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Construction of 10' Diameter Shafts for the Olmsted Approach Walls Project

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The $106 million Olmsted Approach Walls Project, completed in the spring of 2004 by Massman Construction Co., Kansas City, MO, is one phase of a new $1.4 billion Corps of Engineers locks and dam facility located at Mile 964 on the Ohio River, approximately 17 miles (27.4 km) upstream of its mouth at the Mississippi River. When completed, the Olmsted Locks and Dam (Figure 1) facility will replace two aging locks and dams (Locks and Dams 52 & 53) built in the 1920's, and produce an estimated $600 million annual economic benefit to the nation through reduced lockage time and operation and maintenance costs.

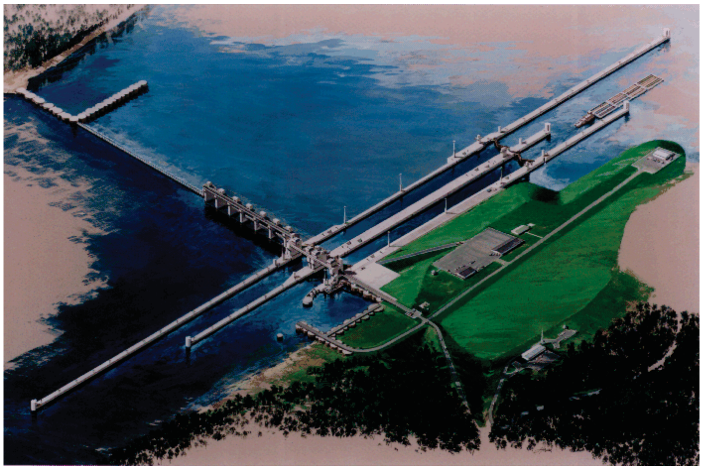

Figure 1 - Rendering of the Olmsted Locks & Dam

PROJECT OVERVIEW

The Olmsted Approach Walls Project consisted of "in-the-wet" construction of four fixed nose pier structures, four floating concrete approach walls, one fixed wall and related underwater excavation, grading and stone placement. Over 12,000 tons (11,000 tonnes) of structural steel and pre-cast concrete elements, up to 335 tons (304 tonnes) in weight, were safely erected during the project.

To support these massive structures, the Louisville District of the U.S. Army Corps of Engineers teamed with INCA Engineers, Inc., Bellevue, WA to design thirty seven 10' (3m) diameter shafts (cast-in-place steel-shell piles) capable of maintaining elastic behavior below ground while resisting barge impact loads, extreme head-on collisions from runaway tows, and environmental forces from river current, wind, and seismic events.

The shafts (typically 150' (45.7m) in length) were founded 100' (30m) into the riverbed through alluvium, a very consolidated fine sand and clay Cretaceous soil, and finally into a hydrothermally altered rock.

Thirty seven shafts were installed under open river conditions at the site, a group of four shafts at each nose pier-pylon location (Figure 2), and 21 at the fixed land wall. Due to difficulty in achieving the required casing tip elevation the team developed a design modification that was successfully implemented for 27 of the shafts.

Concurrent with the work at Olmsted, eleven pontoon segments for the four floating approach walls were constructed at a graving yard in Paducah, KY, developed by **Massman Construction Company** specifically for the project. While the pontoon geometry varied, the pontoons averaged 350' (107m) in length, 40' (12.2m) in width and weighed 4,500 tons (4,082 tonnes). The longest of the four walls, at 1,700' (518m) was comprised of five interconnected pontoons.

