Overview of Geotechnical Issues Involved in the Olmsted Locks and Dam Project

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SYNOPSIS The Olmsted Locks and Dam project consists of new twin 1200-foot locks and dam located on the Ohio River near its confluence with the Mississippi River. The site of the locks and dam presents some unique geotechnical challenges. A major landslide on the Illinois bank, artesian groundwater conditions, significant seismic considerations, and the design of the pile foundation and the construction cofferdams presented major challenges to the design engineers. The geotechnical investigations and the design considerations involved in this project are discussed.

INTRODUCTION

The Olmsted Locks and Dam project consists of new locks and a dam located on the Ohio River about 17 miles above its confluence with the Mississippi River. The Olmsted project will replace two existing locks and dams, Nos. 52 and 53. The project consists of twin 1200-foot locks, a navigable pass section 2200 feet long, and a 426-foot section of fixed weir. The navigable pass section of the dam will have hydraulically-operated wicket gates which will be lowered to the river bottom during high water. Navigation can then proceed over the gates without having to use the locks. An artist's rendering of the project is shown in Figure 1.

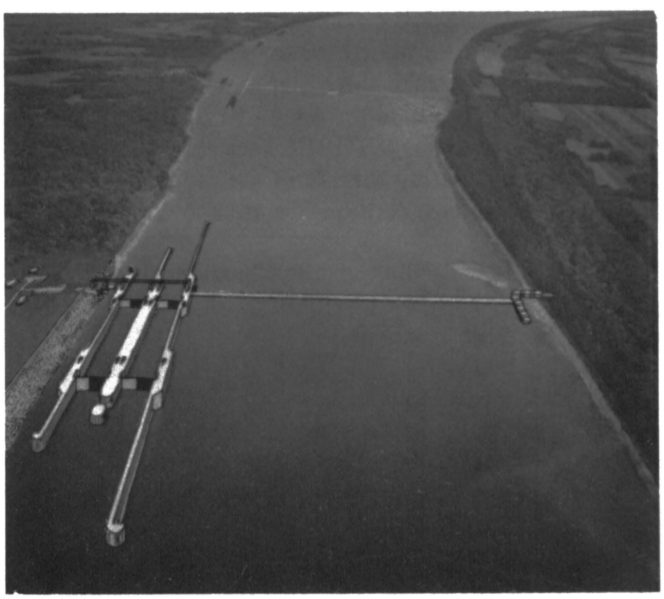

Figure 1. Artist's Rendering of the Olmsted Locks and Dam Project

The project will involve several structures unlike any the Corps has built before. The lock will be a monolithic W-frame structure, and the dam will consist of hydraulically operated wicket gates which will fold down to the river bottom during high water to allow navigation over them. Both of these structures represent new technology. The project will have a total cost over 1 billion dollars, and as such is one of the largest civil works project underway in the Corps of Engineers.

Geotechnical investigations for the project were extensive. Subsurface investigation consisted of SPT borings, undisturbed Denison barrel and Shelby tube sampling, cone penetration testing, and downhole and crosshole geophysical testing. Full scale pump testing and pile testing was also conducted. The site was instrumented with inclinometers, movement monuments, and standpipe and vibrating wire piezometers.

The geotechnical issues involved in the design and construction of this project made for a very large challenge. Alone, each of the issues were significant, but when considered altogether, they became magnified by their effects upon one another. For example, stabilization of a landslide affected the construction of the lock cofferdam, because of construction sequences and the effects of the slide excavation on the stability of the cofferdam. Construction dewatering was a major consideration of all aspects of the design. All of the geotechnical issues became like a jigsaw puzzle, with each piece needing to fit in with every other piece.

Discussions of the various design considerations follow.

GEOLOGIC CONDITIONS

A generalized geologic column of the Illinois bank soils is shown in Figure 2. The soils which are most significant are the Porters Creek clay and the McNairy Zone I. The Porters Creek clay is highly plastic clay with approximately 45% montmorillonite. When dry, it is very brittle and capable of absorbing large quantities of water. When wet, it is slick and has very low strength. The McNairy I is an irregularly layered soil consisting of dense fine sand and clay. The consistency of the clay is variable. Because of the layering, this soil exhibits anisotropic strength properties. A cross section through the Illinois bank is shown in Figure 3.

