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Seventh International Conference on Case Histories in Geotechnical Engineering
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STABILIZATION OF EXISTING SHEET PILE CELL IN THE OHIO RIVER

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ABSTRACT

An existing 40 foot diameter sheet pile cell (cell) was used to support a dry fly ash loading platform at an electrical power generating facility on the Ohio River. The cell was designed and constructed in 1986 for support of a coal barge unloading crane but was never put into service. The cell has leaned riverward several inches in the years after construction. Stability analysis indicated a less than adequate overturning factor of safety without any additional loads. The addition of the loading platform could result in an overturning failure.

A stabilizing system which provided a horizontal stabilizing force of 750,000 lbs. was developed, designed and subsequently installed. The system included a tension belt steel channel bent about the cell circumference connected to two-1¾inch diameter high strength tension bars at each end. The tension bars extended 100 feet landward and were anchored to two pile caps. The pile caps were supported on a tripod of three-120 ton working load HP14x117 piles driven on a 4:1 batter. The horizontal stabilizing force for the cell originates from four-200 foot long rock anchors (20 foot long bonded length) installed in each pile cap at a nominal angle of 45-degrees from the horizontal.

The tension belt channel elongated 3 inches during anchor proof testing while the strand anchors elongated approximately 18 inches. It was necessary to test the anchors in pairs to maintain a balanced loading condition on the tension belt channel, requiring adjusting and balancing the load in the anchors and tension bars continuously to maintain the pile caps in a neutral position.

Each anchor proof-test required six hydraulic cylinders and four power packs operated simultaneously at different pressures. After each pair of anchors was proof-tested, the anchors were de-stressed until all pairs had been proof-tested. Then the anchors were re-loaded in pairs and locked off at 50% of the 120 ton design load.

INTRODUCTION

A regional power company in southwestern Indiana recently completed the installation of a nearly one of a kind dry fly ash tubular conveying system (this installation is the second of this type constructed in the US) with a construction cost of approximately $14M. The dry fly ash is transported by pipeline from the generating facility to a storage silo located about 2,000 feet from a barge fly ash loading facility on the Ohio River in West Franklin, Indiana, just down-stream from Evansville, Indiana. In addition to the storage silo, the dry fly ash conveying system includes a 2,000 foot long elevated tubular conveyer and the fly ash loading platform. Figures 1 and 2 illustrate the existing cell, loading platform and stabilizing measures. The conveyor (see Fig. 3 and 4) discharges at the fly ash loading platform (see Fig. 5) which is located atop the existing 40 foot diameter cell that was constructed in the Ohio River.

The sheet pile cell was designed and constructed in 1986, originally for support of a crane to unload coal barges for the generating facility. The cell was constructed of 80 foot long L.B. Foster 400J Type UNA straight web sheet-piles weighing 27 psf. This section is approximately equivalent to the U.S. Steel PS 28 sheet-pile section. The top of the cell sheeting was set at El. 365 with the top-of-bank grade on the landward side at El. 348 prior to the cell construction. The site grade landward of the river top-of-bank was raised to El. 364.5 following the completion of the cell construction. Normal pool for the Ohio River in this reach is at El. 342 with a 100-year flood level in this area at El. 375. Prior to the construction of the cell the existing grade in the area ranged from El. 337 on the riverside to El. 348 on the landside. Farther to the project north, in the over-bank flood plain area,

