ERDC/GSL SR-23-2

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Old River Control Complex (ORCC) Low Sill: A Literature Synthesis

Benjamin R. Breland, Lucas A. Walshire,
Maureen K. Corcoran, Julie R. Kelley, Janet E. Simms,
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March 2023

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Final report

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Under Project 478533, “MTS ORLSS Collection and Study of Basic Data”

Abstract

The US Army Corps of Engineers (USACE), New Orleans District (MVN), tasked the US Army Engineer and Research Development Center (ERDC) with assessing the condition of a grouted scour hole located at the southeast wall of the Old River Low Sill Structure (ORLSS) at the Old River Control Complex (ORCC) using noninvasive techniques, such as geophysical surveys and physical models. This special report (SR) combines a scientific literature synthesis of previous research with further geologic interpretation as a first step in the overall task assigned by MVN. The results discussed in this SR will be used to inform the interpretation of geophysical surveys, construction of physical models, and input for the slope stability analyses.

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Preface

This study was conducted for the US Army Corps of Engineers, New Orleans District (MVN), under Project 478533, "MTS ORLSS Collection and Study of Basic Data." The technical monitor was Dr. Maureen K. Corcoran.

The work was performed by the Geotechnical Engineering and Geosciences Branch (GEGB) of the Geotechnical and Structures Division (GSD), US Army Engineer Research and Development Center, Geotechnical and Structures Laboratory (ERDC-GSL). At the time of publication, Mr. Christopher G. Price was chief, GSG; Mr. James L. Davis was chief, GS; and Dr. Matthew D. Smith was the technical director for Water Resources Infrastructure. The deputy director of ERDC-GSL was Mr. Charles W. Ertle II, and the director was Mr. Bartley P. Durst.

The authors appreciate the assistance of the US Army Corps of Engineers, New Orleans District, in providing numerous design memorandums and references related to Old River Control Complex. Without its assistance, this effort would not have been possible.

The commander of ERDC was COL Christian Patterson, and the director was Dr. David W. Pittman.

1 Introduction

This special report is divided into topics relevant to understanding the subsurface environment under and surrounding the Old River Control Complex (ORCC), specifically the Old River Low Sill Structure (ORLSS). The topics included are Site Geology, Construction History, Instrumentation, Flood of 1973, Grouting Repair, and Postscour Stability Analysis. The information gathered on these topics is then condensed in the Conclusions section.

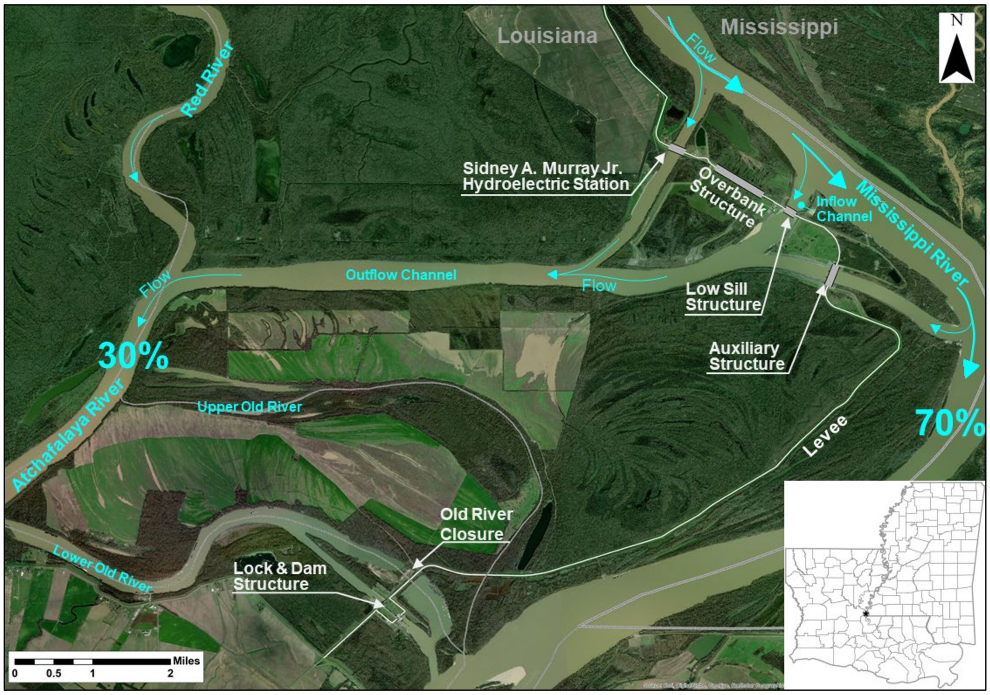
1.1 Background

The ORCC, authorized by the Flood Control Act of 1954, Public Law (PL) 780 under the 83rd Congress, is located approximately 35 miles south of Natchez, MS, and 48 miles northwest of Baton Rouge, Louisiana, along the west bank of the Mississippi River between river miles 317 and 311 in Concordia Parish, Louisiana (Figure 1). The ORCC consists of four primary structures (i.e., Old River Low Sill Structure, Overbank Control Structure [Overbank], Auxiliary Structure [Auxiliary], and the Sidney A. Murray Hydroelectric Station.) The complex also includes a navigation lock, both forebay and tailbay channels, an earthen dam closing the Old River, main-line levee extensions, and bank stabilization along the Red and Atchafalaya rivers.

1.2 Objective

The US Army Corps of Engineers (USACE), New Orleans District (MVN), tasked the US Army Engineer and Research Development Center (ERDC) with assessing the condition of a grouted scour hole located at the southeast wall of the Old River Low Sill Structure at the Old River Control Complex using noninvasive techniques. The results discussed will be used for interpretation of geophysical surveys, construction of physical models, and input for slope stability analyses.

Figure 1. Site location map of the Old River Control Complex (ORCC) (modified from Heath et al. 2015).



1.3 Approach

ORLSS was constructed between 1955 and 1958 along with the Overbank to prevent a natural cutoff from occurring from the Mississippi River into the Atchafalaya River. The Auxiliary was constructed (ca. 1986) to aid in reducing the flow of water through the existing structures, yet still allow the Corps to maintain the distribution of flow. ORLSS and Overbank structures work together with the Auxiliary and Sidney A. Murray Hydroelectric Station (ca. 1990) to maintaina 30 percent flow diversion from the Mississippi River into the Atchafalaya River.

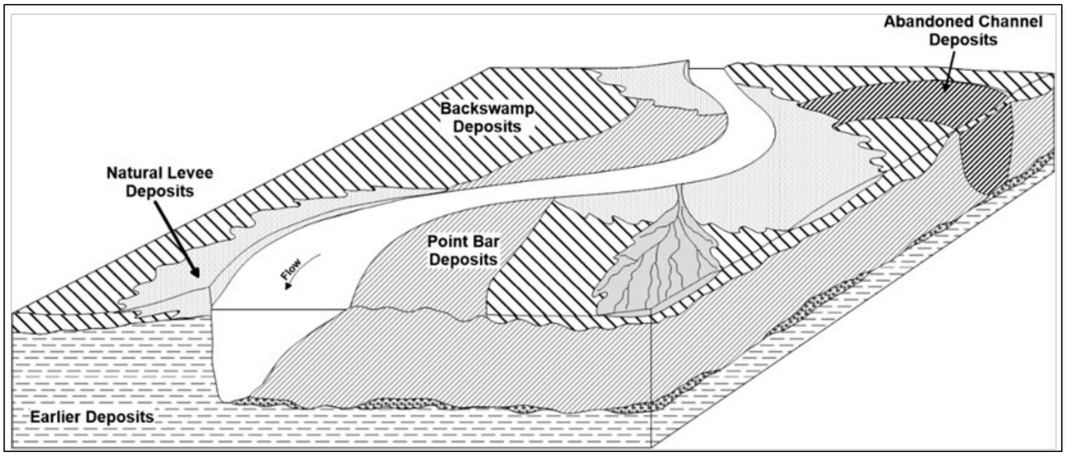
During the Flood of 1973, an eddy current produced a scour hole, proximal to the southeast upstream right wing wall, approximately 300.0 ft in diameter with a maximum depth of 60.0 ft.* Emergency repairs of ORLSS were conducted by riprap placement and grouting of the scour hole using several mixtures of low-cement, low-bond strength grout.

* For a full list of the spelled-out forms of the units of measure used in this document, please refer to *US Government Publishing Office Style Manual*, 31st ed. (Washington, DC: US Government Publishing Office, 2016), 248–252, https://www.govinfo.gov/content/pkg/GPO-STYLEMANUAL-2016/pdf/GPO-STYLEMANUAL-2016.pdf.

2 Site Geology

The geology of the ORCC area consists of four depositional environments common to the Lower Mississippi River Valley (i.e., backswamp, natural levee, point bar, and abandoned channel deposits, as identified by Fisk [1947] and Saucier [1969, 1994]). Figure 2 is a general block diagram of these depositional environments.

Figure 2. Diagram of principal geologic depositional environments in an idealized meandering river floodplain (modified from Allen 1964).



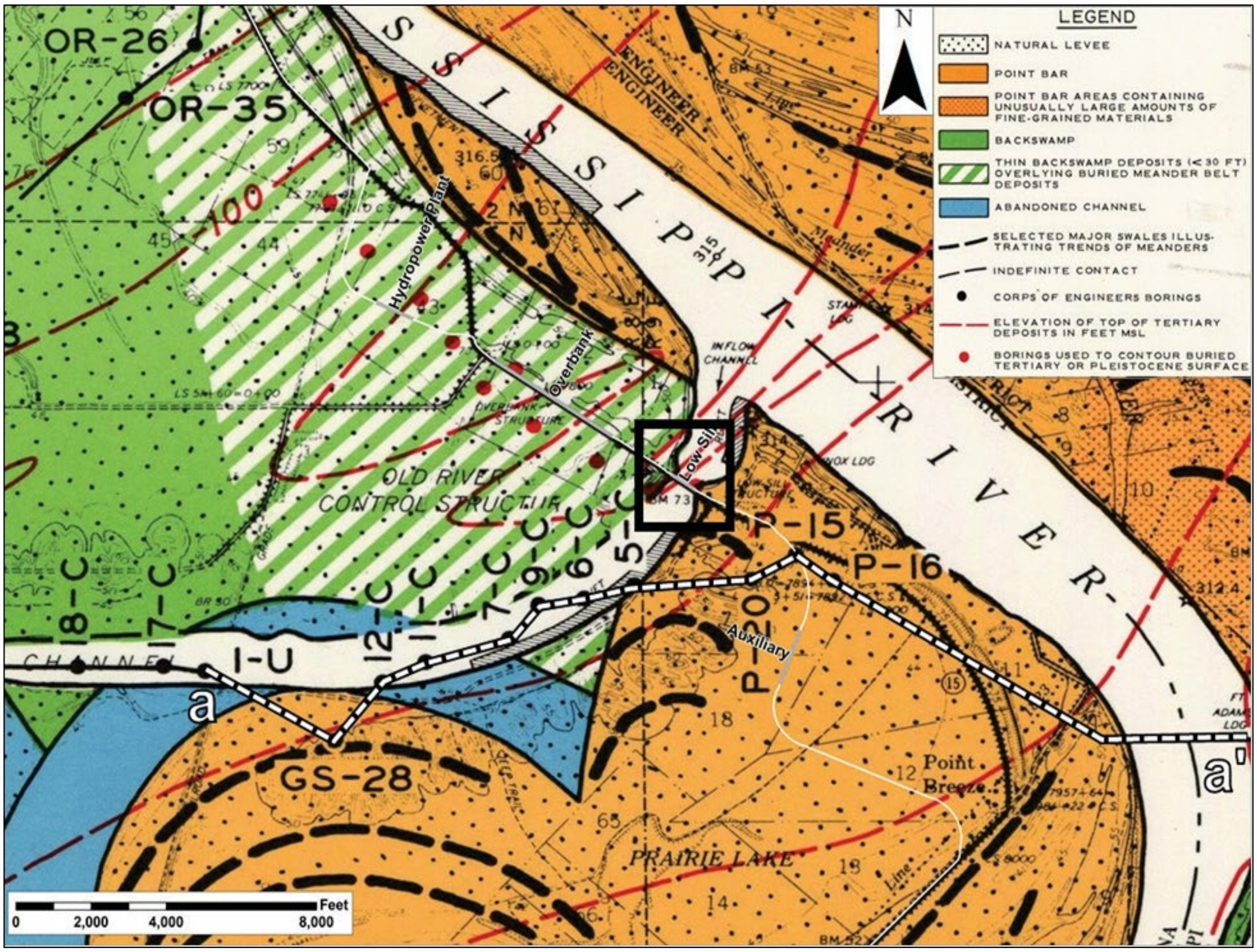
Backswamp deposits are typically broad, topographically flat, clay-rich features that are deposited during high-water events where fine-grained sediments settle out of suspension. Natural levee deposits are wedge-shaped features adjacent to rivers along its banks. Natural levees are generally composed of relatively silty materials deposited when a river overtops its banks with relatively coarse-grained sediment depositing along the banks, which generally is finer with distance from the riverbanks with a subsequent decrease in thickness. Point bars are deposited in a fining-upward sequence along the convex of a river as it migrates laterally. As a point bar grows laterally, a series of arcuate sandy ridges and finer-grained swales develop that can be discerned topographically. An abandoned channel deposit occurs when a river changes course, either gradually or instantaneously, followed by infilling of clay-rich sediment after a flood event. Eventually, an arcuate, topographically low, clay-rich deposit, also known as a clay plug, with a width comparable to the parent river remains. The geologic depositional environments dictate both the soil and foundation conditions at the site.