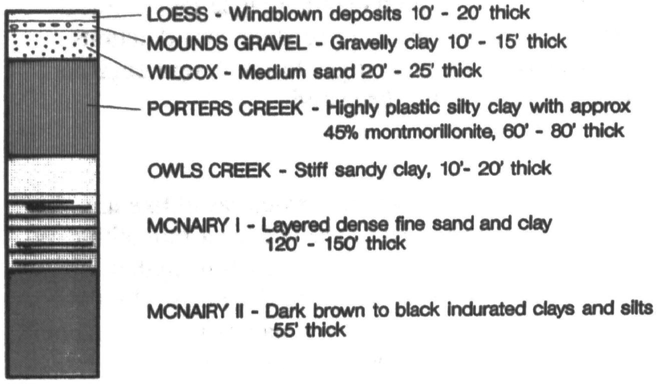

Figure 2. Generalized Geologic Column.

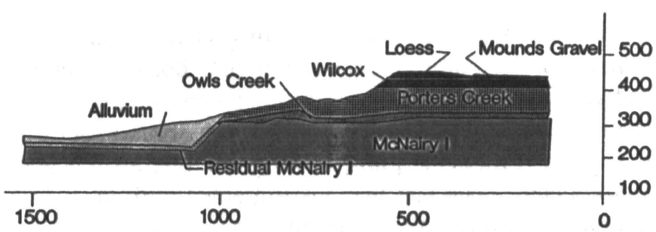

Figure 3. Illinois Bank Cross Section.

The drained shear strengths of the Illinois bank materials are shown in the following table.

TABLE I

Material	Unit Weight	Friction Angle	Cohesion
Loess	122 pcf	23	0
Wilcox	122 pcf	24	0
Porters Creek	105 pcf	8	0
Owls Creek	117 pcf	18	0
Alluvium	117 pcf	10	0
McNairy I	118 pcf		
Horizontal		10	0
Inclined		25	0
McNairy II	126 pcf	24	0

In the river, the geologic conditions are much simpler. The soils overlying the McNairy I have been eroded. A layer of sand and gravel alluvium has been deposited on the eroded surface of the McNairy I.

SEISMIC CONDITIONS

The Olmsted site is located just 50 miles away from New Madrid, Missouri. In the winter of 1811 and 1812, the region was rocked by a series of earthquakes centered in New Madrid so severe that they are regarded as the worst earthquakes ever to have occurred in North America. Between December 1811 and March 1812, roughly 1800 shocks and aftershocks rocked the region.

A seismic study was done for this project by the Corps of Engineers' Waterways Experiment Station. Two design earthquakes were determined for the project. The maximum credible earthquake (MCE) is a major New Madrid earthquake, comparable to the 1811-1812 events. The operating basis earthquake (OBE) is an earthquake that is expected to occur during the life of the project. The characteristics of these two earthquakes are shown in Table II.

TABLE II. Earthquake Characteristics

Maximum Credible Earthquake -

Richter Magnitude = 8+
Intensity = Modified Mercalli X
Mean Acceleration = 1.12 g
Mean Velocity = 80 cm/sec
Duration = 20 seconds

Operating Basis Earthquake -

Richter Magnitude = 6.0
Intensity = Modified Mercalli VIII
Mean Acceleration = 0.44 g
Mean Velocity = 24 cm/sec
Duration = 7 seconds

Since this is a navigation structure and does not impound a permanent pool, failure of this structure poses very little chance of the loss of life. Therefore, the project is being designed for the operating basis earthquake, and is being checked for the maximum credible earthquake. While the OBE is smaller than the MCE, it is still a very significant earthquake.

LIQUEFACTION ANALYSIS

Because the construction of the locks and dam will require removal of most of the alluvium, the concern about liquefaction centered primarily on the McNairy I. In order to evaluate the liquefaction potential of the McNairy I, Seed's simplified method was used (Seed and Idriss, 1983). This method correlates known values of cyclic stress ratio exhibited by sands under actual earthquake shaking conditions with some readily measurable in-situ characteristics of the sands. The Standard Penetration Test (SPT) is a good indicator of cyclic loading liquefaction characteristics. Since the SPT N value measured in the field reflects the influence of the soil properties and the effective confining pressures, the influence of confining pressure is eliminated by using an N value normalized to an effective overburden pressure of 1 ton per square foot.