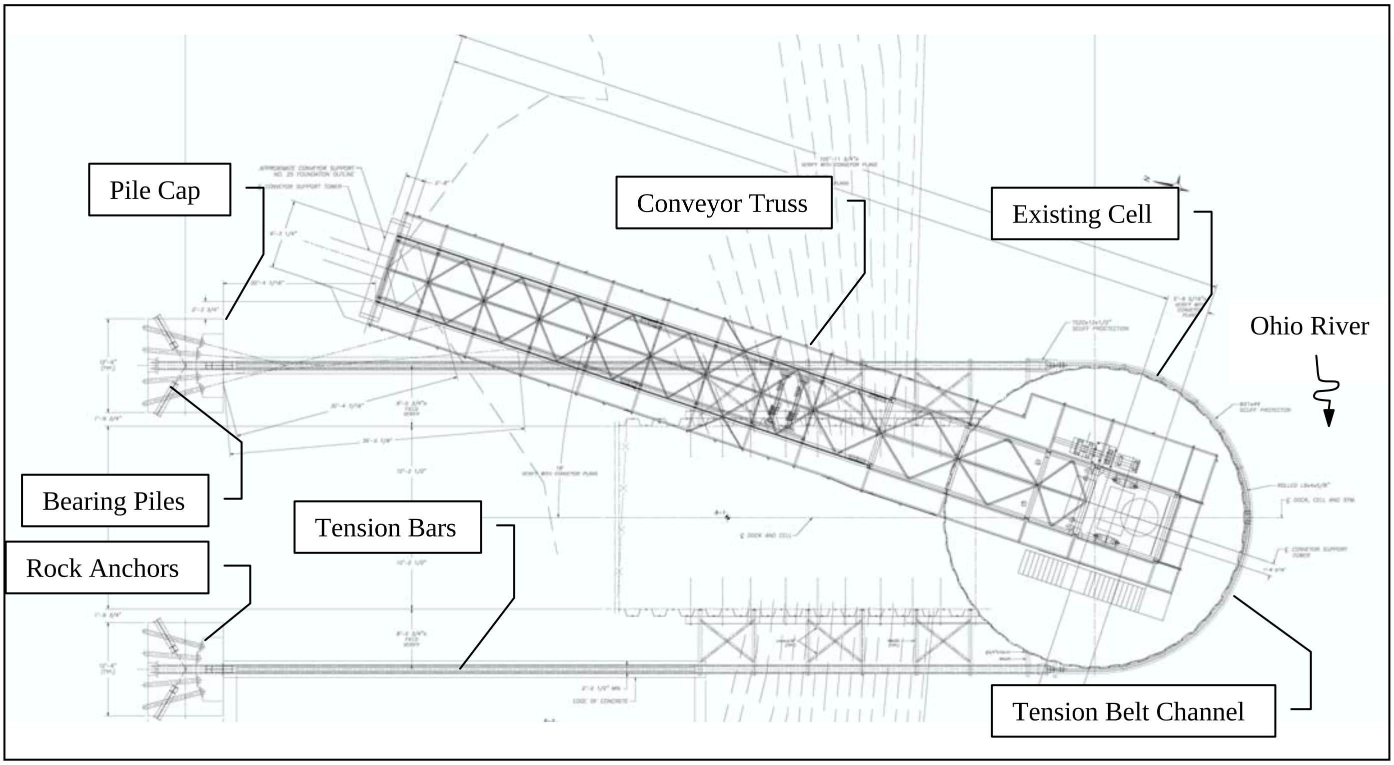

Fig. 1. Plan of existing cell, loading platform and stabilizing measures.

