TEMPORARY CELLULAR COFFERDAM DESIGN, INSTALLATION & REMOVAL AT WILLOW ISLAND HYDROELECTRIC PROJECT

A technical paper presented by: The Ruhlin Company & Mueser Rutledge Consulting Engineers First presented at HydroVision 2014
Abstract
For a hydroelectric powerhouse to be constructed along the Ohio River at Willow Island, a cellular cofferdam was necessary to control ground and surface waters from entering the required 110-foot deep excavation.

The Ruhlin Company was contracted to manage the design and construction of the cofferdam. Analysis was performed for various load cases ranging from normal pool elevations to flood conditions. Engineering solutions included rock anchors to mitigate poor rock strengths, support of earth structures, weighted berm, dewatering wells, and unique tie-in to existing dam pier utilizing steel closure caisson and steel waterstop plate.

Challenging site conditions played a key role in the design. The overall structure, designed by Mueser Rutledge Consulting Engineers to provide protection to 100-year flood levels, included cellular steel sheet pile cofferdam, tie-in to the existing dam structure, emergency flooding structure, deep seated slope stability and instrumentation and monitoring. Design also included engineering analyses for sliding, overturning, pullout, cell interlock stress, wye interlock stress, vertical shear, horizontal shear, and deep seated stability. Sheetpile cells consisted of 62.68 foot diameter cells constructed with PS 27.5 sheets for an average height of 67 feet. The entire undertaking took into consideration the USACE Engineering Manual, with a focus on factors of safety.

Installation required river access while working from barges and providing sufficient granular material in the river bottom to seat sheet piling while driving into bedrock. The structural steel template used during sheet pile installation remained in place during sand backfill operations to maintain cell shape and stability.

Upon completion of the powerhouse, removal sequence will closely follow reverse order of the installation process; however, system stability during deconstruction is a major consideration.
1. Introduction
American Municipal Power, a nonprofit organization that owns and operates electric generation facilities for its member communities, has been in the process of constructing hydroelectric power plants adjacent to existing navigation locks and dams along the Ohio River. The Willow Island Hydroelectric Project is one of them, and is currently under construction on the West Virginia bank of the river at the existing US Army Corps’ (USACE) Willow Island Lock and Dam. The plant will be located on the opposite bank to the lock. The powerhouse will encompass two turbine generating units with an estimated capacity of 35MW at a 20 foot head differential. The Ruhlin Company of Sharon Center, Ohio, has been awarded contracts to construct both the cofferdam and the powerhouse. Construction began in mid-2011, and the plant is expected to come online in 2015.

1.1. General Cofferdam Configuration
The cofferdam for the project was designed to allow excavation and construction of the powerhouse, intake, and tailrace channels in the dry while maintaining upstream to downstream pool separation (a 20 foot head differential under normal pool conditions) before being flooded and decommissioned. It has been designed to provide protection up to 100 year flood levels, which raise the river 20 feet (upstream) to 40 feet (downstream) over normal pool levels. The cofferdam has been outfitted with an emergency floodgate and spillway to allow safe flooding if the river is predicted to exceed those levels. Deep dewatering wells are being used to draw down groundwater during overburden excavation and for depressurization of bedrock during rock excavation. The cofferdam was designed by Mueser Rutledge Consulting Engineers (riverside, SOE, and rock anchors) and Geocomp (landside, stability analyses and instrumentation). All components of the design have been reviewed by the Owner's Engineer, MWH Americas, Inc., and were reviewed and approved by the Federal Energy Regulatory Commission (FERC) and the USACE. An aerial photograph of the cofferdam is shown in Figure 1.1.
Key cofferdam components are identified in Figure 1.2. The cofferdam consists of a 50 foot deep soil bentonite cut-off wall constructed from existing grade beneath an impermeable clay cap up to 25 feet tall around the landside of the cofferdam. On the riverside, a cellular cofferdam consisting of sixteen 62 foot diameter, 67 foot tall cells was installed to the top of rock. Thirty high capacity rock anchors, some requiring support of excavation (SOE) to allow proper location, are installed around the site to maintain stability if a 100 year flood were to occur. The cofferdam allowed the excavation for the powerhouse, including up to 60 feet of rock excavation, to proceed in the dry while water levels on the Ohio are more than 120 feet above the deepest excavation.
Figure 1.2 - Site Plan and Cofferdam Components

1.2. General Site Geology

The overburden at the site consists of alluvial sands and gravels overlain by finer grained silts and clays. The deeper, pervious sands and gravels allowed the use of a widely spaced deep dewatering well system for managing groundwater on the site. Thickness of the overburden varied from 0 feet in the river to as much as 60 feet thick on land, still several feet below the 100 year flood level the cofferdam was designed to protect against.