The SPT test itself must be standardized so that meaningful comparisons can be made. The test should be made using a rope and cathead system with two turns of the rope around the cathead, drilling mud to support the sides of the hole, a relatively small diameter hole, and the penetration resistance measured over the range of 6 to 18 inches penetration into the ground.

The normalized SPT data from the site in question are compared to normalized SPT data obtained from sites that have experienced earthquake shaking. Figure 4 shows an empirical chart from Seed and Idriss for predicting liquefaction potential.

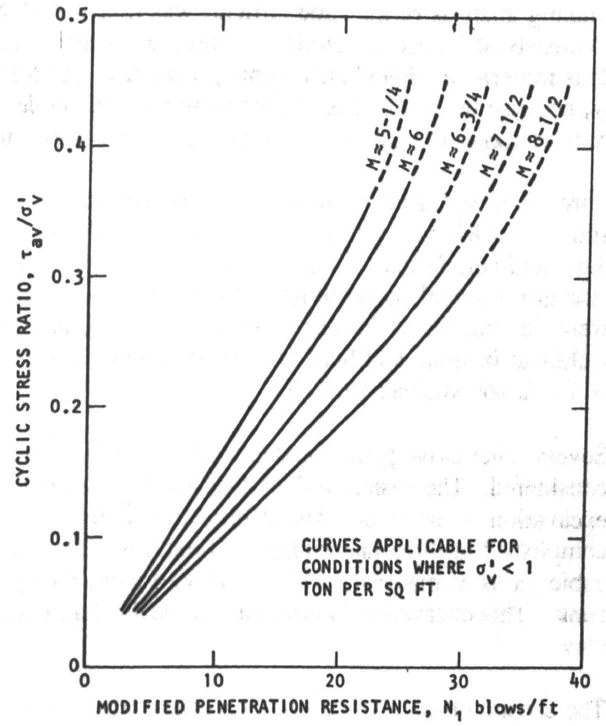

Figure 4. Chart for evaluation of liquefaction potential for sands for different magnitude earthquakes.

It was found that the McNairy I did not have significant potential for liquefaction.

ILLINOIS BANK SLIDE

An ancient slide on the Illinois bank was discovered during the initial subsurface exploration for the project. After a period of drought, river levels reached near-record lows, and renewed movements of the slide were observed.

Investigations of the slide with inclinometers and movement monuments found that there were actually two slides. There was a slide on the upper bank and a slide on the lower bank. The stabilization measures to be undertaken would have to reflect the existence of two slides instead of a single moving mass. Figure 5 shows these two sliding surfaces.

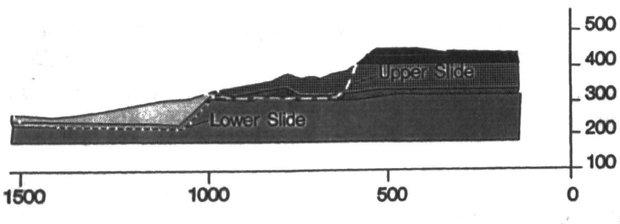

Figure 5. Cross Section of Slide.

Making analysis of the slide difficult was that the McNairy I consists of lenses and laminae of fine sands and clay. This material exhibited anisotropic properties. Additionally, because the sizes of the layers were unpredictable, characterizing the anisotropic properties was complicated.

Through a back-analysis of several cross-sections of the slide using laboratory testing, inclinometer and piezometer data, and correlation of index properties to residual shear strength, residual shear strengths for the McNairy Zone I were determined. This work was done at Virginia Polytechnical Institute and State University under the direction of Professor Michael Duncan.

Several alternative plans for stabilizing the slide were considered. The easiest and least expensive was mass excavation of the slope. The stabilization will be accomplished in three major phases. First, about 1 million cubic yards of material will be excavated from the upper bank. This excavation is primarily in the Porters Creek clay.