Abandoned channels, oxbow lakes, and abandoned courses of the Mississippi River can be discerned by aerial photography and digital elevation maps throughout the Lower Mississippi River Valley. The process known as a river avulsion occurs when the current channel slope decreases to less than a potential channel slope. A river avulsion can occur quite rapidly, for example during a flood event, or gradually by the deposition of sediment in the current channel. Recent investigations on the geomorphology of this area have been conducted that note the increase in channel bed elevation (Heath et al. 2015; Wang and Xu 2016; Knox and Latrubesse 2016). It is difficult to determine a single cause for sediment buildup or river avulsions. For example, the Mississippi River has been switching delta lobe locations for thousands of years prior to any anthropogenic development.

Investigations to determine the foundation conditions and the best locations for both ORLSS and Overbank structures were conducted using split-spoon sampling (USACE 1949, 1954). The foundation studies concluded that the borings north of ORLSS, in the planned site area for Overbank, consisted of sandy silts and silts from natural levee deposits approximately 6.0 to 10.0 ft thick underlain by clay-rich backswamp deposits 10.0 to 20.0 ft thick with silty sands and sands from point bar deposits below the backswamp deposits. Construction of the inflow channel to an elevation of -5.0 ft below mean sea level (MSL) likely resulted in removal of the overlying natural levee and backswamp clays at ORLSS. The clean sands of the substratum lie at an elevation of -65.0 ft MSL, which is why this area was selected for construction of ORLSS as it allowed for excavation of the inflow channel without exposing the clean sands. Below the clean sands of the substratum lies the suballuvial surface comprised of Miocene (deposited 23.0 to 5.3 million yr ago), and Pliocene (deposited 5.3 to 2.6 million yr ago) deposits that are predominantly clay at a depth of 100.0 to 120.0 ft below MSL.

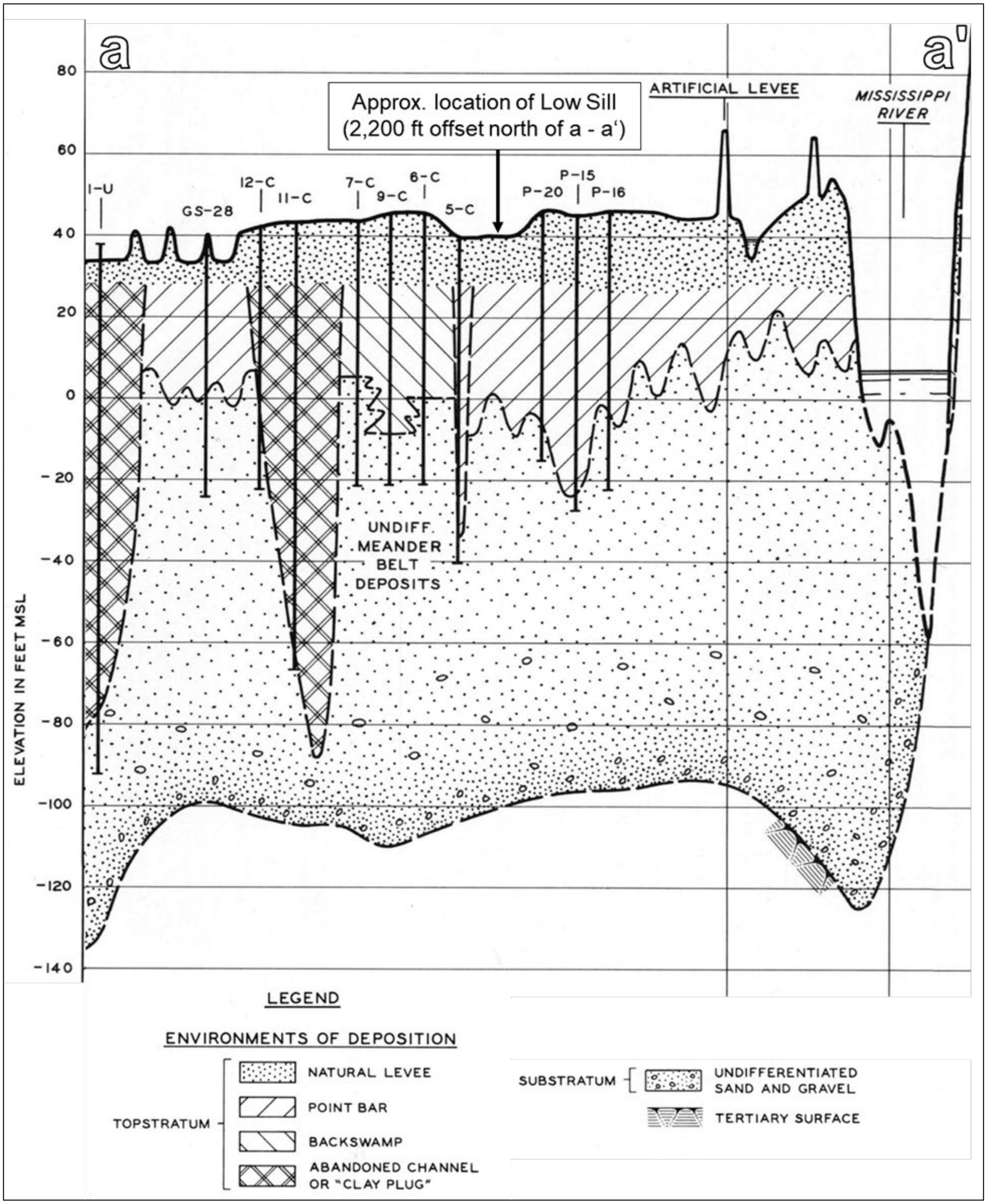
Borings at ORLSS from the foundation investigations showed alternating strata of silts, silty sands, and sandy silts interpreted in these reports (USACE 1949, 1954) to be from an abandoned channel, which would reduce the severity of underseepage beneath the structure. It is plausible that a historical course of the Mississippi River, or a tributary, intersected this area. However, the authors of this SR interpret that the alternating strata of silts and sands are consistent with point bar deposits. Fisk (1947) shows the presence of an abandoned channel but did not have boring information to validate the thickness or lithology of the feature. Furthermore, an abandoned channel is not shown in geomorphic maps from Saucier (1969, 1994) (Figures 3*A* and *B*). Resistivity profiling of the ORLSS outflow channel from Murphy (1977) also suggests the presence of a point-bar depositional environment.

Figure 3. *(A)* Depositional environments around ORCC (modified from Saucier 1969). Black box represents the study area at ORLSS. *(B)* The dashed white line is the location of the adjoining cross section.



(A)

Figure 3 (cont.). *(A)* Depositional environments around ORCC (modified from Saucier 1969). Black box represents the study area at ORLSS. *(B)* The dashed white line is the location of the adjoining cross section.

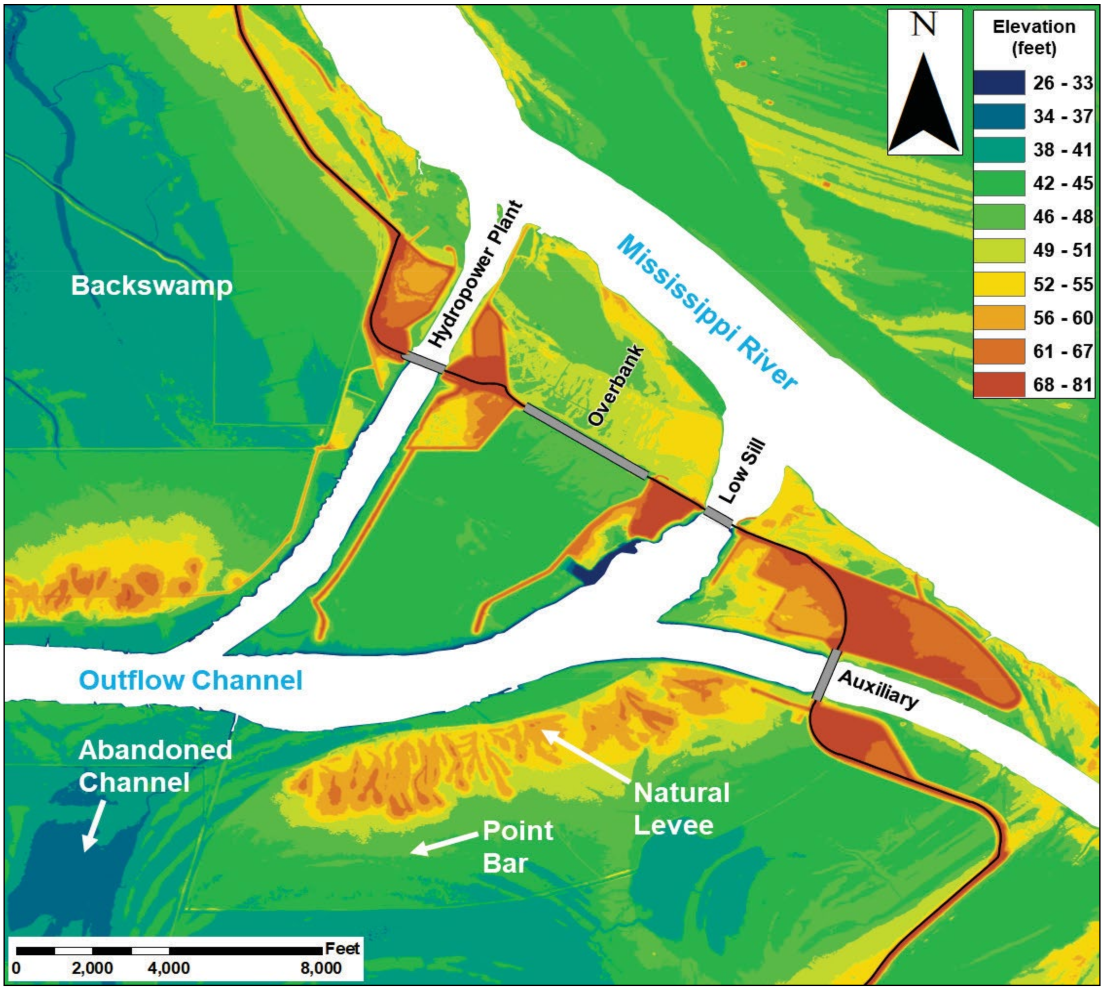


(B)

The following paragraphs are a discussion of further geologic interpretation performed by ERDC during this effort. The purpose of expanding the past research is to increase the understanding of geologic conditions that might influence the interpretation of geophysical surveys and would also be used in physical models or stability analysis.