The top of bedrock was generally found at about El. 558 according to the Ohio River Datum. The bedrock consists of the Pennsylvanian Conemaugh Group, and consists of a variety of interbedded sedimentary rock types, including sandstones, siltstones, shales, claystones, and indurated clays with occasional limestone encountered during the project subsurface investigations. Rock stratigraphy was highly variable both vertically and horizontally throughout the site. Rock jointing was found to be generally low angle with varying dip directions. Evidence of slickensides was encountered in some of the borings, particularly in the finer grained sedimentary rocks. Based on the presence of slickensides in low angle joints, the design of the cofferdam included low
strengths (ϕ=14°) along potential horizontal failure surfaces with higher strengths across rock bedding.

Groundwater on site is generally controlled by river levels, with fluctuations in soil groundwater levels generally lagging behind river levels and water levels within the rock dictated by communication through jointing.

During construction of the existing dam, the USACE identified a fault beneath the planned overflow weir and Pier 9, both of which are within the footprint of the cofferdam for the construction of the powerhouse. See Figure 1.3. A further investigation identified the fault with a strike of N(75-80)°W and dip of 30°. The fault is up to several feet thick, and rock within the fault is highly fractured. Design of the cellular cofferdam, in particular deep seated sliding stability, carefully considered the presence of this fault on site.

1.3. Design Parameters and assumptions
The following river water levels were used in the design of the cofferdam:

- Normal upstream pool: EL 602.0
- Ordinary high water downstream pool: EL 594.7
- Normal minimum downstream pool: EL 582.0
- Upstream 100 year flood pool: EL 621.8
- Downstream 100 year flood pool: EL 620.3

The seismic load condition has been evaluated during design of the cofferdam using the Construction Basis Earthquake (CBE) as directed in the Contract Documents. The accelerations used are:

- Peak Horizontal Ground Acceleration: 0.02 g
- Peak Vertical Ground Acceleration: 0.02 g

2. Cofferdam Design
2.1. Cellular Steel Sheetpile Cofferdam
The design for the Cofferdam and Excavation Contract of the Willow Island Hydroelectric Project includes the construction of sheet pile cellular structures as part of the cofferdam to prevent river and groundwater from infiltrating into the excavation for the construction of the powerhouse. The cellular cofferdams have been designed such
that the cells remain stable during installation, dewatering, excavation, and throughout powerhouse construction.

Both internal and external stability checks have been performed for cells on the upstream and downstream sides of the Willow Island dam. Additionally, deep seated stability of the cofferdam and underlying rock mass has been considered post-excavation. The cellular cofferdams have been checked for sliding, overturning, pullout, bursting, and shear following methodologies prescribed by the U.S. Army Corps of Engineers (USACE). Minimum factors of safety for each condition have been established in accordance with USACE and Federal Energy Regulatory Commission (FERC) guidelines and were met for a variety of different load cases.

The following configuration was used for the construction of the cellular cofferdam:

- Cell diameter: 62.68 feet.
- Cell sheet count: 120 sheets per cell.
- Cell height: approximately 67 feet.
- Arc radius: 15.48 feet.
- Sheets used to construct cells and arcs: PS 27.5 straight sheet. Wyes: PS 31 straight sheet, bolted to bent PS 31 half sheet with backing plate.
- Minimum interlock strength = 24 kips/inch.
- Minimum soil overburden during placement and initial filling: 8 feet.
- Final berm: fill to EL 600 upstream and downstream after dewatering to top of underwater berm.

2.1.1. Design Methodology

The overall geometry of the cellular cofferdam has been established as indicated in the contract documents. Because of this, the design of the cellular cofferdam involved determining support berm sizes to maintain stability in all potential failure modes throughout the stages of construction and operation. Proposed berm heights were iteratively developed, analyzed, and checked until satisfactory results were met for all analyzed conditions. Maximum water levels within both the cells and support berms were determined based on expected drainage performance of the cells.