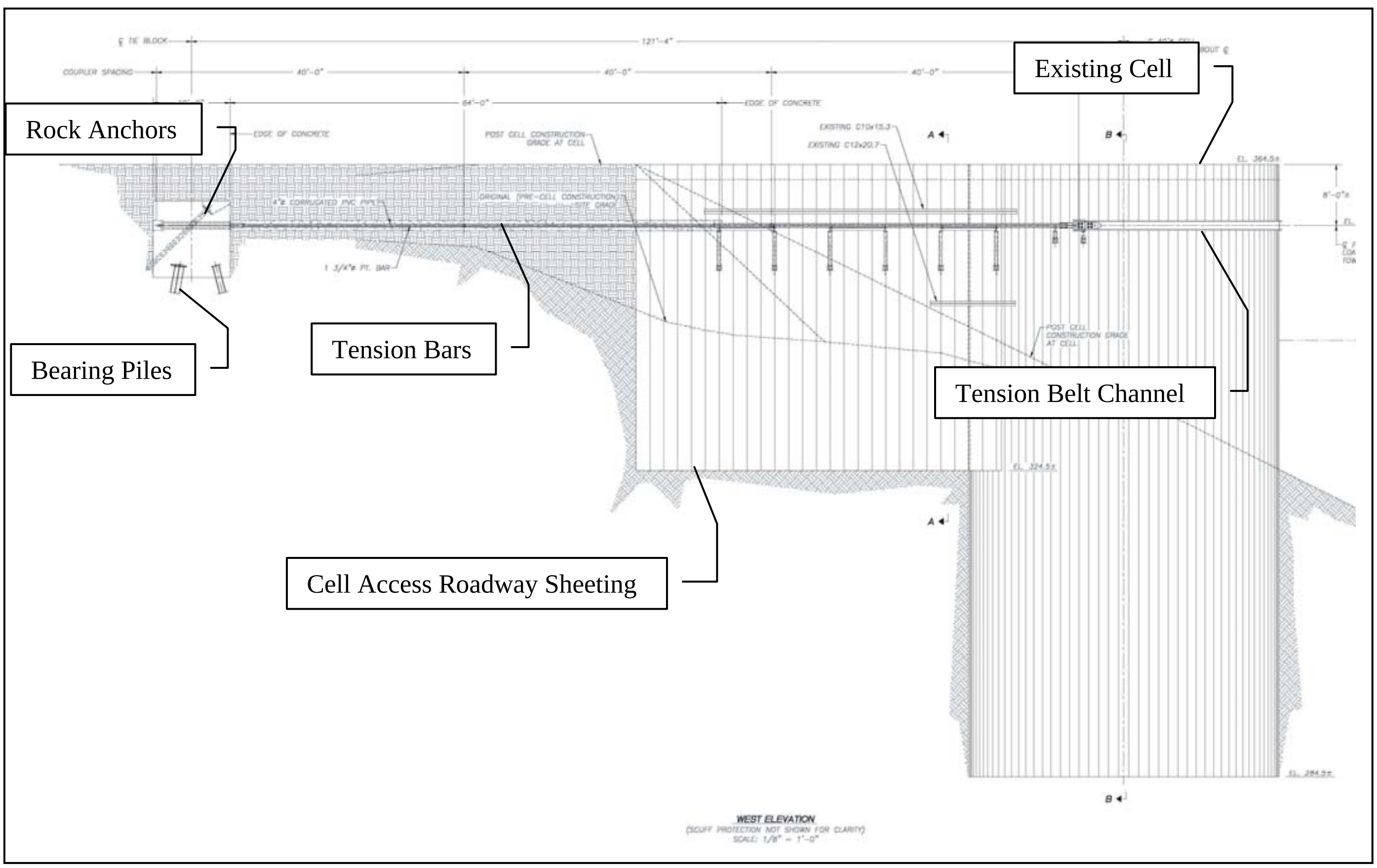

Fig 2. West elevation of existing cell.

the existing grade was generally between El. 358 and 362 before the earthwork associated with the cell construction. Grade was raised in this area with the cell construction to approximately El. 364.5.

Although the cell was constructed, it was never put into service for coal barge unloading. In the years since the cell was constructed, the fill within the cell and between the cross-tied access roadway sheet piles had settled almost a foot (see Fig. 6) and the cell had leaned out riverward several inches from the access road to the cell.

A static stability analysis of the cell revealed a lower than normal factor of safety against overturning without any additional loads, thus confirming the reason for the tilting of the cell. Construction of the dry fly ash loading platform on the inadequately stable cell could result in an over-turning failure of the cell under certain loading conditions, which would result in major damage to the new conveyor system and would remove the loading platform from service for an extended time.

Fig. 3. Storage Silo and Tubular Conveyor.

GEOTECHNICAL CONDITIONS

Regional Geology

The project site is located in southwestern Indiana at the eastern edge of Posey County. This area is classified as the Wabash Lowland Section of the Southern Hills and Lowland Region Physiographic Province. The Wabash Lowland Section was not glaciated and is flatter than the Boonville Hills Section located to the east. This region is characterized by a broad lowland tract consisting of smooth bedrock hills and wide valleys. This area was subsequently modified and thick deposits of lacustrine outwash and alluvial sediments were deposited during the Pleistocene age. The upland tracts of the Wabash Lowland are rolling plains blanketed by thick deposits of loess (windblown silt) that decreases in thickness east of the Wabash River and north of the Ohio River.