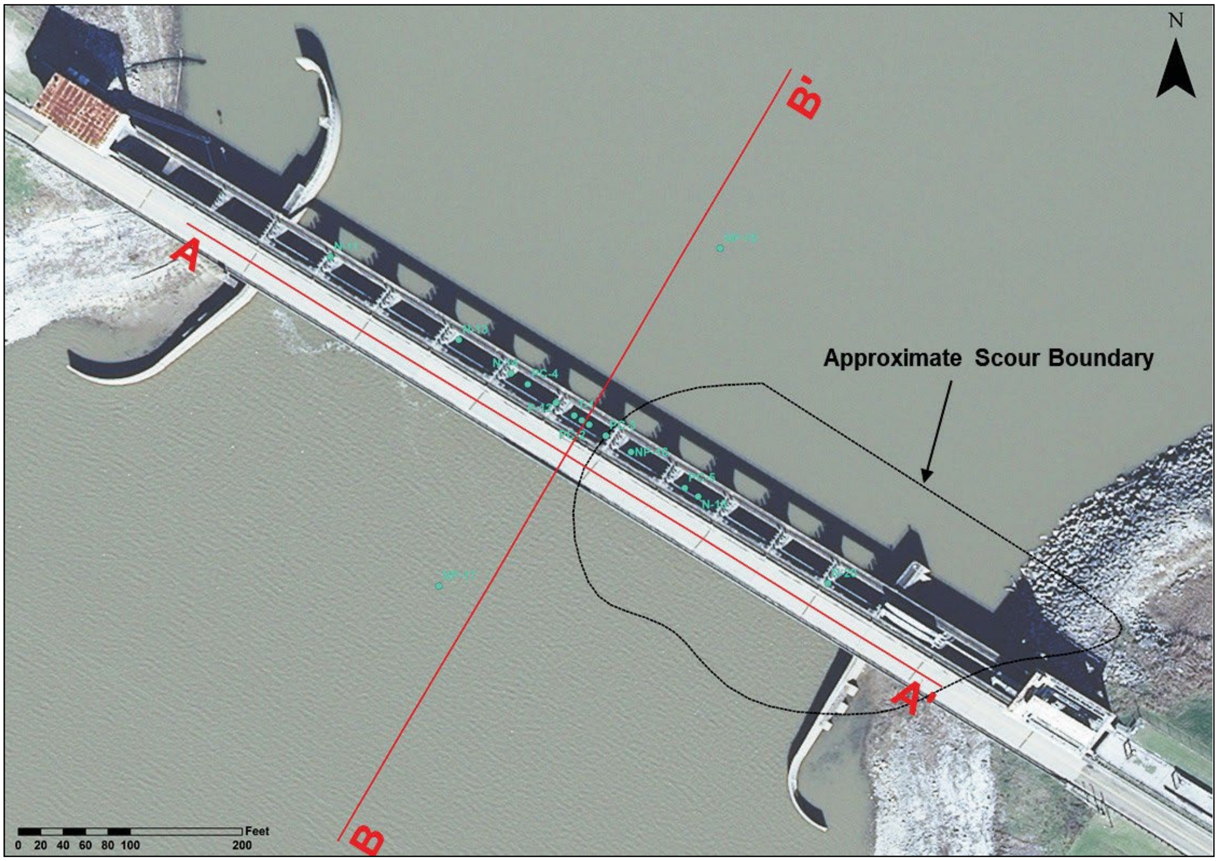
First, high-precision, ground-surface digital elevation model (DEM) data were downloaded from the US Geological Survey (USGS) National Elevation Dataset (NED). The resolution of the data is 1/9 arc-second resolution (approximately 3.0 m or 9.8 ft). Figure 4 shows the DEM with the depositional environments, as previously discussed, and ORCC structures identified. The earthen levee centerline is the solid black line. The point bar deposits are discerned by the arcuate, alternating contours. The rill-shaped contours are natural levee deposits with the higher elevations adjacent to the river that slope landward and, as described previously, decrease in thickness landward. Backswamp deposits are noted at the top left of the figure by the flat, broad, low-lying contours west of Overbank. An abandoned channel is identified by the low-lying topography with an arcuate pattern coincident of river morphology.

Figure 4. Digital elevation map (ft) of ORCC (DEM data courtesy of the USGS National Elevation Dataset, 1/9 arc-second (approx. 3.0 m or 9.8 ft) (USGS 2009).



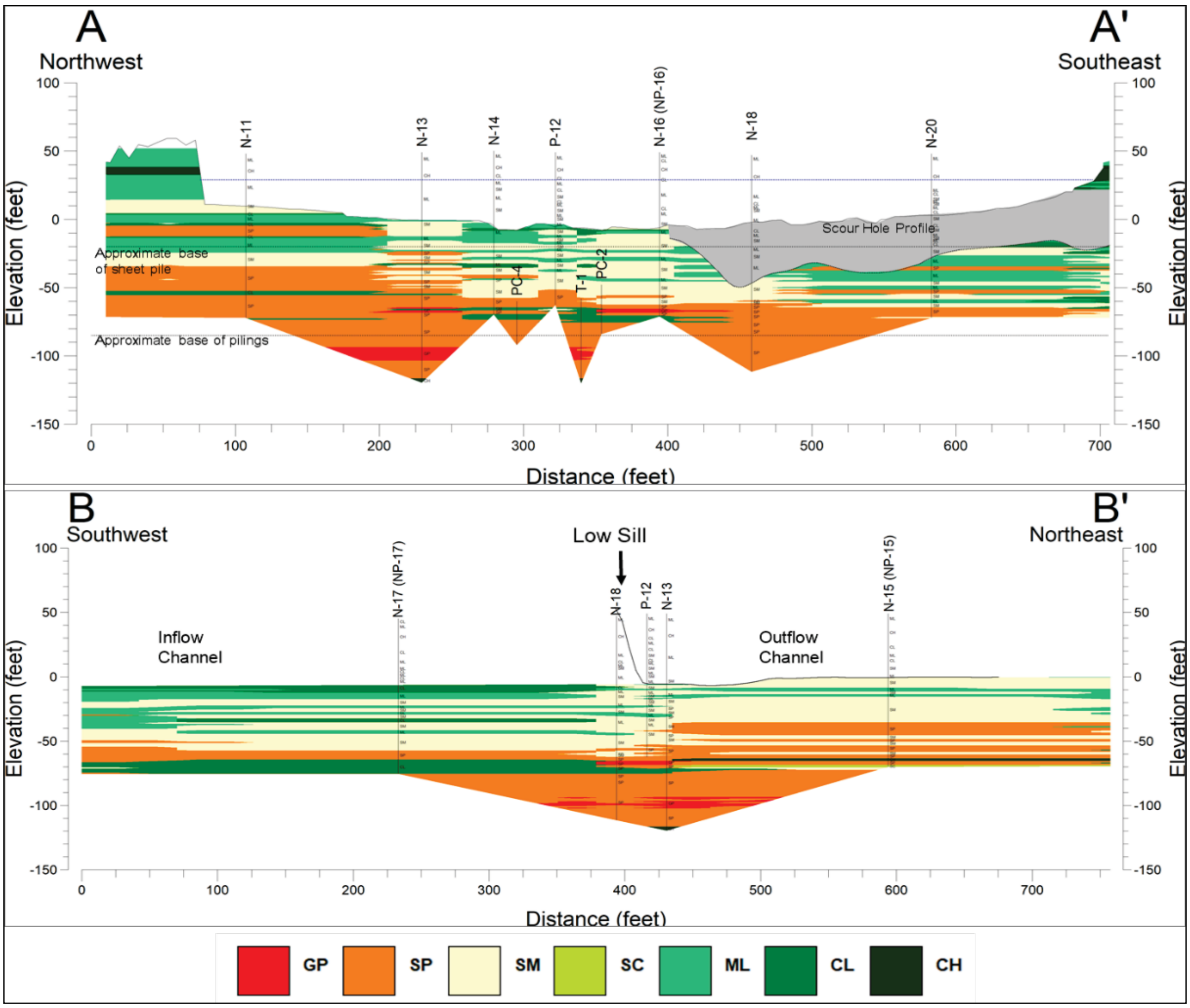
Second, boring locations from USACE (1954) were georeferenced using both the annotation geographic positions from the design memorandum and a USGS topographic map (USGS 1965) to the NAD83 datum. Next, the lithology data from the borings were compiled into a geologic database along with the geographic locations. Figure 5 shows the georeferenced boring locations and cross-section locations in relation to ORLSS. Figure 6 represents the lithologic cross sections A-A' and B-B' in respect to the Unified Soil Classification System (USCS) that were drawn and interpreted from the boring lithology data composited from USACE (1949, 1954, 1955). In addition, the approximate scour boundary of the grouted scour hole from Wilson (1978) was georeferenced in relation to the gates, monoliths, and wing walls of ORLSS (Figure 5). The profile surveys from the same report were digitized in plan view using the previously georeferenced scour boundary map and then in section view using the provided scale on the scour survey profile. Bathymetric surveys, data courtesy of MVN, were included to estimate the depth to the inflow channel bottom.

Figure 5. Locations of cross sections at ORLSS. Cross sections are shown in Figure 6.



Cross section A-A′ (Figure 6) was drawn approximately northwest to southeast along the crest of ORLSS. From northwest to southeast, substratum clean sands were encountered at an elevation of -35.0 ft and were overlain by alternating silt (ML) and silty sand (SM) layers with a poorly graded sand (SP) layer approximately 10.0 ft thick. From N-13 to the southeastern end of the section were alternating layers of SM and ML with occasional CL clays and SP strata. Poorly graded gravel (GP) strata lie at approximate depths of -65.0 ft and -100.0 ft MSL near ORLSS between borings N-13 and N-16 on A-A' and between borings N-18 and N-13 on B-B'. The GP strata were not encountered at either N-17 in the inflow channel or N-15 in the outflow channel. However, boring penetration depths were less for N-17 and N-15 than the borings closest to ORLSS on the corresponding cross sections. A 10.0-ft thick CH clay layer was encountered at an elevation of -60.0 ft that extended laterally under the central portion of ORLSS and gradually thinned eastward in the inflow channel. The elevation of the top of the clean substratum sands was approximately -60.0 to -65.0 ft as noted by USACE (1954). The tops of the Miocene-Pliocene clays (CH) are shown in boring N-13 at an elevation of -120.0 ft below MSL. The area between boring N-16 and the southern end of the section is the approximate profile of the grouted scour area. This is a gridded representation based on digitized points from the scour survey profiles (Wilson 1978). Cross section B-B' was drawn perpendicular to the axis of ORLSS roughly southwest to northeast. The soil types showed a similar pattern as A-A' with alternating SM and ML strata underlain by SP at an elevation of -35.0 ft in the inflow channel and -55.0 ft in the outflow channel.

Figure 6. Cross sections A-A' parallel to ORLSS and B-B' perpendicular to ORLSS.

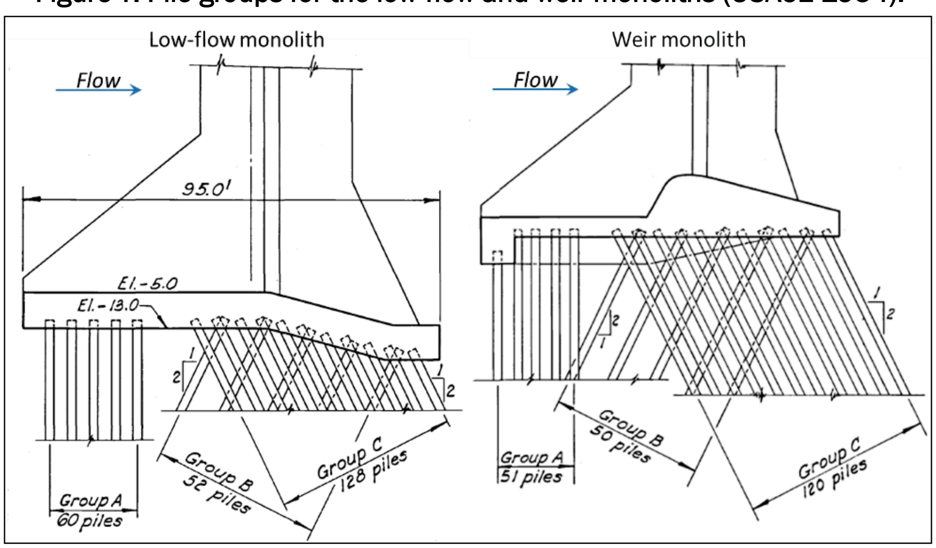


3 Construction History

ORLSS was designed to pass 350,000 ft^3/s of water and 700,000 ft^3/s when used in conjunction with Overbank structure. ORLSS is a pile-founded, reinforced concrete structure with 11 44 ft wide steel-gate bays separated by concrete monoliths with an overall length of 566.0 ft between two concrete wing walls. The three central bays are for low flow with a crest elevation of -5.0 ft MSL, and the outer eight bays are a weir type with a crest elevation of 10.0 ft MSL. The inner three bays are open-channel, nonuniform type with a sudden change in the bottom from horizontal at elevation -5.0 ft to a slope of 1 on 4 at a point opposite the beginning of the downstream curve of the outer weirs. The gates were designed to close from the bottom up with multiple leaves—four for the central bays and three for the eight outer bays. The gates were placed by a gantry crane mounted at the top of the structure. The crest of the gates is at elevation 67.0 ft MSL.

The pile foundation of ORLSS consists of three groups of piles noted as group A, B, and C. Pilegroup A is oriented vertically; group B is battered at a 2 on 1 slope with tip battered towards the downstream direction; and group C piles are battered towards the upstream direction. Design loads for the piles are 200 kips compression and 80 kips tensions, although only pile group C was installed with tension fasteners (USACE 1975). Figure 7 shows the pile group locations for the low-flow and weir monoliths. The primary reasons for using a pile foundation for this structure were to reduce settlement and increase resistance to sliding.

Figure 7. Pile groups for the low-flow and weir monoliths (USACE 1954).



Prior to construction of ORLSS, pile load tests were conducted. Results of these load tests indicated that a 14.0 in. H-beam and 20.0 in. precast pile, with 27.0 ft and 12.0 ft penetration in the sand aquifer, respectively, would result in a factor of safety of 1.35 for the underlying sand foundation (USACE 1956b). Final design incorporated the 14.0-in. H-beams. The foundation below ORLSS, as previously stated in the Site Geology section of this SR, consists of silts (ML) and silty sands (SM) to an elevation between -60.0 and -77.0 ft MSL. A range is given because of the variable nature of this contact at the site. Below the silt (ML) and silty sand (SM) layers are clean SP sands that grade coarser with depth. This was considered a suitable bearing layer and, during design of the pile load tests, it was planned that 20% of the load would be carried through skin friction (or adhesion) and 80% of the load would be carried by end bearing of the piles.

The abutments and wing walls were not founded on piles and used shallow foundations. A preload was applied for a period of approximately 1 yr to accelerate consolidation of the abutment foundation and to minimize postconstruction settlements of the abutments. Without the preload, it was predicted that differential settlement between the pile-supported weir and abutments could be on the order of 10.0 in.