2.1.2. Cell analyses – potential failure modes

Stability checks were performed in accordance with the recommendations found in the Army Corps’ Engineering Manual EM 1110-2- 2503 “Design of Sheet Pile Cellular Structures.” Several checks are performed to confirm the external stability of the cell as a whole, including sliding, overturning, and deep seated sliding. Internal stability checks confirm the performance of the individual components of the cellular structure, and include pullout, interlock tension, and vertical and horizontal shear.
Sliding stability investigates the likelihood of the soil at the base of the cell shearing and the cell sliding. Failure surfaces are assumed planar, and all driving (water pressure and active earth pressure) and resisting (base friction, passive earth pressure and water pressure) forces are considered. We have conservatively assumed a weak rock zone at or near the top of rock in our evaluation of sliding stability. The safety factor is defined as the ratio of total available resisting forces to driving forces.

The overturning analysis assumes that the soil filled cell is a rigid structure and could fail by rotating about the inboard toe. Overturning is generally not considered to be a likely failure mode, as other failure modes will occur before the cell will overturn. The check against overturning resolves all forces on the cell (including cell weight) and confirms that the resultant of all forces falls within the central third of an equivalent rectangular base.

Due to the presence of the excavation near the cell structures and weak claystone and shale layers within the rock on site, deep seated stability was considered over a larger area than just the footprint of the cells, and considered the potential for movement of the rock mass and cell into the rock excavation. Meeting deep seated stability required the use of high capacity stranded rock anchors.

Penetration of sheets into overburden needs to be sufficient to prevent pullout of outboard sheets due to tilting. Piles founded on rock, as proposed for Willow Island, have a risk of the outboard piles rising as overturning moments are applied to the cells. Resistance is computed based on the friction forces generated by soil on the embedded portion of the sheet piles.

Soil contained within the cells will induce a hoop stress in each cell structure, which puts the cell sheets in tension. The sheet interlocks need to be able to resist this force or the cell will burst. The factor of safety is checked by comparing the ultimate interlock strength to the tension created by the maximum internal cell pressure.

The net overturning moment on the cell structure attempts to tilt the cell. This tilt will be resisted by shear within the cell. Two different modes of failure are considered: failure in a vertical plane through the center of the cell and failure in a horizontal plane. In each case, moments are resisted by the shear strength of the cell fill and by the friction that develops within the sheet pile interlocks.

Each of these failure modes, when appropriate, was analyzed for eight different load cases that represent the stages of cellular cofferdam construction, operation, and flooding recovery. The minimum safety factors for each analysis were determined based on the minimum required safety factors found in EM 1110-2-2503 and Chapter 10 of the FERC’s “Engineering Guidelines for the Evaluation of Hydropower Projects.”
2.1.3. Analysis Results

Based on the proposed cell and berm configuration, the cellular cofferdam is stable during construction, normal operating conditions, flood and earthquake loads. Design assumptions in specific cases are summarized in the following sections. All factors of safety, provided in the “Required Factors of Safety” section of this report, have been met in all load conditions for both the upstream and downstream cells. Sliding, overturning, and shear were considered not critical during initial filling and were not analyzed in cases 1A and 1B. Bursting and interlock forces are the only safety concern during initial filling as the load condition involves balanced water levels and overburden on both the inboard and outboard sides of the cofferdam.

Load cases 2, 3, and 4 were analyzed in iterations to determine the most effective size for the underwater berm. The elevation of the underwater berm was chosen to meet the required safety factors during construction while providing flexibility during dewatering of the cofferdam. Higher driving forces will be encountered if the cells drain slower than the ponded water can be removed during initial dewatering, which could limit how quickly the cofferdam is dewatered. An underwater berm constructed to elevation 586 upstream and elevation 580 downstream stabilizes the cells such that the head differential between the cell water level and the inboard water level is not an issue; the cells remain stable with the water lowered to the underwater berm level inboard with the cell fill saturated to the normal pool level upstream or downstream.

The weighting berms, both underwater and final, have been sized so that the required minimum factors of safety are met for each loading condition considered.