The second phase involves the excavation of the lower bank. This excavation will be done within the cofferdam that will be built for the construction of the locks. A large quantity of the material to be excavated is under water much of the year. Additionally, the wells that will be used to dewater the foundation will serve to provide stability for the cut bank. The dewatering wells will be installed along the Illinois bank, then the cofferdam will be pumped out. Excavation of the lower bank will proceed within the cofferdam. All of the lower bank material cannot be excavated, because it would unload the toe of the slope too much. Wick drains will be installed in the remaining material to accelerate consolidation of the sediments.

The third phase involves the placement of 1.5 cubic yards of fill on the lower slope. This fill will serve to load the toe of the slope, preventing movements of the lower slope. This fill will be placed against the lock wall, and will also serve as the service mound for operation of the locks.

PILE FOUNDATION

The design of the pile foundation proved to be one of the least complicated aspects of this project. In 1988, pile load testing was conducted on the river bank near the lock and dam site. The testing program had several objectives:

- To determine the depths to which H-piles and sheet piles could be driven.
- To determine what types of pile driving equipment is most suitable for the site.
- To determine compressive and tensile load capacity.
- To determine horizontal modulus of subgrade reaction.
- To assess the value of the Pile Driving Analyzer (PDA) for bearing capacity determination.
- To measure sound levels generated by pile driving equipment.

A total of twenty piles were driven for the testing program. Of these, three were testing in compression, four were tested in tension, and two were tested in lateral loading. The remainder of the piles served as reaction piles for the testing frame. All of the piles driven were tested with the PDA.

The Pile Driving Analyzer consists of piezoelectric accelerometers and bolt-on strain gages that are attached to the pile near its top. They remain there while the pile is being driven. The signals from the instruments are transmitted to the Pile Driving Analyzer, which conditions and calibrates these signals and computes average pile force and velocity. From the pile force and velocity, the PDA computes the bearing capacity.

Dynamic testing of the piles was done under three different conditions. All of the piles driven with an impact hammer were tested as they were being driven. Those piles which were driven to refusal with a vibratory hammer were tested with the PDA when continuation of the driving was attempted with an impact hammer. And, selected piles were restruck after a period of time to allow the setup to develop.

The pile driving conditions were good. It was found that the piles developed a great deal of setup after driving. These conditions produced high capacity piles without tough driving conditions. It was found that if driving were interrupted for even a short period of time, it was very difficult to restart the piles. Therefore, the specifications will call for uninterrupted driving of piles.

The piles will be designed with an allowable bearing capacity of 413 kips and an allowable tensile capacity of 200 kips. It was found that the piles derive about 90% of their capacity from skin friction. With the variable nature of the McNairy I, it could not be anticipated what the actual load transfer mechanism would be. This was determined through the use of strain gages on the piles, a telltale on the pile tip, and through the analyses of the PDA results.

Because of the high setup with time, it was found that PDA testing at initial driving would greatly underestimate the pile capacity. However, restrike testing showed good agreement between PDA restrike testing and static compression load testing.

To apply the results of the compression and tension testing to our site, we used Decourt's analysis (Decourt, 1989). This is a simple and reliable method of predicting the bearing capacity of piles based on the SPT. The adhesion between the pile and the soil is given by

$$q_s = \frac{\overline{N_L}}{3} + 1\ (t/m^2)$$

where $\overline{N_L}$ is the average of N along the shaft of the pile. This correlation is independent of the soil type and the position of the groundwater level.

Point bearing pressure (q_p) is given by

$$q_p = K\overline{N_p}$$

where $\overline{N_p}$ is the average of N close to the pile tip. K is given in Table III and is a function of the soil and type of pile.

TABLE III

K as a Function of Soil and Type of Pile

SOIL	Driven Piles t/m^2	Bored Piles t/m^2
Clays	12	10
Silty Sands (Residual Soils)	20	12
Clayey Sands (Residual Soils)	25	14
Sands	40	20

Dr. Jean-Louis Briaud of Texas A&M University reviewed the data from the pile tests and recommended this approach. Decourt's analysis was used to make a preliminary determination of pile capacity at the test site. Then the results of the pile testing were compared to the determination of pile capacity from Decourt. It was found that the results from Decourt's method correlated well with the actual pile test results. Therefore, the method was then used with the N values measured at the lock and dam site to predict the pile capacity at that location.