The inflow channel that connects ORLSS to the Mississippi River has an invert elevation of -5.0 ft MSL and is 1,000.0 ft wide. Channel protection was provided from the centerline of the weir and extended 575.0 ft upstream. The initial 100.0 ft upstream from the weir is a 1.5 ft thick concrete apron. At the end of the apron, 25.0 ft of 3.5 ft derrick stone were placed, followed by 75.0 ft of 2.5 ft riprap, and the remaining distance contained 1.5 ft riprap. The 3.5 ft derrick stone and 2.5 ft riprap extended beyond the approach wing walls. The slopes of the approach channel were designed for a 1 on 4 slope with 1.5 ft riprap. A 6.0 ft thick impervious compacted clay blanket was installed beneath a seepage filter and concrete apron. Beyond the concrete apron, the clay blanket extended 175.0 ft and was 3.5 ft thick overlain by a filter blanket and stone (derrick or riprap).

The stilling basin is a rectangular concrete channel with a width of 566.0 ft at the downstream edge of the gate piers and flares to 592.0 ft at the end sill. The central portion coincides with the low flow bays and has a floor elevation of -12.0 ft that terminates at an end sill with an elevation of -9.0 ft. The outer portion of the stilling basin coincides with the eight bays with weirs and has a floor elevation of -5.0 ft that terminates at an end sill with an elevation of -2.0 ft. Two rows of staggered baffle blocks 10.0 ft high and spaced 12.0 ft apart are provided across the stilling basin.

A drainage system was installed beneath the gate bays and stilling basin. Beneath the gate bays, the drainage system consists of an 8.0 in. gravel and 6.0 in. sand filter blanket running the full length of the gated portion and is drained by two collector pipes, which discharge into manholes, numbered 1 and 2, on the outside of each end pier. These manholes then drain into manhole number 4, which is located on the outside of the stilling basin. The collector pipes are 12.0 in. clay tile pipe—the lower half of which is perforated with 0.25 in. holes. Manhole 4 discharges through a valve into the stilling basin; the valve prevents flow from the stilling basin into the system during periods of reverse flow.

The stilling basin drainage system consists of an 8.0 in. gravel and 6.0 in. sand filter blanket that run the full length of the stilling basin and is also drained by two collector pipes, similar to the gate bay drainage system. The stilling basin drainage collector system drains into manholes 3 and 5 on the outside of each stilling basin end wall. Manhole 5 drains into manhole 3, and this discharges into the stilling basin through valves, as in manhole 4.

In addition to the drainage system, a row of seven relief wells was installed. A factor of safety with regards to heave, or upward movement of the ground surface, was calculated to be greater than 1.5 from the stilling basin to the interface of the ML silts and clean SP sands. The stratified nature of the foundation between the bottom of the structure and the clean SP sand interface led the designers to believe that pressures could increase at a higher elevation, so relief wells were installed. The relief wells are in the stilling basin and are plumbed through the previously mentioned manhole system.

Uplift is a force that acts on structures when pressures are not dissipated effectively with a drainage system. It results in upward movement downstream of the structure. The uplift beneath the structure considered for design of ORLSS consisted of two cases. The first case was 0% uplift beneath the structure, which assumes the impervious upstream blanket, drainage blanket, and relief well system are operating as designed. The second case assumes no drainage system and 50% net head added to tailwater head applied at the upstream edge of the foundation and tailwater head plus 10% net head applied at the downstream edge. The design hydraulic loadings are shown in Table 1; maximum differential head considered was 35.0 ft.

Table 1. Design hydraulic loadings (negative differential head values indicate reverse loading.)

Gate settings	Headwater Elevation (ft MSL)	Min. Tailwater Elevation (ft MSL)	Differential Head (ft)
Closed	35.0	0.0	35.0
Closed	23.0	32.0	-9.0
Closed	23.0	38.0	-15.0
3 center open	54.0	20.0	34.0
3 center open, 1 outside gate open each side	67.0	40.0	27.0
3 center open, 1 outside gate open each side	67.0	30.0	37.0
Two gates closed on one side	62.0	49.0	13.0
All open	62.0	52.0	10.0
All open	67.0	53.5	13.5

After construction of ORLSS, Mississippi River conditions were abnormally low for the first 10 yr of operation. The operation of the structure was adequate with only a few issues, which were related mainly to hydraulic performance and hazards to river navigation. These two issues were related in that water currents from the Mississippi River into ORLSS's inflow channel would divert unwary vessels or those that broke free of their moorings. The removal of these vessels required closure of the gates, resulting in a large differential head across the structure. The differential heads could approach those of design. On reopening of the gates, extensive downstream scour would result (USACE 1979).

Table 2 shows a list of incidents that occurred in the inflow and outflow channels in the first years of operation of the structure. The major incidents are all pertaining to scour either in the inflow or outflow channel. The scour hole that developed in 1967 on the inflow channel is documented in Periodic Inspection number 1 (USACE 1967). The scour hole was identified near the south bank of the inflow channel and, in September of 1967, an articulated concrete mattress was placed over part of the scour hole; riprap was placed over the remaining area in January 1968. USACE (1979) states that scouring of the bed and banks of the inflow and outflow channels of ORLSS tended to be a recurring problem, but without any damage to the structure. Recent scour events have been noted downstream of ORLSS near river mile 310 in the Mississippi River after the 2011 flood (Knox and Latrubesse 2016).

Table 2. ORLSS structure scour incidents during early operation (USACE 1975).

Date	Repair	Location
Jan 1962	10,700 tons of riprap	North outflow channel
Feb-Aug 1962	58,000 tons of riprap	Outflow channel
Apr-Dec 1964	276,000 tons of riprap and 17,560 squares of concrete mattress	Outflow channel
Sep 1965	20,200 tons of riprap	Inflow channel
Mar 1966	6,300 tons of derrick stone	Outflow channel
Mar-May 1966	38,500 tons of derrick stone and 3,700 tons of riprap	Outflow channel
Nov-Dec 1967	22,200 tons of riprap	Inflow channel
Jan 1968	16,900 tons of riprap	Inflow channel
Dec 1968	3,840 squares of concrete mattress	Outflow channel
Aug 1971	4,700 squares of concrete mattress	Inflow channel
Apr-Jun 1973	335,000 tons of riprap and 33,000 cubic yards of grout	Forebay, wing wall, below ORLSS
Apr-Jun 1974	210,000 tons of riprap	Outflow channel
Mar 1975	8,700 tons of riprap	Forebay

4 Instrumentation

The USACE technical report 3-602, Review of Soils Design, Construction and Performance Observations Low Sill Structure, Old River Control (USACE 1962), provides an in-depth overview of the construction process from 1955 to 1958. Monitoring instrumentation at the site began with the construction of a dewatering system that consisted of seepage blankets, relief wells, and a drainage network. The instrumentation consisted of settlement reference marks, alignment reference marks, piezometers, and scour ranges. The Flood of 1973 damaged some of the dewatering system as well as some of the monitoring instrumentation

4.1 Dewatering system during construction

The required area of excavation for ORLSS was 1,500.0 ft long and 600.0–1,000.0 ft wide to a depth ranging from 55.0 to 70.0 ft (USACE 1956a). Piezometers installed in 1954 into the deep sand layer showed that the hydrostatic pressure reflected changes in the stage of the Mississippi River. A consideration was made at the time that a maximum river stage elevation of 60.0 ft, which might happen during construction, would be expected to cause a net head of approximately 70.0–80.0 ft beneath the excavation. This differential pressure would possibly lead to sand boils in the middle of the excavation unless a system of relief wells was placed to relieve this pressure (USACE 1956a). Figure 8 shows a series of both temporary and permanent drainage wells placed to address dewatering (USACE 1962).

Excavation began in October 1955 and progressed to an elevation of 30.0 ft without dewatering the slopes or relieving the hydrostatic pressure in the deep sand. Twenty temporary relief wells were constructed to relieve pressure in the deep sands once the excavation passed from +30.0 to +20.0 ft. Pumping in these wells commenced from February through March 1956. Construction began on the first of the slope well points during this period of time as well. Seven permanent relief wells were installed before excavation reached an elevation at 20.0 ft and during April and May 1956 (USACE 1962). During that time, five more temporary wells were installed along with a line of temporary well points in each wing of the excavation. Table 3 lists the relief wells and the result of pumping tests that were conducted.

Figure 8. Plan view of dewatering system (USACE 1962).

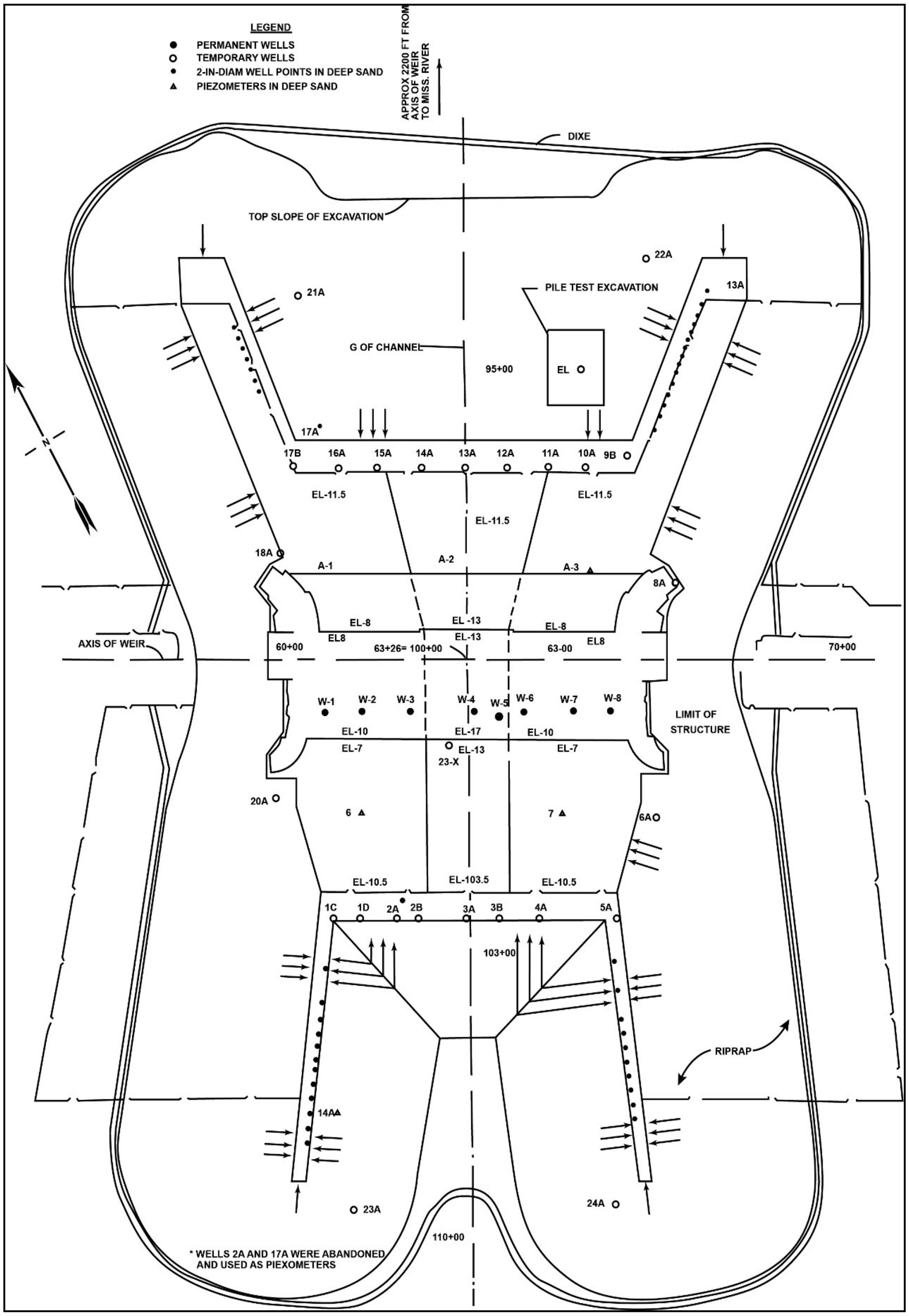


Table 3. A list of temporary and permanent relief wells with pumping test results (USACE 1962).