2.2. Stabilization of Rock Excavation

2.2.1. Stability of Slopes

The slope stability analysis of the excavation was analyzed in thirteen design sections using 2D limit equilibrium techniques in accordance with USACE EM 1110-2-1902 “Slope Stability” and by the tension crack “wedge” method in accordance with EM 1110-2-2503 for Cellular Cofferdam analysis sections. Due to the presence of slickenside and the potential for weak planes in the indurated clay, claystone, and siltstone, a low design friction angle of 14 degrees was selected for horizontal failure surfaces in rock.

Results of the analyses showed inadequate deep seated stability factors of safety through the rock for the steeply cut (1H:5V in rock) excavation on the river side and the flatter intake and tailrace design sections under 100 year flood conditions.

Further complicating stability is the presence of a fault passing through the site. The fault had previously been encountered during the construction of the original dam. That structure underwent a redesign during construction after it was exposed to address
stability. Based on the location of the fault relative to the excavation for powerhouse construction, the potential for stability issues existed in significant rock blocks up to 200 feet wide by more than 40 feet deep.

The evaluation of the results of deep seated stability analyses established that supplemental support was needed to allow the necessary excavation on site. The primary means of support selected were rock anchors. However, additional steps were taken to minimize the required anchor loads. Required anchor line loads under 100 year flood levels were quite high, because the wedge analysis required to determine deep seated stability is highly sensitive to the depth of the excavation, as hydrostatic driving pressures increase more rapidly than the frictional resistance at the base of the potential wedge. The construction of the powerhouse included the installation of a more than 11 foot thick base slab throughout the powerhouse footprint. This slab, once sufficiently built up, braced the deep excavation to the rock on the other side of the excavation.

The design was modified to utilize a lower design flood level while the deepest portion of the excavation was exposed. This “intermediate” flood level, equivalent to about a 10 year flood, lowered design water levels by more than 10 feet relative to the 100 year flood. This reduction in design water level reduced the load on the deepest failure surfaces, and the required rock anchor loads. The cofferdam was returned to 100 year flood protection as soon as the powerhouse base slab construction (which provided the remaining support) was complete.

2.2.2. Rock anchor design

With the modified approach to reduce required anchor loads, final design line loads varying from 42 to 86 kips per linear foot were needed to improve deep seated stability safety factors above required minimums under the 100 year flood. In addition, almost 3,000 additional tons of support was required to stabilize potential fault blocks. In order to maintain appropriate factors of safety during all stages of construction, rock anchors were installed prior to the start of rock excavation.

The rock anchors were designed in accordance with USACE EM 1110-1-2908 “Rock Foundations” and Post-Tensioning Institute (PTI) recommendations. The final design consisted of thirty high capacity anchors that utilized fifty nine 0.6 inch diameter, seven wire strands designed to provide a design load of 2,074 kips of support at 60 percent of the strand capacity. Anchors were installed in blocks of two to three anchors across the site. The rock anchors were placed as close to the outside of the cofferdam as practical, where they would provide the most benefit to the stabilization of the rock mass.
Anchor blocks that could accommodate two anchors and three anchors were designed for use around the site. Use of the two or three anchor blocks and their spacing were varied at the different design sections in order to accommodate varying line loads, the tributary length of the design sections, and site geometry and construction constraints.

The anchors were designed with a 12 inch borehole diameter, at an inclination of 45 degrees. Bond zones were all 50 feet long. Free lengths were determined as the minimum length that would satisfy all of the following: prevent potential rock mass pull out (even though the anchors were installed into the mass and not an external structure, making pull out unlikely), keep the anchor bond zone below all potential failure surfaces requiring stabilization, keep the anchor bond zone 5 feet below the critical stability failure surface, and meet the requirements of USACE EM 1110-1-2908. Based on these requirements, anchor free lengths ranged from 77 to 88 feet.

2.2.3. Anchor bond stresses
The rock to anchor bond stress determines the capacity of anchors of a given geometry. A supplemental subsurface investigation was carried out that collected large diameter rock cores for testing. Pullout tests, in which a threaded steel bar is grouted into a hole drilled in the core and then pulled out, were performed on two different samples of each of the rock lithologies found on site (limestone, sandstone, siltstone, and claystone). Results generally showed quite good bond stress, with test results averaging anywhere from 214 psi (claystone) to 579 psi (siltstone) for the different rock types.