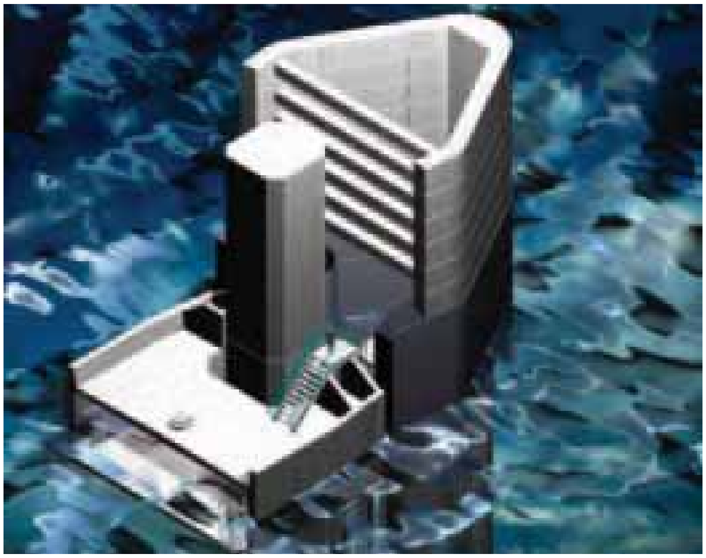

Figure 2 -Rendering of a Nose Pier and Pylon

SUBSURFACE CONDITIONS

In general, the foundation materials consist of 2 to 20 ft. (0.6 to 6m) of alluvial sands, 50 ft. (15m) of very dense fine sand and interbedded clay of the McNairy formation, and 30 ft. (9m) of hydrothermally altered rock of the Fort Payne formation. The alluvial deposits near the Illinois bank consist mainly of reworked silts, clays and occasional fine sand lenses, some of which are likely a result of colluvial materials from landslides. These materials are very soft to stiff, and are normally consolidated. Further from the shoreline, the alluvium changes to a loose to medium dense poorly graded sand.

Below the alluvium, the McNairy (a Cretaceous-aged soil deposit) consists of approximately 80% very fine to fine dense sands with the remainder being interbedded layers of stiff clays and silts. This formation ranges from 40 ft. (12m) in the river channel, to 120 ft. (36.6m) thick on the Illinois bank. The soil is very stiff with Standard Penetration Test N-values ranging from 40 to over 100. The groundwater within the McNairy sands is pressurized with heads up to 10 ft. (3m) above the river water surface. Self-Boring Pressuremeter tests conducted in the McNairy indicate that the magnitude of effective in-situ horizontal stress is very high.

It was determined that the Paleozoic rock formation directly underlying McNairy is a leached and silicified Mississippian formation, known as the Fort Payne Formation (John Nelson, personal communication). The Fort Payne is normally a dark-colored, siliceous limestone that contains bands of dark chert. In the Olmsted area, nearly all of the carbonate minerals have been leached or dissolved away and/or replaced with silica, leaving a dark brown silt-like rock that contains occasional bands of hard chert.

DRILLED SHAFT DESIGN

Loads

The large lateral loads from possible barge impact and earthquakes governed the design of the drilled shaft foundations. Site specific ground motions were developed by Geomatrix Consultants. The Operating Basis Earthquake (OBE) with a return period of 144 years has a peak ground acceleration at the base rock of 0.065g. The Maximum Design Earthquake (MDE) with a return period of 1,000 years has a peak baserock acceleration of 0.37g. Site response analysis was then performed to develop the freefield ground surface motions for seismic design. For barge impact, lateral forces up to 4000 kips (17,800 kN) were applied to the nose piers. Smaller forces ranging from 300 to 1,000 kips (1,335 kN to 4,448 kN) were used for impacts on the floating pontoons.

Soil Response

Since the performance of full scale lateral load tests are difficult and very expensive for larger diameter shafts, self boring pressuremeter (SBPM) tests were performed at each nose pier location and used to develop P-Y curves for design. The tests were performed by Dr. Jean Benoit (University of New Hampshire) and the drilling by FMSM Engineers (Lexington, KY). The non-linear load-displacement (P-Y) curves for the soils adjacent to the embedded portion of the shafts were developed using the recommendations by the American Petroleum Institute (API, 1993). The results of the SBPM tests and lateral load tests on H-piles performed for the locks' foundation were then used to refine the P-Y curves. Since the zones of influence of the closely spaced drilled shafts overlap, the load transfer characteristics along a shaft are affected by adjacent shafts. The interaction among drilled shafts was accounted for by using P-multipliers to scale the P-Y curves. These multipliers were developed based on an empirical correlation developed by Dunnavant and O'Neill (1986).