Bedrock at the site is primarily shale of the Pennsylvanian age (Carbondale Group and the McCleansboro Group), but does include sandstone, siltstone, limestone, and coal. The project is located in the bottomlands of the Ohio River which is known as the Wabash Lowlands Section and includes bottomlands along the Ohio River that are underlain by sandy outwash deposits and alluvium. The unconsolidated thickness (i.e., soil thickness) ranges from 100 to 120 feet at the project site.

Fig. 4. Tubular Conveyor.

Soil and Rock Conditions

The general subsurface conditions were investigated by drilling two borings drilled by ATC Associates, which extended to 123 and 156 feet in depth. The soil conditions at the boring locations typically consisted of silty clay fill material extending to approximately 8 foot depth (El. 356). Natural, gray silty clay (CL) or clayey silt (ML or ML-CL) was then encountered to approximately 60 to 70 foot depth (El. 295 to 305) where loose to medium dense brown fine or fine to medium sand was encountered. The sand became coarser and denser with depth grading into dense fine to coarse sand and gravel. Gray shale bedrock was encountered at depths of 112 to 118 feet (El. 246 to 251). The shale extended to 131 feet depth (El. 233) where siltstone was encountered, extending to the 156 foot explored depth of the deepest boring.

The fill materials were generally stiff to hard with estimated unconfined compression strengths by calibrated hand penetrometer ranging from 4 to 4½+ tsf. Natural moisture contents in the fill ranged from about 9 to 20 percent.

The natural cohesive soils were soft to medium, although some very soft areas were noted, with calibrated hand penetrometer test results ranging from 2¼ tsf to less than ¼ tsf, with the strength typically decreasing with depth. The natural moisture content of these soils was generally in the range of 25 to 30 percent with several in excess of 30 percent. The granular soils encountered below approximately El. 320 to 288 (deeper closer to the river) generally exhibited Standard Penetration Resistances (N-Values) in excess of 30 blows per foot (bpf). These soils were judged typical of the Valley Train glacial deposits commonly found along the larger rivers in the region related to meltwater from glacial activity.

Fig. 5. Loading Platform at cell.

ANALYSIS

Cell Overturning Stability

The static overturning analyses were performed using unfactored loads. The loading combinations provided in Section 2.4 of ASCE 7-05 were used for the overturning analysis. Based on our evaluation of the cell stability, the existing cell appears to be under-designed from an overturning perspective. The normal industry practice is to design a cell of this type with a factor of safety relative to overturning in the range of 3 to 3.5. The factor of safety relative to overturning of this cell based on lateral earth pressure loads and normal pool hydrodynamic loads only (without the imposition of any new fly ash loading platform loads), is approximately 2.3 with a maximum theoretical applied toe bearing pressure of 9,700 psf and a net heel theoretical tension stress of 200 psf from the maximum riverward overturning moment. The maximum theoretical applied bearing pressure would be the bearing pressure under the toe of the cell if it were a geometric solid of the same size and mass as the dimensions of the circular sheet-pile structure.

Fig. 6. Settlement of existing access roadway cap.

When evaluating the cell with regard to overturning in two directions (riverward and downstream), the peak toe and heel stresses are located at the approximate quarter points of the cell circle. For this case, the approximate maximum quarter point toe bearing pressure is 7,200 psf and the minimum heel bearing pressure is 2,200 psf. Although the factor of safety relative to overturning is less than the desired level, prior to construction of the loading platform, the cell likely had compression across the entire base area due to secondary effects (such as uplift capacity of sheet pile on the heel side of the structure as well as the stabilizing effect of the connection to the access roadway support sheeting) that were not specifically included in the static overturning analysis.