Installation and Pumping Test Data for Relief Wells							
				As Installed Elevation (ft)			
Well No.	Station	Distance and Direction from ∅ (ft)	Date of Installation	Inside Bottom of Well	Top of Well Screen	Top of Filter	Specific Yield (gpm/ft)
Permanent Wells							
W-1	100+89.7	256N	3/30/56	-108.1	-78.8	-48.0	50.3
W-2	100+89.7	190N	4/3/56	-111.8	-80.6	-48.3	57.3
W-3	100+89.7	102N	4/10/56	-112.1	-80.9	-48.7	52.2
W-4	100+89.7	11S	4/13/56	-111.4	-80.3	-50.3	55.8
W-5	100+89.7	102S	4/23/56	-112.3	-81.2	-49.3	42.9
W-6	100+89.7	109S	4/25/56	-111.6	-80.2	-48.0	35.4
W-7	100+89.7	256S	5/20/56	-111.9	-80.9	-50.0	30.6
Temporary Wells							
1-C	104+50	246N	2/21/56	-109.3	-80.2	-70.2	43.1
1-B	104+50	197N	1/17/56	-114.5	-84.5	-74.5	45.1
2-B	104+50	92N	2/21/56	-110.7	-80.6	-70.6	45.1
3-A	104+50	5N	1/18/56	-111.9	-81.9	-71.9	50.2
3-B	104+50	53S	5/23/56	-112.9	-82.9	-72.0	57.0
4-A	104+50	125S	1/26/56	-112.0	-82.0	-72.0	32.8
5-A	104+50	264S	2/8/56	-108.2	-78.2	-68.2	31.1
6-A	102+74	337S	3/21/56	-110.0	-80.0	-70.0	41.9
8-A	98+71	375S	2/25/56	-111.5	-79.4	-69.4	36.8
9-B	96+48	289S	1/13/56	-108.5	-78.5	-68.5	—
10-B	96+68	216S	1/21/56	-115.6	-85.6	-75.6	45.1
11-A	96+68	150S	12/28/55	-114.6	-84.6	-74.7	43.1
12-A	96+68	75S	12/31/55	-114.1	-84.1	-74.0	35.1
13-A	96+68	1.58	1/5/56	-113.4	-83.4	-73.4	37.7
14-A	96+68	78N	1/9/56	-114.0	-83.9	-74.0	50.0
15-A	96+68	158N	1/12/56	-113.9	-83.9	-73.9	53.6
16-A	96+68	227N	1/11/56	-114.2	-84.2	-74.2	46.5
17-B	96+65	308N	1/20/56	-115.9	-85.9	-75.9	50.0
18-A	98+15	333N	2/24/56	-106.8	-76.7	-66.7	66.7
20-A	102+39	346N	2/23/56	-107.6	-77.6	-67.6	45.4
21-A	93+76	287N	2/15/57	-112.1	-82.1	-70.0	48.6
22-A	93+13	333S	2/17/57	-115.4	-85.4	-75.0	—
23-A	90+40	256S	2/26/57	-115.2	-85.1	-75.0	—
24-A	90+40	208N	2/18/57	-114.5	-84.5	-74.0	—
25-X	101+63	55N	7/12/57	-115.2	-60.5	—	—

Note: *N* and *S* denote north and south, respectively; ∅ refers to center line of channel.

Slope dewatering was accomplished by the construction of a well-point system from 1956 to 1957 that consisted of three stages of well points connected to header pipes installed at elevations 20.0, 5.0, and -10.0 ft (USACE 1962). Figure 9 shows the basic components of the slope dewatering system. The well points were installed at a depth of 25.0 ft below the header pipes and were surrounded by sand. They were 1.5 in. in diameter with 60.0-gage mesh screens 36.0 in. long (USACE 1962). Erosion of the slopes by surface runoff occurred early in the excavation. To avoid further erosion, a small dike was placed around the excavation with a system of collection ditches that led to a series of sumps and pumps (USACE 1962).

A perimeter trench with a bottom elevation at 5.0 ft was constructed so that the second stage of the well-point system could be installed. Discharges from surface runoff, deep wells, and the rest of the dewatering system were routed through two 30.0 in. discharge pipes that drained into the Mississippi River on one side and, on the other side, drained through a cofferdam into the Red River. Final excavation in the stilling basin area was completed by January 1957 (USACE 1962).

It was later noted that pumping efficiency had declined slightly in some of the wells because of iron encrustations that developed (USACE 1962). An additional five deep, larger-diameter permanent wells were installed along with some additional well points placed in the training wall extension to the main excavation. Cleaning these wells with acid began in July 1958 and continued for several months.

4.2 Permanent drainage systems in the foundation

USACE Technical Report No. 3-602 (USACE 1962) describes drainage blankets beneath the gate bays and stilling basin that were placed to control uplift beneath the structure. In addition, the report lists the presence of an impervious blanket upstream of the gate bay, relief wells penetrating the deep sands beneath the stilling basin, and sheet piling beneath both the upstream and downstream portions of the structure. Figure 10 shows a plan view of the foundation drainage system after construction.

Figure 9. Slope dewatering system (USACE 1962).

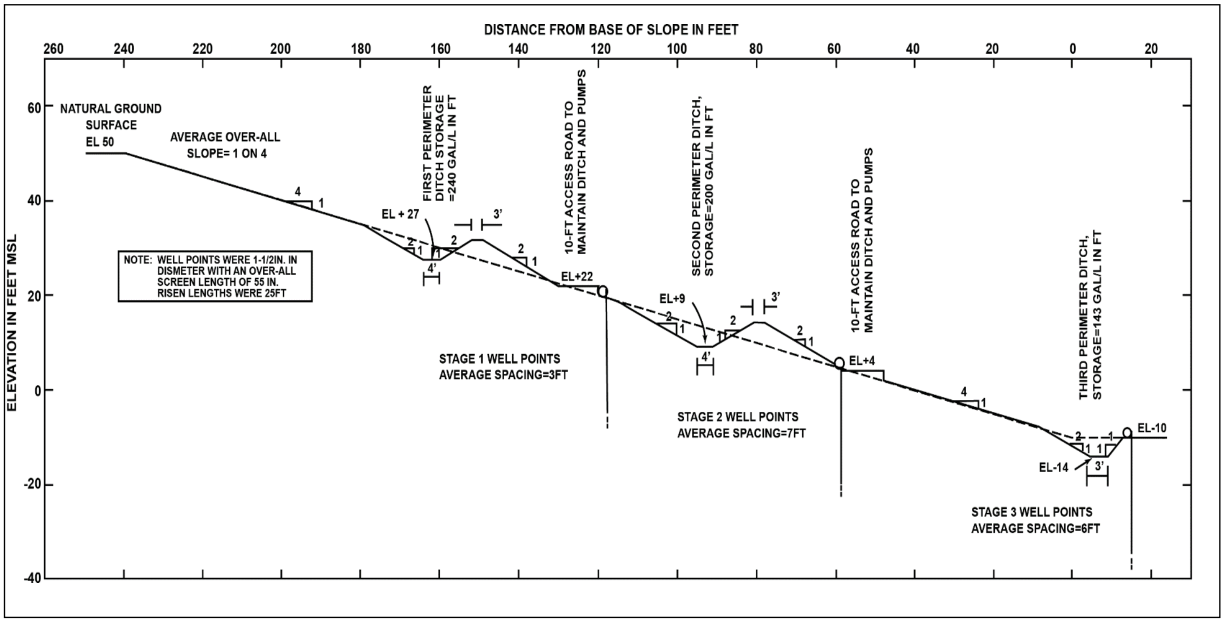
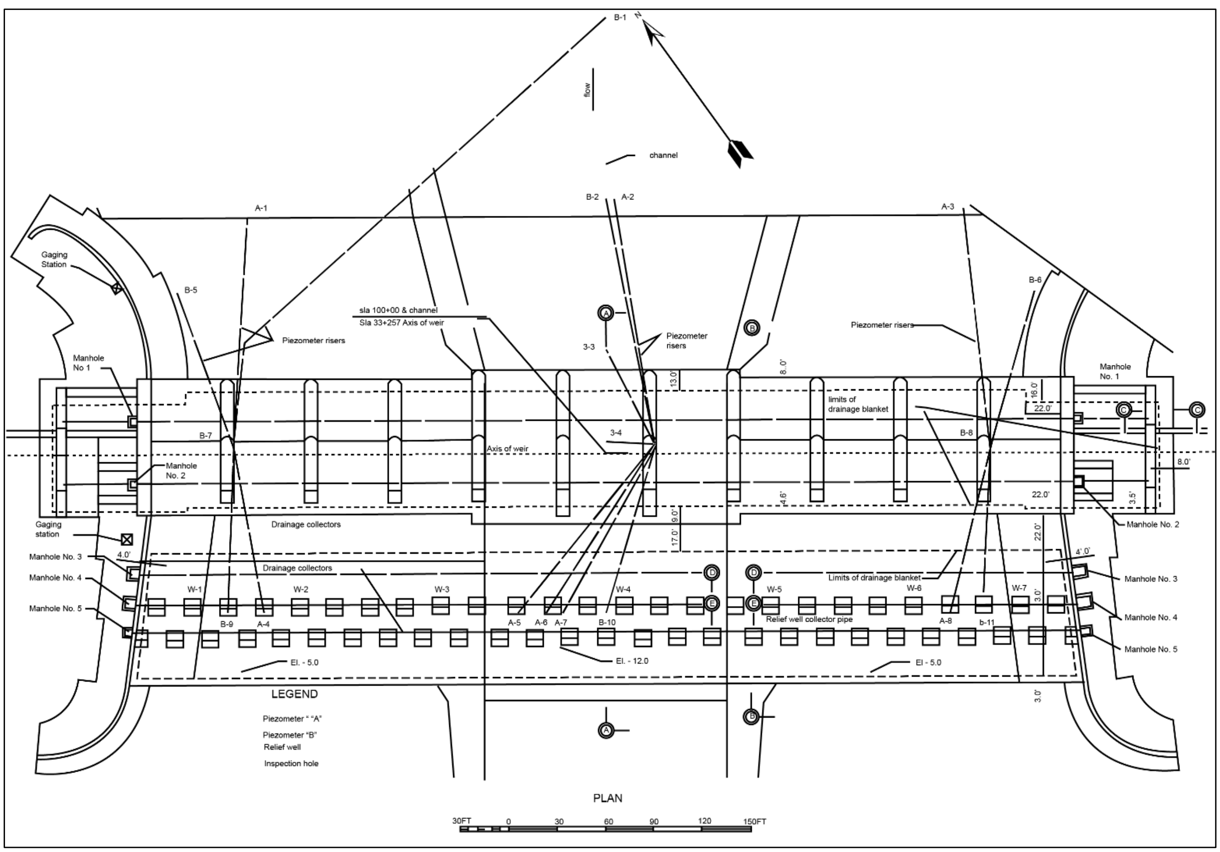


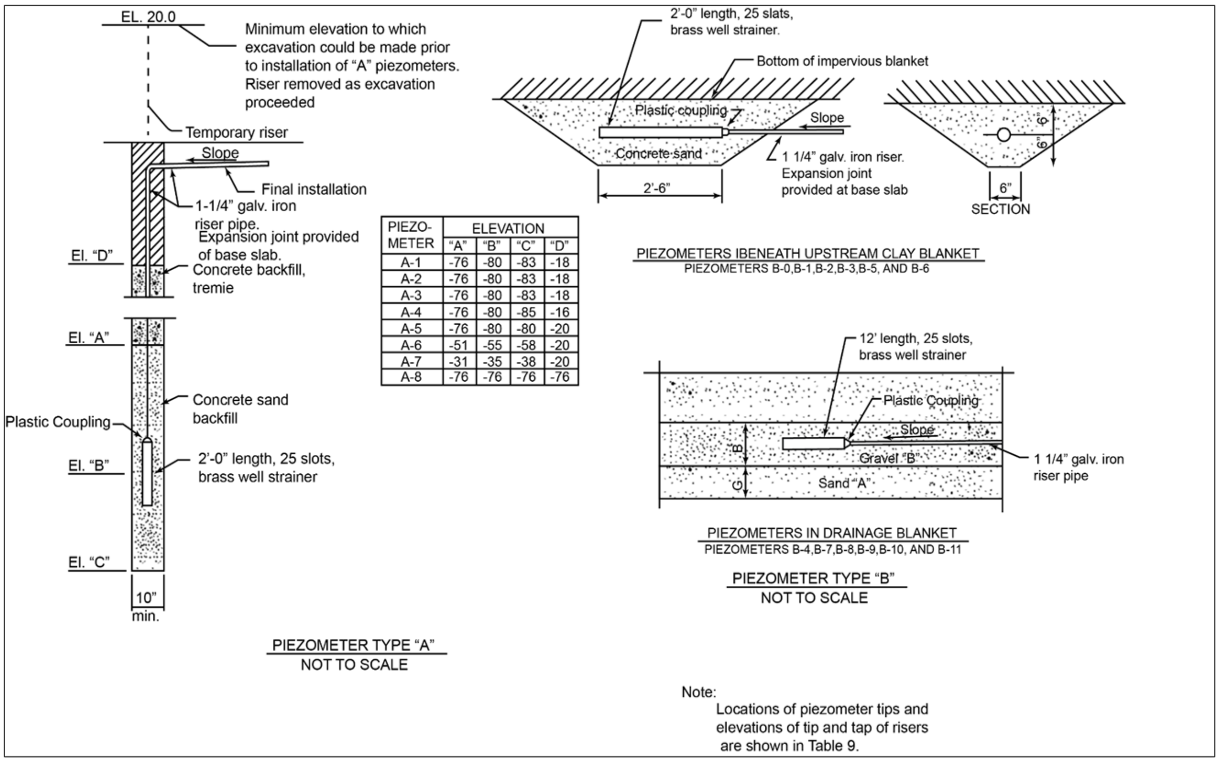
Figure 10. Plan of drainage system, relief wells, manholes, and piezometers (USACE 1962).