Confirmatory testing of production piles will be done during the initial stages of production pile driving. The confirmatory test program will incorporate the PDA, so that a correlation between the capacity at initial drive and the capacity with time can be made. Also, it is felt that the PDA will be valuable as a construction quality control device.

LOCK COFFERDAM

The cofferdam for the construction of the lock will consist of 51 cells and connecting arcs arranged in a pattern that is roughly rectangular. The area inside the cofferdam is 46 acres. The cells are 63 feet in diameter, and are constructed with PS 27.5 sheets that are 110 feet long. The cells will be filled with sand and gravel from the river near the Kentucky bank. The cells will require approximately 1.3 million cubic yards of fill. Figure 6 shows a plan of the lock cofferdam.

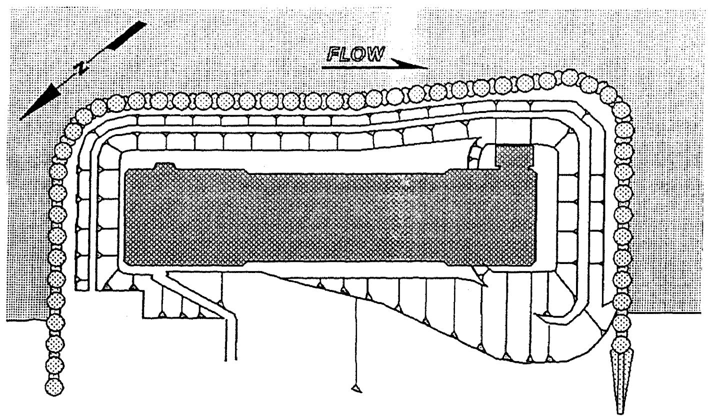

Figure 6. Plan of Lock Cofferdam

Because rock is very deep at the site, the cells could not be founded on rock. Therefore, the cells will be founded in the McNairy Zone I. In order to limit the depths to which the sheet piles had to be driven to reach suitable founding material, the alignment of the cofferdam will be pre-excavated before the cells are constructed. Pile driving will be limited to no more than 20 feet. The excavation will be backfilled after the cells are driven.

The cells were designed based on adequacy to prevent failure in sliding, overturning, deep-seated stability, and for structural stability. The cells were designed according to the Corps of Engineers design manual EM 1110-2-2503, *Design of Sheet Pile Cellular Structures*. This manual gives criteria for many different failure modes, such as overturning, rotation, vertical shear resistance, pullout of outboard sheets, and penetration of inboard sheets. However, the manual also says not to base the design on these failure modes, but to employ them as sensitivity checks only.

The preliminary and final cofferdam designs were reviewed by Mr. Paul Swatek. Many of his comments were related to the fact that cofferdams do not fail as a rigid block. This meant that the calculations for rotation, vertical shear, pullout of the outboard sheets, penetration of the inboard sheets, and overturning have no basis in the actual behavior of cellular cofferdams. Sliding and interlock tension were found to be the critical design parameters for the cofferdam.

Because of the existing slide on the Illinois bank, it was assumed that there is a zone of material at the top of the McNairy I which possess residual strengths. In order to have the sliding stability required, it was necessary to have a large berm within the cofferdam. Then the stability of the berm itself became an issue. It was flattened and enlarged. By the time all of the requirements of sliding, deep-seated stability, and berm stability were satisfied, the berm was very large. When the cell was checked for overturning, it found that the net moment was in the wrong direction!

The berm is 70 feet above the bottom of the cells, and will be constructed prior to pumping out the cofferdam. The berm has two benches, one at the top, and an intermediate bench 20 feet below the top of the berm. Figure 7 shows a cross section of a typical cofferdam cell.

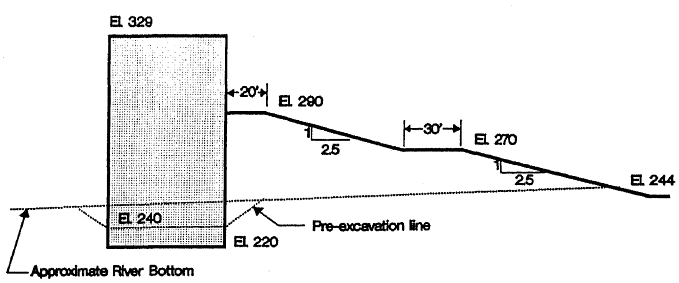

Figure 7. Cross Section of Typical Cofferdam Cell

In order to satisfy the requirements of interlock tension, it was required that the water level within the cells be reduced and remain at a low level for the duration of the construction of the lock. In order to achieve this, wells will installed from the tops of the cells into the cell fill in order to drain the cell fill.