The addition of load to the cell from the proposed loading platform and tower reduced the factor of safety relative to overturning and increased the toe bearing pressure while decreasing the heel pressure. Most of the eight loading conditions evaluated resulted in net theoretical tension on the heel of the cell. In actuality, it is unlikely that the cell would experience a net tension on the heel of the cell under the loading configurations considered due to secondary effects of soil friction on the sheeting, however, it does serve to show the relative reduction in the safety of the cell with increasing shear loads and/or overturning moments. Load Combination No. 8, (60% of Dead Load, 70% of Earthquake Load and 100% of Earth and Water Pressure) from the above noted ASCE publication, is the most severe loading condition. With regard to Load Combination No. 8, we estimate the factor of safety relative to overturning is approximately 1.5 with a maximum theoretical toe pressure of 19,800 psf and a net theoretical heel tension stress of 9,900 psf. In our judgment, this results in an unacceptably low factor of safety relative to overturning and apparent theoretical bearing pressure that would be excessive. Thus, we did not recommend the loading platform and tower be supported on the existing cell unless the stability of the cell and new structure was modified or improved.

REMEDIAL SUPPORT MEASURES

Support System Design

The author, while consulting with ATC Associates, was tasked with developing a means of improving the overturning stability of the cell under the new loading platform. The result was to provide a horizontal stabilizing force of up to 750,000 lbs to the cell through a tension belt steel channel bent about the perimeter of the cell (see Fig. 7).

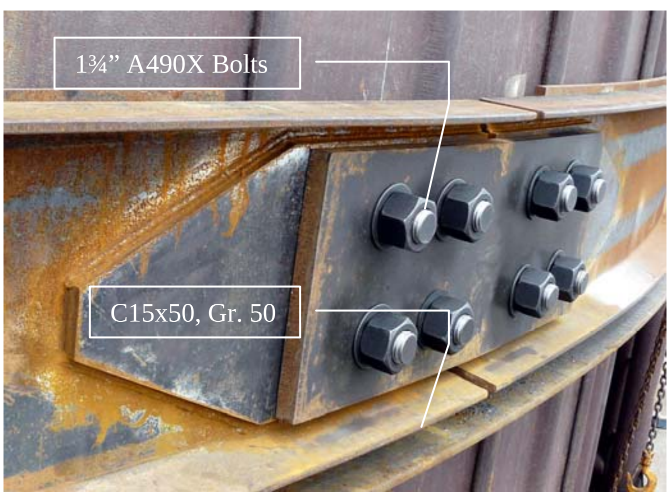

Fig. 7. Tension Belt Channel Splice.

The tension belt channel (a C15x50 section in Grade 50 steel) was connected to two-1¾ inch diameter high strength steel Dywidag Systems Inc. (DSI) tension bars (Grade 150) at each end with a fabricated transition piece (see Fig. 8).

The tension bars then extended approximately 100 feet landward from the cell and were anchored to two pile caps; one at each end of the tension belt channel (see Fig. 9).

The pile caps were supported on a tripod of HP14x117 piles driven on a 4:1 batter by the cell stabilization project General Contractor, JT Crawford, Inc. (Crawford) of Commerce Township, Michigan. The H-piles were driven approximately 120 feet to friction and end bearing through soft clay, wet sand and ultimately weathered shale just above the sound shale bearing for a design working load capacity of 120 tons each.

Fig. 8. Transition from Tension Belt Channel to Tension Bars.

The tension belt channel is protected from damage with a W21x44 wide flange that was bent at the same radius as the cell and installed over the tension belt channel (see Fig. 10).

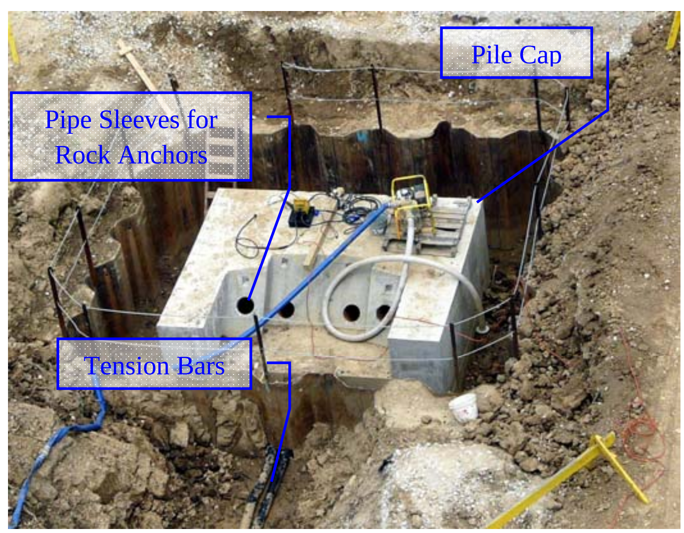

Fig. 9. West pile cap.