4.3 Piezometers in place following construction

A system of piezometers was placed to monitor hydrostatic pressures in the foundation and to check how well the drainage system was functioning. Figure 11 illustrates the differences between type A and type B piezometers with Table 4 showing the names and locations of each piezometer.

Figure 11. Typical arrangement of type A and B piezometers (USACE 1962).



PIEZO-METER	ELEVATION "A"	"B"	"C"	"D"
A-1	-76	-80	-83	-18
A-2	-76	-80	-83	-18
A-3	-76	-80	-83	-18
A-4	-76	-80	-85	-16
A-5	-76	-80	-80	-20
A-6	-51	-55	-58	-20
A-7	-31	-35	-38	-20
A-8	-76	-76	-76	-76

Table 4. A list of piezometers after construction.

Location of Piezometers					
Piezometer No.	Channel Station	Offset, ft	Elevation Center of Tip (ft)	Elevation Top of Riser (ft)	Remarks
B-1	97+25	Center line	-12.00	74.01	Beneath upstream clay blanket
B-2	98+38	Center line	-14.50	74.01	Beneath upstream clay blanket
B-3	99+33.5	Center line	-13.50	73.95	Beneath upstream clay blanket
B-4	99+91.5	Center line	-13.50	73.94	In drainage blanket
B-5	98+98	269 N	-8.50	73.95	Beneath upstream clay blanket
B-6	98+98	267.5 S	-8.50	73.95	Beneath upstream clay blanket
B-7	99+91.5	338 N	-5.33	74.00	In drainage blanket
B-8	99+92.5	236.5 S	-5.33	73.95	In drainage blanket
B-9	101+02	237.5 N	-12.35	74.00	In drainage blanket
B-10	101+02	Center line	-19.33	73.93	In drainage blanket
B-11	101+02	236 S	-12.32	73.96	In drainage blanket
A-1	98+48.17	223 N	-80	74.00	In deep sands, upstream of structure
A-2	98+40	4 S	-80	74.00	In deep sands, upstream of structure
A-3	98+48.17	223 S	-80	73.95	In deep sands, upstream of structure
A-4	100+98.17	212 N	-80	74.00	In deep sands, beneath stilling basin
A-5	100+98.17	55 N	-80	74.00	In deep sands, beneath stilling basin
A-6	100+98.17	38 N	-55	73.93	In silt stratum, beneath stilling basin
A-7	100+98.17	28 N	-35	73.94	In silt stratum, beneath stilling basin
A-8	100+98.17	212 S	-80	73.96	In deep sands, beneath stilling basin

Three type A piezometers were installed in the deep sands beneath the upstream approach channel paving so that the distance to the source of seepage could be determined (USACE 1962). Five more type A piezometers were installed beneath the stilling basin to check the performance of the relief well system. Two of these piezometers were place in the silt stratum that overlies the deep sands to monitor the upward gradient beneath the stilling basin when it was dewatered. Five type B piezometers were installed beneath the upstream clay blanket to monitor the performance of the clay blanket during periods of high water. Also, these piezometers monitored uplift pressures beneath the blanket when subjected to reverse head. Six more type B piezometers were installed in the drainage blanket for the purpose of monitoring the drainage system and to check valves (USACE 1962). It was planned to have the piezometers read at 6-month intervals until the construction was completed. After that, piezometers were to be read only if the river stage had a significant rise.

In December 1981 and January 1982, three open-type piezometers were to be installed at the base of the downstream stilling basin to monitor hydrostatic uplift and to replace the original instrumentation that was destroyed by the scour and flood in 1973. Only RPS-6 (a replacement for B-10) and RPS-8 (a replacement for B-11) were completed (USACE 1981). A memorandum of record (USACE 1981) describes cleaning of many of the piezometers that were plugged with silt or mud. The effort was successful with the exception of a return of oil after RP-2 was flushed. USACE (2014) gives the time of installation of all the piezometers on the structure as well as the current operational condition (Table 5).

Table 5. Functioning piezometers as of 2014 (USACE 2014).

Piezometer No.	Tip, Elevation (ft)	Location	Status
A-1 (1956)	-80	In deep sands below concrete apron	Currently being read
A-2 (1956)	-80	In deep sands below concrete apron	Currently being read
A-3 (1956)	-80	In deep sands below concrete apron	Currently being read
A-4 (1956)	-80	In deep sands beneath stilling basin	Currently being read
A-5 (1956)	-80	In deep sands beneath stilling basin	Currently being read
A-6 (1956)	-55	In silt stratum beneath stilling basin	Currently being read
A-7 (1956)	-35	In silt stratum beneath stilling basin	Currently being read
A-8 (1956)	-80	In deep sands beneath stilling basin	Currently being read
B-1 (1958)	-12	Beneath Upstream (U/S) clay blanket	Currently being read
B-2 (1958)	-14.5	Beneath U/S clay blanket	Currently being read
B-3 (1958)	-13.5	Beneath U/S clay blanket	Currently being read
B-4 (1958)	-13.5	In U/S drainage blanket	Currently being read
B-5 (1958)	-8.5	Beneath U/S clay blanket	Currently being read
B-6 (1958)	-8.5	Beneath U/S clay blanket	Currently being read
B-7 (1957)	-5.3	In U/S drainage blanket	Currently being read
B-8 (1957)	-5.3	In U/S drainage blanket	Currently being read
B-9 (1957)	-12.3	In Downstream (D/S) drainage blanket	Currently being read
B-10 (1957)	-19.3	In D/S drainage blanket	Currently being read
B-11 (1958)	-12.3	In D/S drainage blanket	Currently being read
RP-1 (1975)	n/a	Beneath stilling basin—remote piez	
RP-2 (1975)	n/a	Beneath stilling basin—remote piez	No longer monitored due to technical problems. Replaced by RP-14 thru RP-19
RP-3 (1975)	n/a	Beneath stilling basin—remote piez	
RP-4 (1975)	n/a	Beneath stilling basin—remote piez	
RP-5 (1976)	-11.5	At base of gate segment	Currently being read
RP-6 (1976)	-13.5	At base of gate segment	Currently being read
RP-8 (1976)	-23.5	At base of grout beneath gate segment	Currently being read

Piezometer No.	Tip, Elevation (ft)	Location	Status
RP-9 (1976)	-13.5	At base of gate segment	Currently being read
RP-10 (1976)	-34.5	At base of grout beneath gate segment	Currently being read
RP-11 (1976)	-11.5	At base of gate segment	Currently being read
RP-12 (1976)	-36	At base of grout beneath gate segment	Currently being read
RP-13 (1976)	-11.9	At base of gate segment	Currently being read
RP-14 (1976)	-12.5		
RP-15 (1976)	-19.5		
RP-16 (1976)	-19.5	Remote piezometers installed beneath stilling basin. They are equipped with a device that sends out radio signals according to water pressure. Antennas picked up signals and relayed them to a panel	Readings were discontinued in March 1979. Transmitter failed due to weak battery voltage.
RP-17 (1976)	-12.5		
RP-18 (1976)	-12.5		
RP-19 (1976)	-12.5		
RP-20 (1976)	-5.9	Beneath gate segment	Currently being read
RP-21 (1976)	-6.2	Beneath gate segment	Currently being read
RP-22 (1976)	-6.1	Beneath gate segment	Currently being read
RP-23 (1976)	-30.9	At base of grout beneath gate	Currently being read
RP-24 (1976)	-5.2	Beneath gate segment	Currently being read
RPS-6 (1981)	-19.5	Beneath stilling basin	Currently being read
RPS-8 (1982)	-12.3	Beneath stilling basin	Currently being read
RWS (1983)	-5	n/a	Abandoned
RPS-11 (1987)	-12.5	Beneath stilling basin	Currently being read
RPS-14 (1987)	-12.5	Beneath stilling basin	Currently being read
300N (1987)	-12.20	N - WW (manhole)—inflow	Currently being read
300S (1987)	-12.20	S - WW (manhole)—inflow	Currently being read
500N (1987)	-12.20	N - WW (manhole)—inflow	Currently being read
500S (1987)	-12.20	S - WW (manhole)—inflow	Currently being read

4.4 Damage to dewatering system from 1973 flood

The scour hole that occurred in 1973 resulted in the loss of part of the foundation and dewatering system. Most of the damage occurred in the vicinity of the left descending upstream wing wall. Severe damage resulted to portions of the drainage and pressure relief systems as well as a number of piezometers in both the main structure and in the stilling basin (USACE 1974, 1975).

An inspection report (USACE 1974) reviewed damage to the structure after the flood and after some of the repairs that included grouting. A detailed list of what drainage structures and manholes were either damaged or plugged with muck, grout or debris was given. Table 6 lists the conditions of the manholes before cleaning efforts were started.

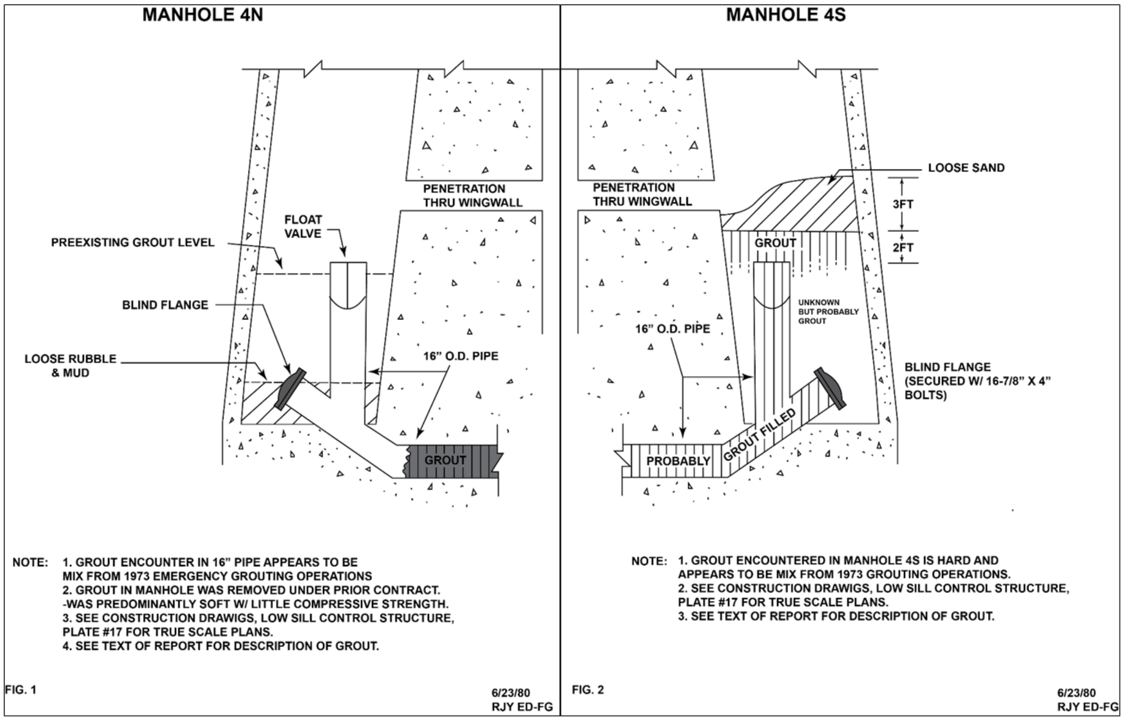
Cleaning of the manholes was completed in October and November 1974, and resulted in the opening of manholes 3S, 3N, 5S, and 5N and opening of a number of drainage pipes around 5S and 5N. Remote-sensing piezometers were to be installed in the stilling basin to monitor uplift pressures in the stilling basin (USACE 1974). Following this initial cleanout effort, there were ongoing problems with some of the other manholes. USACE (1976) discusses cleanout efforts at manholes 1N, 1S, 4N and some of the connecting pipes that were initially filled with grout.

Table 6. Conditions of manholes in structure after flooding in 1973 (USACE 1974).

Manhole	Elev. Top Man.	Elev. Bott. Man.	Depth to Muck	Elev. Top Muck	Depth to Water	Elev. Top Water	Depth of Muck	Remarks
1-S	+55.0	0.0	47.2	+7.8	17.4	+37.6	7.8	Diesel oil
2-S	+55.0	0.0	53.2	+1.8	33.9	+21.1	1.8	Strong smell of diesel
3-S	+40.0	-11.3	47.3	-7.3	17.8	+22.2	4.0	Muck with OR-5 and OR-23 at bottom
4-S	+40.0	-5.0	32.2	+7.8	17.8	+22.2	12.8	Hard grout—OR-23
5-S	+40.0	-10.0	45.2	-5.2	17.8	+22.2	4.8	Muck Mt1. No grout
1-N	+55.0	0.0	52.2	+2.8	35.2	+19.8	2.8	Diesel oil
2-N	+55.0	0.0	51.0	+4.0	34.3	+20.1	4.0	Diesel oil
3-N	+40.0	-11.3	36.9	+3.1	21.9	+18.1	14.4	Muck
4-N	+40.0	-5.0	28.4	+11.6	21.8	+18.2	16.6	Muck with hard material @ +6.5'
5-N	+40.0	-10.0	47.7	-7.7	21.6	+18.4	2.3	Muck

Problems with grout blocking more of the drainage structures continued with ongoing remediation efforts. USACE (1980) notes problems with manholes 4N and 4S related to being plugged with grout. Drawings of the conditions in the manholes were included in the report to illustrate the problem and are shown in Figure 12.