Design Method

The final design was completed by separating the structure into a linear superstructure model and a nonlinear substructure model. The Superstructure was modeled with GTSTRUDL and the Substructure stiffness determined with LPILE. To verify the validity of this approach for the seismic design, a three dimensional soil-structure interaction analysis was performed using SASSI (Lysmer et. al, 1981) on one nose pier/pylon group. The shaft embedment depths were determined to ensure long pile behavior for all load conditions. Flexible long pile behavior was required to minimize permanent set in the pile after removal of loading. The structural moment capacity of the shafts was determined by considering the composite strength of the concrete, reinforcing steel, and the permanent casings. The drilled shaft casings are 10.0 ft. (3m) in diameter and approximately 150 ft. (46m) in length. The permanent casings range in thickness from 3/4 to 2 1/8 inches (19mm to 54mm) and were designed to extend to the bottom of the shafts. In the maximum moment areas the reinforcing steel consists of 72 #18 (57mm) bars.

SHAFT CONSTRUCTION

The drilled shafts were designed to resist large lateral loads from barge impact as well as seismic loading. Based on these requirements, specific construction constraints were required to ensure lateral resistance was not lost during installation of the casings. This was complicated by the very stiff soils, high lateral stresses, artesian pressures, and the presence of fine sand layers in the McNairy formation.

Casing Fabrication

In order to eliminate delays due to casing field splices, and radiographic weld testing, the casings (Figure 3) were delivered to the site full length. Eaton Metal Products fabricated the casings, weighing up to 130 tons, at their facility in Pocatello, ID, and transported them via rail to Memphis, TN, where they were transferred to hopper barges and towed to the site.

Figure 3 – Typical Shaft Casing

Template

The Ohio River at Olmsted fluctuates in elevation seasonally as much as 40 feet (12.2m) with high velocity flows of up to 6 feet/second (1.83m/second) throughout the year. Prior to casing installation, a 100 ton (91 tonnes), two-level structural steel pile template, designed to limit the casing deflection from the river current to 1/16” (1.6mm), was installed at each nose pier location. This template enabled the casings to be installed within 1-1/2” (38mm) of their plan location (less than half of the 4” (102mm) specified tolerance).

Casing Installation and Clean Out

The casings were driven into the substrata using a HPSI 2000 vibratory hammer with a rated eccentric moment of 21,445 in-lbs (2,423 Nm) (see Figure 4). This hammer, the largest of its kind, was designed and built specifically for the project by **Hydraulic Power Systems, Inc**. In order to obtain optimal hammer performance, the operating frequency and amplitude of the hammer was varied during driving.

Figure 4 – HPSI 2000 Vibratory Hammer

During the course of driving, periodic clean out of the inside of the casing was required. This was accomplished with various cleanout buckets and brushes. The pile top drilling system was developed by the **Steven M. Hain Co**. (Figure 5). This system allowed for relatively quick cycles for cleanout and driving operations. Water levels inside the casings were required to be maintained above or near the river level at all times. When material was to be loosened or removed within 10 feet of the tip, slurry was required to replace the water in the casing. The slurry consisted of AQUAGEL bentonite with a density maintained between 66 and 75 pcf (1,057 to 1,201 kg /m^3). It was later determined that the insitu soils in the cuttings produced a stable slurry mixture and was able to maintain a fluid density near 80 pcf without adding bentonite.

Installation Issues

Driving began at the upstream riverside nose pier in late September of 2000. It was soon determined that the casings could be successfully installed by driving them continuously to the top of the Fort Payne formation, cleaning out the casing and adding slurry, then drilling ahead of the casing and driving in 6 ft. (1.8m) increments. The eight upstream drilled shafts were advanced to their final tip elevation using this procedure.

Figure 5 – Steven M. Hain Drill with 18-ft bucket

However, casing refusal above the final tip elevation (near the top of the Fort Payne formation) was encountered during driving the casings for the downstream riverside nose pier. Several attempts were made to advance the shafts including using the more powerful (26,000 in-lb.or 2,938 Nm) tandem King Kong **APE** Vibratory Hammer. See Figure 6. The skin friction on the casing had "set-up" from the time initial driving halted until cleanout occurred. This made it impossible to advance the casings even after 6 feet (1.8m) of material was drilled out below the tips.