DEWATERING

To evaluate the permeability of the McNairy Zone I, a full scale pumping test was conducted. This test was conducted in the river from a barge. The location of the test was at the upstream end of the locks. A 6-inch well was installed from the barge in the McNairy Zone I below the river. The well was installed in a 16-inch hole drilled with a cable-tool rig. Four pneumatic piezometers were also installed in the McNairy I below the river. The test was conducted somewhat close to the Illinois bank so that standpipe piezometers on the shore could be monitored during the pumping and rebound. The well was pumped continuously for 40 hours. After pumping ceased, the piezometers were monitored for an additional 21 hours. A horizontal permeability for the McNairy I was determined from this test.

It was found that although the McNairy I has relatively high pressures, higher than the river at times, it produces relatively little flow. Therefore, the objective of the dewatering system would be to reduce pressures rather than produce high flows.

The dewatering system for the cofferdam will have four major components:

- Wells installed into the McNairy I
- Wells installed into the McNairy I and II
- Wells in the cofferdam cells
- Predrainage system

The wells in the McNairy I are designed to lower pressures in that zone to a level five feet below the deepest part of the excavation. Some of the wells in the McNairy I will be drilled to penetrate the McNairy II. The artesian pressures that exist in that zone must be reduced to prevent blowout of the bottom of the excavation. Wells will also be installed in the cofferdam cells to reduce interlock tension to acceptable levels. The predrainage system will be used to drain the sand and gravel alluvium within the cofferdam so that it can be excavated easier.

ENVIRONMENTAL CONSIDERATIONS

Environmental considerations also had an effect on the geotechnical considerations. On the Kentucky side of the river, there is a wildlife management area. Geese, turkey, ducks and bald eagles nest or winter there. Precautions must be taken to minimize disturbance of the waterfowl and eagle population. Therefore, sound generated during construction is a major consideration. Sound levels from pile driving equipment will be restricted and will be monitored during driving. If necessary, muffling devices will be used on the hammers. Pile driving will be done during daylight hours only.

Lighting for the site will not be directed toward the Kentucky shore in order to minimize disturbance.

Another environmental consideration is a bed of mussels about 1.5 miles downstream from the site. This mussel bed is the largest, most diverse and most populous bed of mussels on the Ohio River. There are several endangered species of mussels in this bed. Mussels are filter feeders, and when the water becomes too turbid, they close. If they stay closed too long, they die.

Fill for the cofferdam cells will be obtained by dredging from an area near the Kentucky shore. One of the major concerns was that the dredging and transport of the fill would produce turbidity levels which would adversely affect the mussel population.

Sampling of the material to be used as fill was done not only to evaluate its suitability for this use, but also to determine of there would be any adverse effects on the mussels from the dredging of this fill. A study was undertaken by the Corps' Waterways Experiment Station (WES) using the properties of the material and the flow patterns before and during construction to determine if there would be increased turbidity for increased periods of time at the site of the mussel beds. WES produced a mathematical of the site and the river downstream of the site. It was found that dredging of this fill will not adversely affect the mussels.

ACKNOWLEDGEMENTS

One of the most satisfying things about working on this project has been the opportunity to meet so many highly regarded professionals in the civil engineering field. The author wishes to acknowledge Dr. J. Michael Duncan, Dr. G Wayne Clough, Dr. Robert Pyke, Dr. Jean-Louis Briaud, Dr. Ben Gerwick, Mr. Charles Mansur, Mr. Paul Swatek, Dr. Richard Barksdale, and the talented engineers and scientists at the Waterways Experiment Station. These people, in their capacity as consultants to this project, have made significant contributions to its development. Meeting and working with these people has truly been an opportunity of a lifetime.

Also, the author wishes to acknowledge the tremendous talent, motivation, and sheer hard work of the Louisville District's Olmsted geotechnical team. It has been a pleasure to work with these capable engineers.

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