The weekly barge traffic at the cell is kept off the channel and protective wide flange with 12 inch square oak timbers that are attached to the cell with clip angles and through bolts (see Fig. 11 and 12).

The tension rods, where exposed riverward of the existing river bank, are subject to damage from debris floating downstream in the river (see Fig. 13 and 14). The highly stressed tension bars were protected from floating debris damage by being placed within a TS6x14 structural tube (see Fig. 15). After stressing of the system was completed, the annular space between the bars and the structural tube was filled with cement grout for further protection from impact damage and for corrosion protection purposes. The tension belt channel to tension bar transition connection was also encased within a protective structural tube and the annular space filled with waterproof grease for corrosion protection (see Fig. 16).

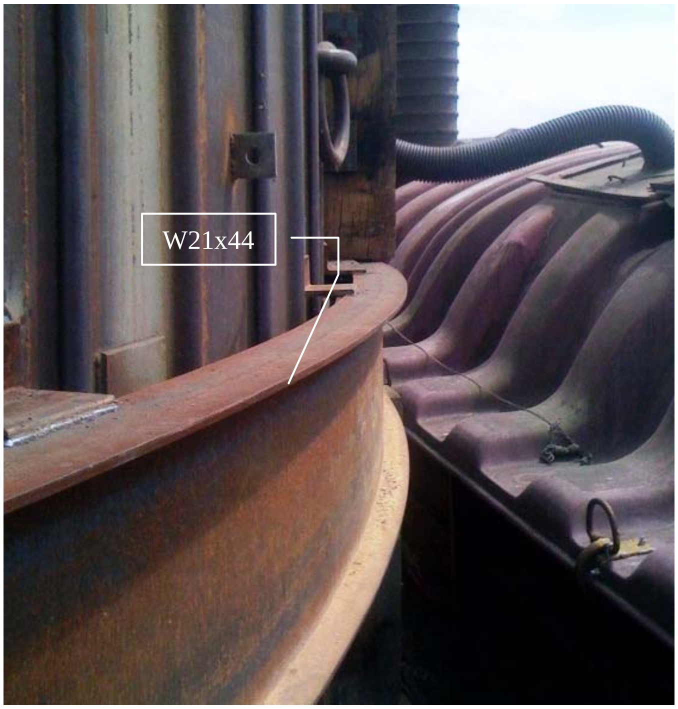

Fig. 10. Protective W21x44 section.

ROCK ANCHOR INSTALLATION

Rock Anchor Drilling

The horizontal stabilizing force for the cell originates from 4 high capacity (120 tons design load each) rock anchors installed in each pile cap at a nominal angle of 45 degrees from the horizontal. The rock anchors were installed by Spartan Specialties, Inc. (Spartan) from Sterling Heights, Michigan. One of the challenges of the rock anchor installation was maintaining an open hole for the installation and grouting of the seven strand high strength DSI tendons (Grade 270 steel). Spartan elected to use permanent 6 inch diameter flush joint casing from the pile cap to the competent bedrock. Casing lengths up to 180 feet were left in the ground at each rock anchor location to assure a quality installation of the rock anchors. A Klemm duplex drill rig was used to install the permanent casing and to drill the bond zone.

Wash rotary methods were used with a counter rotating casing advancer which permitted the drill string and the casing to rotate in opposite directions as the hole was advanced. The anchors were splayed out on the horizontal to prevent problems with group effects of the anchors. The west pile cap anchors were installed at 43^{o} from the horizontal while the east pile cap anchors were installed at 47^{o} from the horizontal. The angles of the two pile caps were varied to prevent interference between the closest anchors as they crossed paths and made their way towards the anchorage over 120 feet below grade and almost 200 feet away.

Fig. 11. Timber fenders.

Fig. 12. Installed timber fender.

Fig. 13. Debris on tension bar protective structural tube.

Fig. 14. Debris at mooring cell upstream of load platform cell.