Figure 6 – APE Tandem King Kong Hammer

The design tip elevation was 159 ft. (48.5m). The four casings for the downstream riverside nose pier and pylon refused between el. 179 ft.(54.5m) and 188 ft. (57.3m). A design modification was implemented for these shafts. The tops of the casings were cut off at the design top elevation. An uncased socket was then drilled to the original tip elevation of 159 ft. (48.5m). Additional reinforcing steel was added to compensate for the absence of the casing. Number 18 (57mm) reinforcing bars placed in 4 bar bundles were required to achieve the required moment capacity. To reduce the risk of early refusal of the remaining casings and having the thickened high moment capacity section of the casings in the wrong place, the remaining shafts were modified by reducing the casing embedment depths and increasing the length of the reinforcement.

During the installation of the lower land wall shaft 3, the casing refused in the soil well above the top of the Fort Payne formation. The casing could not be advanced even after drilling below the casing tip. Before revised work procedures to advance the casing could be implemented, the river rose above the work platform and templates. Even though the casing was full of a drilling mud, the walls of the excavation collapsed undermining the casing which then fell about 10 ft. (3m) under its own weight. To remediate this situation the casing was backfilled and removed, then reinstalled in its proper alignment. To restore soil strength and confinement around the casing, a compaction grouting program, designed by the Terracon Engineering Group, was performed by The Judy Company.

Reinforcement

The length and diameter of the required reinforcing cage, along with the large number of #18 (57mm) bars, made it impractical to construct horizontally. The cages, weighing up to 70 tons, were built vertically directly in the open shafts. Number 6 hoop bars were positioned adjacent to the shaft and vertical reinforcement consisting of #18 bars (57mm) were fixed to a circular lifting ring and spliced by threaded couplers. The vertical reinforcement was then lowered entirely into the shaft and slowly raised to add the hoop bars. As the completed cage was lowered back into the shaft casing, eight steel access tubes for integrity testing were installed at equal spacing within the cage. See Figure 7 for typical reinforcement installation. The tubes extended above the shaft and were fitted with removable caps.

Concrete

Concrete placement was made by use of a floating batch plant. The floating batch plant was located on a 35 ft. by 200 ft. (10.7m by 61m) hopper barge and had a capacity of 75 cubic yards per hour. Concrete from the batch plant was delivered to a tremie hopper via a Schwing 105 ft, (32m) placing boom and a 5-inch (127mm) diameter concrete pump. Slump for the concrete placed was 6 to 9 inches (150 to 225mm). Logs were kept of slump loss,

Figure 7 – Typical Reinforcing Steel Cage

concrete temperature, and volume discharged. The duration of concrete placements varied, but generally took about 10 hours per shaft.

Following successful placement of the concrete, Crosshole Sonic Logging (CSL) was performed by Olson Engineering to verify the shaft integrity. The CSL tests are able to detect concrete defects such as voids, soil intrusions, defect seams, water-filled zones and low strength concrete. Minor defects found in two shafts were cored for verification then repaired by grouting.

SUMMARY AND CONCLUSIONS

- Construction of an innovative shaft design for large lateral loads in difficult subsurface conditions proved to be challenging.
- Soil behavior during driving of large diameter shafts by vibratory methods is not well understood in the engineering community, and therefore warrants special considerations. The skin friction developed by some of the casings could not be overcome by the largest vibratory hammers available.
- Designers and contractors must be flexible in order to achieve project success for these types of foundations. Careful field observations are often the key to making these proper revisions.
- The innovative construction methods used by Massman and their subcontractors, and the teamwork with the Army Corps of Engineers, and INCA Engineers provided the flexibility needed to successfully overcome unforeseen soil conditions on the project. Figure 8 shows final installation floating approach wall structures.

Figure 8 – Final Installation of the Floating Approach Wall

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