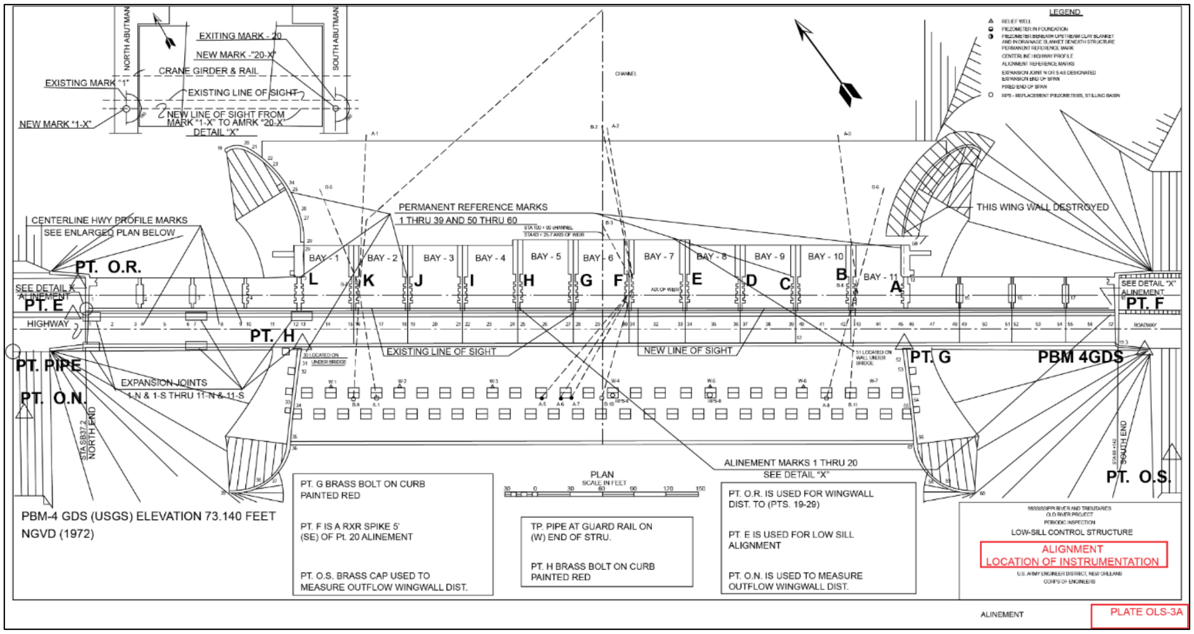
Figure 12. Illustration of grout blocking manhole 4N (left) and 4S (right) (USACE 1980).



4.5 Settlement

Settlement reference markers can be found on the bridge abutments, main structure, and the wing walls. A total of 60 reference markers were placed in 1971 (USACE 2014). Twenty alignment marks are located on the structure as shown on Figure 13. A memorandum, “2014 Low Sill Structure Instrumentation Report No. 19” (USACE 2014), states that the settlement analysis that had been completed indicated that most of the settlement occurred before 1971 (Kaufman and Sherman 1971) and that there was no longer significant vertical movement. Also, alignment analysis indicated that there was also no significant horizontal movement and that the structure was stable (USACE 2014).

Figure 13. Locations of alignment markers (USACE 2014).



5 Flood of 1973

A wet winter and subsequent flood in the spring of 1973 resulted in Mississippi River flows of 2,024,000 ft^3/s (project design flood 2,720,000 ft^3/s) and a maximum headwater elevation of 61.6 ft MSL and tailwater elevation of 59.3 ft (USACE 1975). As a result of this flooding event, an eddy developed at the south upstream wing wall, which caused a collapse of a major portion of the wing wall on 12 April 1973. Surveys were conducted on the scour hole that indicated a lateral extent between gates 5–11 to a maximum depth of approximately -55 ft below MSL near gate 11 (Wilson 1978). Additionally, a majority of the concrete apron in the inflow channel was removed. A timeline of the scour activities in 1973 is in Table 7.

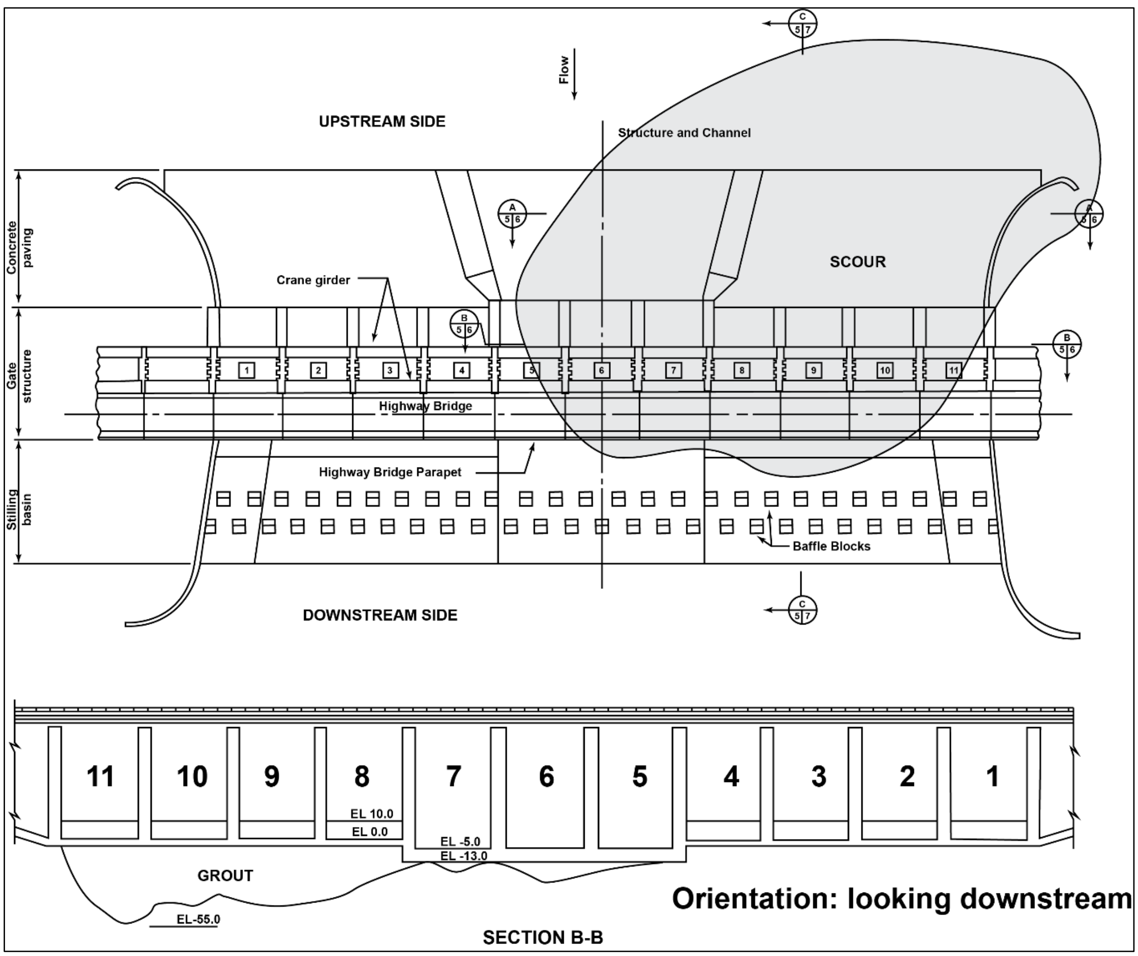
Table 7. Timeline of the 1973 scour events.

Date	Event	Remarks
12 Apr 1973	Monolith A of left approach wing wall parted caused by scour	Rock dike construction initiated in the vicinity of threatened wing wall
14 Apr 1973	Monolith A slowly sinks and disappears; monolith B and C sank later in the evening followed by monoliths D and C. Headwater elevation 59.7 ft and tailwater elevation 54.9 ft were recorded during this time.	Upon notification of the wing wall failure, the Overbank structure was opened to relieve pressure on Low Sill. Following wing wall failure, the rock dike construction was redirected to provide a rock dike to replace the collapsed wing wall; this effort was completed 7 May 1973.
5 May 1973	A preliminary survey of the scour hole was conducted using the Low Sill gantry crane; strong currents prevented use of a boat. The results of the survey indicated that the scour hole in the inflow channel was 55 ft deep and ranged from 200 ft upstream of the base slab and extended from gate bay 5 to 11; the hole was approximately 320 ft wide; plan view of the scour is shown in Figures 5 and 14.	—
9-31 May 1973	Rock fill placed in the scour hole using a derrick barge.	—
Sep 1973 to Oct 1973	Exploratory drilling performed to investigate the damage to the foundations of the Low Sill and stilling basin.	Results revealed extensive void beneath the structures leaving the steel H-piles unsupported laterally. The drainage system was largely destroyed. Stilling basin subject to 90% of headwater uplift pressures.
Nov 1973 to Feb 1974	Foundation grouting beneath structure.	—
	Continued exploratory drilling and intermittent grouting to seal remaining voids until September 1974.	—

6 Grouting Repair

Repair efforts began by placing riprap in the scour hole in the forebay area with the additional construction of a rock dike where the wing wall had previously been located (Figure 14). Drilling efforts began to investigate the extent of the scour hole beneath the main structure and the stilling basin. A total of 48 borings were completed, and grout mixes were developed to fill in the void spaces. Grouting repairs were conducted using several mixtures of a low-cement-content and low-bond-strength grout, which would have minimum bonding to the sheet piling (Wilson 1978; Ainsworth 1979; Wilson and McDonald 1981).

Figure 14. Plan view showing extents of 1973 upstream scour (USACE 1975).



The grout mixes had to meet certain criteria as follows (Wilson 1978):

a. A low-strength, nonsanded, medium-cement grout for the specific purpose of incorporating the muck material into the grout

b. A sanded, low-cement grout to be used as a bulk filler in the scour area

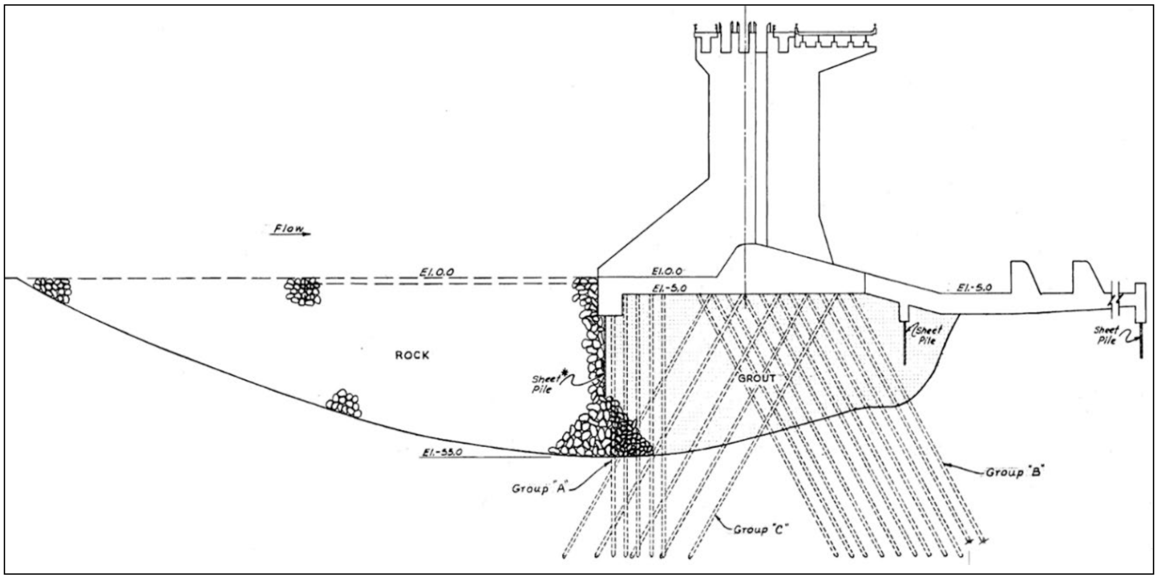
c. A nonsanded, high-cement grout with high compressive strength for use as a cap to bond the underside of the structure sill and the bulk grout and to be resistant to erosion

d. All self-leveling, highly pumpable, and heavy enough to be placed in wet, sluggish environments.