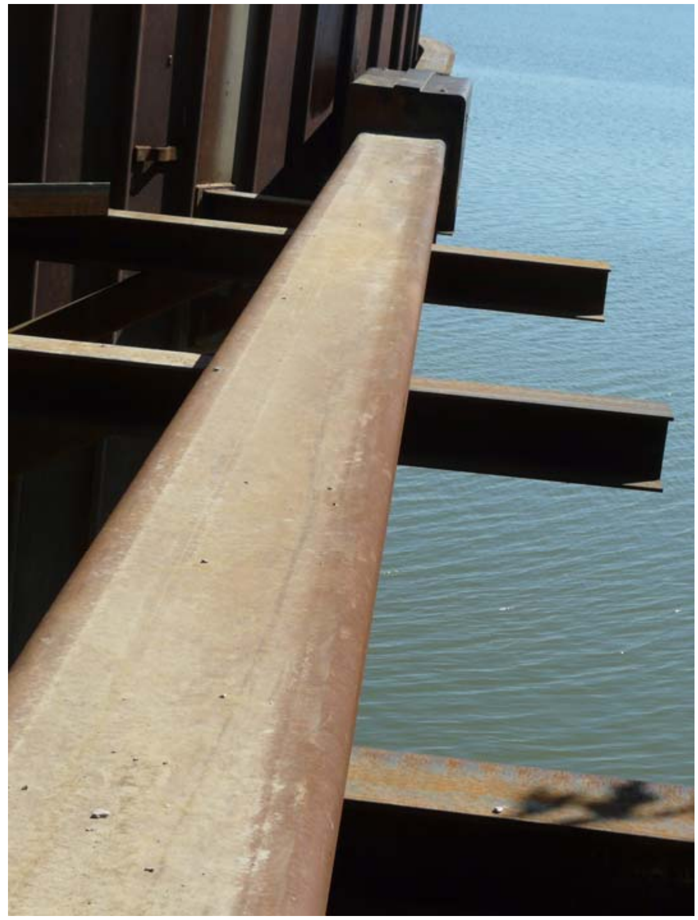

Fig. 15. Protective TS6x14 over tension bars.

Fig. 16. Tension belt channel to tension bar transition protective structural tube.

Anchor Tendon Installation

The bond zone length of the double corrosion protected strand anchors was pre-grouted in the corrugated PVC protective casing by DSI, the tendon manufacturer. Although the pre-grouting of the bond zone makes the tendons more difficult to ship and handle, it provides an increased measure of reliability of the corrosion protection system as the grout encasement of the strand is considered one of the two protection systems (the corrugated PVC casing being the other) in the double corrosion protection required for permanent anchors.

Fig. 17. Klemm duplex drilling system rock anchor drill.

The anchor tendons were suspended from the forks of a high-lift Lull rough-terrain fork lift and lowered into the anchor casing approximately 20 feet at a time (see Fig. 18). The tendons were tied off at the pile cap while the forks were repositioned for the installation of the next section of tendon. Spartan elected to over-drill the boreholes approximately 5 feet to make certain the tip of the tendon would reach the required minimum embedment.

Anchor Grouting

The anchors were grouted in-place with a site-mixed cement grout consisting of Type I cement, potable water and Fox Industries FX-349 (a grout fluidifier and expanding agent). The grouting work was performed in July and August 2010 when temperatures were frequently in the high 90's F and exceeded 105^{o} F on several occasions.

Spartan used a refrigerated semi-trailer to store 2,000 gallons of City of Evansville potable water as well as the bagged cement to maintain the fluidity of the grout and prevent problems with flash setting of grout mixed with ambient temperature water and cement. A mixing water temperature of about 60^{o} F was used to start the mixing to result in grout temperatures in the mid-70's after mixing.

A tremie pipe of ¾ inch Schedule 80 PVC which extended almost 200 feet from the ground surface to the bottom of the 20 foot bond zone was used to place the grout in a continuous operation without interruption for each of the eight rock anchors. Each anchor required between 30 and 35 bags of cement, mixed in five bag batches in a double tub, high shear rotary type mixer. A continuous cavity Moyno style pump was used to pump the grout mix (see Fig. 19).

