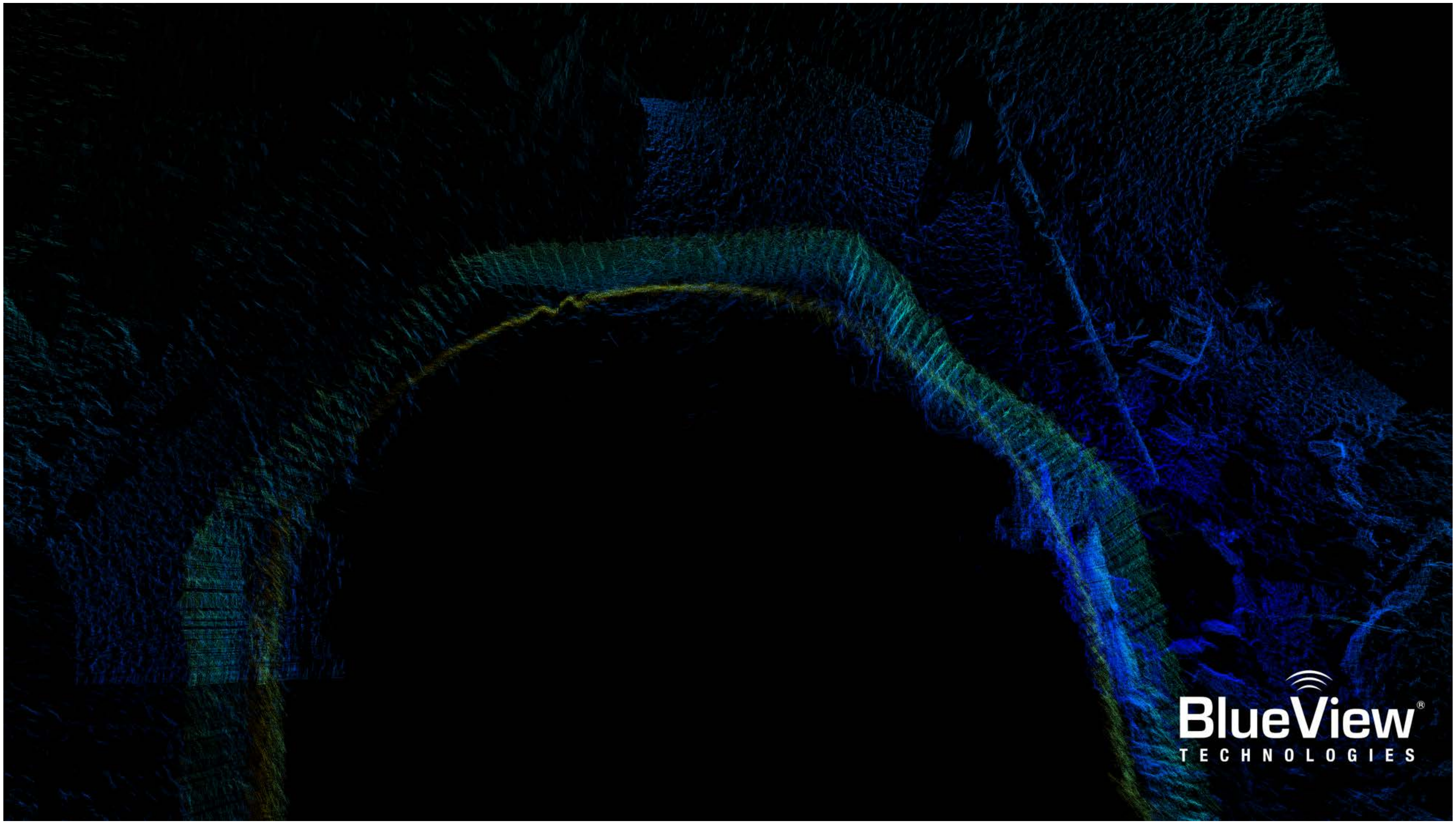


Figure 27: Void opening in Pier 5 footing.