Approximately 30,000 cubic yards of grout were needed to fill the voids (Wilson 1981). Ainsworth (1979) gives a detailed description of the use of instrumentation to monitor the flow, height, and any contaminants in the grout during the repair operations. Check borings were made following the placement of grout and indicated that most of the voids were filled (Wilson and McDonald 1981). Grout also was used in 1971 to repair stilling basin erosion between the end sill wall and downstream row of baffles. A later inspection in 1978 revealed that additional steel plates had been ripped away and that minor erosion had again occurred. Fiber-reinforced concrete was injected below the anchored steel plates for reinforcement (Wilson and McDonald 1981). In 1980, fiber-reinforced grout was used again to repair spalled and eroded concrete on the surface of the stilling basin slab (Wilson and McDonald 1981).

Figure 15 shows section C from Figure 14, which shows the results of the remediation through gate bay 9 following the 1973 flood event. The status of the structure reported in DM-12 (USACE 1975) was that the drainage blanket beneath the bays was either destroyed by the scour process or filled with grout. Only one drainage collector on the stilling basin was functioning properly, and the relief wells were destroyed. Alignment and settlement following grouting operations showed initial elastic movement, but these dissipated, and the structure showed no permanent movement. Four remote telemetry piezometers were installed in line 73 ft downstream of the weir centerline during remediation. Differential heads across the structure were limited to 13 ft.

Figure 15. Cross section showing foundation remediation (reference 14 section C) following the 1973 flood event (USACE 1975).



7 Postscour Stability Analysis

A stability analysis was performed following the 1973 scour event, and the results of the analysis are reported in DM-12 (USACE 1975). The objectives of the stability analysis were as follows:

1. Evaluate the stability of the ORLSS and Overbank structures for existing and predicted future design cases.
2. Evaluate the stability of the structures for existing and predicted future operating conditions and develop allowable operating curves.
3. Evaluate alternatives for further rehabilitation and/or replacement.
4. Address the question of risk presently involved in operation of the complex.

The objectives in numbers 1 and 2 state that the analysis was performed on predicted future design and operating conditions. Because of unforeseen changes in the Atchafalaya River basin resulting from normal operations of the ORCC and ORLSS, the tailwater conditions were updated. The new design hydraulic conditions anticipated are shown in Table 8. The important assumptions made for the DM-12 analysis pertain to the uplift conditions beneath the structure and the performance of the grout installed following the scour incident.

Table 8. Updated design hydraulic loadings (USACE 1975).

Gate Setting	Headwater Elevation (ft MSL)	Min. Tailwater Elevation (ft MSL)	Differential Head (ft)
Closed	35	0	35
Closed	23	29.5	-6.5
Closed	23	33.8	-10.8
3 center open	54	20	34
3 center open, 1 outside gate open each side	67	35.5	31.5
3 center open, 1 outside gate open each side	NA	NA	NA
Two gates closed on one side	62	45.7	16.3
All open	62	49	13
All open	67	50.7	16.3

Because of the partial loss of the upstream impervious blanket for half of the structure, it was assumed that full headwater uplift would be experienced on the upstream edge of the foundation. It was also assumed that there would be no reduction in uplift pressures beneath the structure because of loss of the drainage system. The application of these assumptions was applied for three cases:

1. Full uplift from headwater applied across the base of the structure
2. Full uplift from headwater applied across the grout-soil contact
3. Full uplift from tailwater applied across the base of the structure.

The purpose of the grout installed beneath the structure was to fill the scour hole, provide lateral support to the piles, and provide an impervious barrier between the forebay and stilling basin. Two assumptions were made in the numerical analysis regarding the grout; the first of which is that the grout did not bond to the piles and only provided lateral support. The second assumption was that the grout bonded to the piles, and an additional compressive load was applied to the piles.

The modeling software used in this analysis was "Indeterminate Pile Analysis—3 Dimensions" by Matrix. The program was stated to be for two- and three-dimensional analysis of pile groups based on the matrix derivation by the Saul and Hrennikoff theory (USACE 1975). The program assumed a rigid pile cap, elastic behavior of the system, and the superposition of the effects of six degrees of freedom. In the DM-12 (USACE 1975) analysis, pile tops were assumed pinned to the slab base, and the pile was pinned to the grout (where the grout was considered in place). The results of the analysis yielded pile axial loads, transverse loads, and pile cap deflections and rotations. The foundation response was modeled using the modulus of horizontal subgrade reaction. It was assumed that the subgrade modulus parameter for the soil and grout was homogeneous and, therefore, a constant modulus was assumed along the length of the pile. The analysis was conducted for a weir monolith and low-flow monolith. Table 9 shows the cases modeled in the DM-12 analysis for both the low-flow and weir monoliths. Cases 6 through 9 in Table 9 represent the probable future loading conditions.

Table 9. Cases modeled in DM-12 for the low-flow and weir monoliths (USACE 1975).

Case	Part	Gates (open or closed)	HW (ft MSL)	TW (ft MSL)	Com. Grout Load on Piles	Uplift	Uplift Location	Hor. Grout Load	Highway Live Load	Crane Live Load	Group A Piles
I	A	closed	67	40	N	HW	slab base	N	Y	N	Y
	B	closed	67	40	Y	HW	grout base	Y	N	N	Y
	C	closed	67	40	Y	TW	slab base	Y	Y	Y	Y
	D	closed	67	40	N	HW	slab base	N	N	N	N
II	A	1 per monolith closed	67	40	N	HW	slab base	N	Y	N	Y
	B	2 per monolith closed	67	40	Y	HW	grout base	Y	N	N	Y
	C	3 per monolith closed	67	40	Y	TW	slab base	Y	Y	Y	Y
	D	4 per monolith closed	67	40	N	HW	slab base	N	N	N	N
III	A	open	67	40	N	HW	slab base	N	Y	N	Y
	B	open	67	40	Y	HW	grout base	Y	N	N	Y
	C	open	67	40	Y	TW	slab base	Y	Y	Y	Y
IV	A	closed	35	0	N	HW	slab base	N	Y	N	Y
	B	closed	35	0	Y	HW	grout base	Y	N	N	Y
	C	closed	35	0	Y	TW	slab base	Y	Y	Y	Y
V	A	closed	23	38	N	HW	slab base	N	Y	N	Y
	B	closed	23	38	Y	HW	grout base	Y	N	N	Y
	C	closed	23	38	Y	TW	slab base	Y	Y	Y	Y
VI		closed	70	51.3	N	HW	slab base	N	Y	N	Y
VII		closed	62.5	40.8	N	HW	slab base	N	Y	N	Y
VIII		open	62.5	40.8	Y	HW	slab base	N	Y	N	Y
IX		dewatering	0	0	Y	NA	NA	N	Y	Y	Y

HW=Headwater; TW=Tailwater; NA=Not applicable

The stability criteria used to judge whether the results indicated structural stability or failure were the following:

1. Tension in either group A or B piles
2. Tension load greater than 80 kips in group C piles
3. Compression load greater than 200 kips in either group A, B, or C piles
4. Axial and lateral loads such that the combined stress formula exceeds a value of one.

The results of the analysis for the weir monolith showed that most cases failed for one of the listed stability criteria. That is, of the 21 cases modeled, the weir was deemed unstable for 16 of them. The only cases that proved stable were those regarding the original design cases. The results of the analysis on the low-flow monolith resulted in 11 cases that were unstable. The additional compressive load on the piles caused by the grout was less of an issue for this monolith because of the shallow nature of the scour hole at these monoliths.

8 Conclusions

ORLSS is a pile-founded, reinforced concrete structure with 11 44-ft-wide steel gate bays separated by concrete monoliths with an overall length of 566.0 ft between two concrete wing walls designed to pass 350,000 ft^3/s of water (700,000 ft^3/s when used in conjunction with the Overbank structure). It includes a seepage filter, drainage system, upstream concrete apron, and downstream stilling basin. After construction of ORLSS and until the Flood of 1973, the issues were mainly related to hydraulic performance and river navigation, such as the differential hydraulic head when gates were open and vessel diversion toward ORLSS.

The geological foundation of the ORCC area chiefly consists of natural levee, backswamp, point bar, and abandoned channel deposits. The clean sands are typically encountered at an elevation of -50.0 to -75.0 ft MSL that extend down to an elevation of about -120.0 ft MSL to the Miocene-Pliocene clays. The cross sections A-A′ and B-B′ (Figure 6) of ORLSS, along with past research (Saucier 1969), suggest that ORLSS is underlain by alternating silty and sandy strata that are coincident of a point bar depositional environment as described in the Site Geology section of this document. The grouted scour area shown from A-A′ does not represent the extent of the scour hole surveyed down to an elevation of -65.0 ft (Wilson 1978). This would suggest that a contact of the scoured foundation with the clean sands might occur at the deeper portions of the scour based on the estimates from the scour survey (Wilson 1978) and the top of clean sands (USACE 1954). There is a possibility that the scour eroded the top of the substratum sands, which would have implications in terms of underseepage. In respect to geophysical surveys, the site geology at ORLSS can be differentiated using various geophysical techniques, such as electrical resistivity. However, clay-rich zones, such as backswamp deposits, tend to attenuate geophysical soundings resulting in relatively lower depth imaging. Point bar deposits can also prove difficult to image because of the lateral variation in lithology. It is worth noting, though, that past geophysical surveys in alluvial terrain and the ORCC, specifically, have proven effective (Murphy 1977). This assessment of the subsurface conditions at ORLSS, described in this technical note, provides a better understanding of the geologic properties that will be used to prepare future geophysical collections.

Instrumentation at the site began with the construction of a dewatering system that consisted of seepage blankets, relief wells, drainage system, settlement reference marks, alignment reference marks, and Type A and B piezometers. This was done to monitor both the seepage and uplift pressures in the foundation of ORLSS as well as identify any early signs of differential settlement of the foundation into the underlying soils. The scour hole that occurred in 1973 resulted in the loss of part of the foundation and dewatering system. Most of the damage occurred in the vicinity of the left descending upstream wing wall. Severe damage resulted to portions of the drainage and pressure relief systems as well as to several piezometers in both the main structure and in the stilling basin (USACE 1974, 1975).

Settlement reference marker analysis revealed that most of the settlement of the structure occurred prior to 1971, with no significant vertical movement afterwards or since that time. Hydrostatic pressures are still currently being monitored by piezometers at ORLSS.

The 1973 flood developed an eddy behind the southeast upstream wing wall and, on 12 April 1973, the major portion of this wing wall collapsed. Surveys in the forebay area indicated that a large scour hole had developed in front of gate bays 8, 9, 10, and 11 to a maximum depth of 65.0 ft. Because this scour hole extended 29.0 ft below the bottom of the sheet pile beneath the structure, a detailed boring program was conducted by MVN to investigate the foundation beneath the structure and the stilling basin. The borings indicated a cavity beneath the structure and stilling basin in the vicinity of gate bays 6 through 11 that varied in depth from a maximum of 52.2 ft in gate bay 11 to 1.8 ft in gate bay 7. Furthermore, this has not been the only documented case of scouring activity in the ORCC area (i.e., the scour near the south bank of the inflow channel in 1967 as well as downstream on the Mississippi River near river mile 310 after the 2011 flood [Knox and Latrubesse 2016]). This demonstrates the need for continuing investigation on the river dynamics and foundation of ORLSS and surrounding area. In addition, there exists a likelihood that riprap placement in the inflow channel and replacement of the south wing wall could have changed the flow dynamics.

The scour hole that occurred in 1973 was filled in by riprap and approximately 30,000 cubic yards of level-seeking, low-cement grout. A stability analysis was performed in 1975 on ORLSS in response to the 1973 scour event to predict future design and operating conditions.

However, it was assumed that (1) the grout did not bond to the piles and only provides lateral support and (2) that the grout bonded to the piles and an additional compressive load was applied to the piles. In addition, it was assumed that the subgrade modulus parameter for the soil and grout was homogeneous and, therefore, a constant modulus was assumed along the length of the pile (USACE 1975).

The information provided here will assist in determining the best technical approach for the characterization and analysis of the condition of the grouted foundation that was the result of the scour hole that occurred during the 1973 flood. This literature synthesis provides a better understanding of the foundation conditions present at ORLSS and will supplement future research dedicated to the subsurface assessment of the foundation.

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Abbreviations

Term	Definition
CH	Miocene-Pliocene clays
DEM	Digital elevation model
ERDC	Engineer Research and Development Center
GP	Poorly graded gravel
ML	Silt
MVN	New Orleans District
NED	National Elevation Dataset
ORCC	Old River Control Complex
ORLSS	Old River Low Sill Structure
PL	Public law
SM	Silty sand
SP	Poorly graded sand
USACE	US Army Corps of Engineers
USCS	Unified Soil Classification System
USGS	US Geological Survey

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14. ABSTRACT
The US Army Corps of Engineers (USACE), New Orleans District (MVN), tasked the US Army Engineer and Research Development Center (ERDC) with assessing the condition of a grouted scour hole located at the southeast wall of the Old River Low Sill Structure (ORLSS) at the Old River Control Complex (ORCC) using noninvasive techniques, such as geophysical surveys and physical models. This special report (SR) combines a scientific literature synthesis of previous research with further geologic interpretation as a first step in the overall task assigned by MVN. The results discussed in this SR will be used to inform the interpretation of geophysical surveys, construction of physical models, and input for the slope stability analyses.

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