Fig. 18. Anchor tendon installation.

Anchor Testing

The proof testing of the anchors and loading to the lock-off of 50% of the anchor design load presented some especially challenging issues to the construction team. Due to the length of the tension bars and the use of the high strength steel, as much as 3 inches of elastic movement was predicted for the tension bars and the tension belt channel from the loads induced by the proof-testing of individual anchors. Since most of the resistance to the horizontal component of the rock anchor tension during proof testing was to be provided by the companion pile cap and anchors on the other side of the cell, it was necessary to test the rock anchors in pairs to maintain a balanced loading condition, otherwise, the tension belt channel could shift around the circumference of the cell.

Although the pile tripod system was not designed to resist the full horizontal component of the proof-test load, it would develop a significant horizontal capacity by virtue of the battered piles and the high axial compressive and tension capacity of the piles. Thus, it made for a very "stiff" surface for the rock anchors to react against. On the other hand, the tension tie rods were very flexible given the use of the high strength steel and high tension loads. It was clear that as the rock anchors were load tested, the stiff pile caps would pick up more load than the tension tie rods and would overstress the piles both in tension and compression before the full measure of the tension tie rod capacity was achieved.

Fig. 19. Site mixed grout plant.

To counter this problem, the design-construction team developed a load transfer beam which could be attached to the tension tie rods where they protruded from the rear of the pile cap. The tension on the tie rods was then adjusted with two 100-ton jack cylinders located at each end of each load transfer beam (two cylinders per beam per pile cap). Each pair of these jacks was powered by a single power pack through a manifold system that equalized the load on each tie rod. The cylinders were located within the flanges of the load transfer beam which provided more clearance for the beam in the confined space behind the pile cap to accommodate the elastic stretch of the channel and tension bars and the tightening of the channel around the somewhat irregular cell.

As each load test was performed, a total of six hydraulic jack cylinders (one each for the two anchors being tested – see Fig. 20, and two each for the load transfer beam on each pile cap), four hydraulic power packs (one for each anchor being tested and one for each pair of tie rods being tension adjusted) were operated simultaneously, each individually calibrated and operated at different hydraulic pressures. The tension load in the tie rods was increased simultaneously in all four tension bars as the rock anchor load was increased to keep the pile cap in a neutral position where the horizontal component of the rock anchors equaled the tension in the tie rods on each pile cap.

The rock anchor movements were measured with five inch stroke dial gauges while the movement of the pile caps was tracked with a theodolite reading metric scales to the nearest millimeter attached to each pile cap, all at the same time with a crew of six. The nearly 18 inches of elastic movement of the rock anchor tendons required resetting of the anchor movement dial gage three times during the proof-testing for each anchor as well as during the de-stressing of the anchors after completion of the creep portion of the proof-testing. Things got very busy during the creep stage of the rock anchor tests when anchor movement readings are required every minute as well as adjustments in the pressure in each of the 6 jack cylinders.

Fig. 20. Anchor stressing jack cylinder.

All of the rock anchors passed the creep requirements for the proof-tests and were de-stressed to no load prior to testing the next pair of anchors. After completion of the anchor testing, the anchors were again loaded in pairs to 50% of the design load where they were locked off with the tension load confirmed through lift-off testing. Again, the tension load in the tie rods was increased simultaneously in all four tension bars as the rock anchor load was increased to keep the pile cap in a neutral position where the horizontal component of the rock anchors equaled the tension in the tie rods on each pile cap.

The design of the supplemental support system was to increase the factor of safety against overturning of the cell to well above the normally required levels when exposed to the "normal working loads" (wind, water force, dynamic equipment loads). The rock anchors are designed to be stressed to their design load of 120 tons should the most severe load combination be experienced by the system, excluding earthquake loads. The entire system is designed to yield if the structure is exposed to earthquake loads but still maintains a factor of safety in excess of 1.0 under this circumstance so the cell should not tip over even under the worst set of circumstances.

The challenges presented by this project could not have been met without the ingenuity, cooperation and determination of the all engineers and contractors involved in the design and construction of this unusual and innovative solution to a difficult problem.

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