While the test results for claystone showed a reasonable average pullout stress, one of the tests showed a low result of just 78 psi. Additional evaluation of the claystone was undertaken to better understand the potential bond strength in this rock type.

Unconfined compressive strength (UCS) test data was available in this area of the site from both the original USACE design of the dam, the investigation performed for the design of the hydroelectric plant, and the supplemental subsurface investigation performed for rock anchor design. Bond stress was related to the UCS test results based on the following relationship:

$$\sigma_{b_{ult}} = 0.1 \cdot UCS_{peak}$$

Eleven additional data points were established based on test specimens that were directly identified as claystone. The equivalent ultimate bond stresses from the UCS tests were then plotted with the results of the pullout tests in histogram form and a cumulative lognormal distribution ($\mu=5.55$, $\sigma=0.755$) was fit to the data, as shown in Figure 1. Based on this evaluation, a lower third pullout value for the claystone of 190 psi was selected. Combined with the other rock lithologies found throughout the site, a conservative ultimate design bond stress of 200 psi was selected.
During review of the anchor design, there were concerns about the performance of the bond zone, both in the short term and long term due to creep. Several measures were taken to prove out the performance of the anchors. All anchors were water tested in the bond zone to confirm the quality of the rock there prior to installation, and a two-stage grouting procedure, where the bond zone was grouted, tested, and stressed prior to grouting the anchor free length, was utilized. The first anchors that were installed were successfully performance tested to 133% of the design load to confirm the design capacity. A successful extended creep test was carried out on one of the performance tested anchors, and all remaining anchors were proof tested, and locked off at 105% of the design load.

The design included several contingencies in case the target bond was not reached. Secondary grout tubes were included in the anchors, and would have been utilized for post grouting, had any of the anchors not passed testing. Had the post grouting not provided sufficient capacity, the anchor blocks, which were designed for two or three anchors, were designed with spare sleeves to accommodate the installation of additional supplemental anchors, and would have allowed for a 33 percent reduction in bond stress. Fortunately, the system worked as designed, and these features did not need to be utilized.
2.3. **Support of Excavation**

Support of excavation (SOE) was installed in some areas to locate rock anchors in areas that would provide the most slope stability improvement, and allow additional intake and tail race channel excavation within the limits of the anchors. Support of excavation (SOE) was needed to allow construction of rock anchors required for deep seated sliding stability. The SOE design is not dependent upon the final rock anchor design. All SOE design sections include a stage during anchor construction that completely ignores all resistance in rock above the bottom of the rock anchor pits. The compressive load that the rock anchors introduce into the rock mass is not used to provide any benefit to the SOE.

Installation of the SOE wall allows removal of a portion of the cellular cofferdam support berm. Under the 100 year flood, removal of this portion of the berm reduces the factor of safety against sliding below the required minimum of 1.5. In order to maintain the required factor of safety, the walls have been designed to support the load needed to maintain the sliding safety factor in addition to soil pressures. The flood load was applied as a triangularly distributed load, and was considered in all SOE construction stages except rock anchor pit excavation, when pits would have been backfilled with concrete if a flood was predicted).

### 2.3.1. SOE Design

The support of excavation (SOE) was designed to allow partial removal of the weighting berm to accommodate the installation of rock anchor pits and provides supplemental sliding resistance under a 100 year flood event. A staged analysis has been performed at each design section, and includes an analysis of the SOE with an open rock pit in front of the wall under normal pool conditions and an analysis of the wall under a 100 year flood with sliding stabilization load applied where applicable.

The SOE consists of soldier piles and lagging to support the excavated cut. The wall is in cantilever or uses tie backs as required depending on the depth of cut. Both cantilever and anchored sections have been designed by the Free Earth method, in accordance with EM 1110-2-2504. Locations of design sections are shown in plan on Figure . Five different design sections were analyzed with wall heights varying from 32 feet to 12 feet.
Prior to SOE installation, the cellular sheet pile cofferdam resisted horizontal hydrostatic forces through sliding resistance at the base of the cells and weighting berm and provided factors of safety against sliding of 3.1 to 4.4 under normal pool and 1.5 to 1.6 under flood pool. Partial removal of the weighting berm for SOE and rock anchor installation reduced these factors of safety to 1.7 to 2.4 under normal pool and 1.3 to 1.5 under flood pool. The requisite factors of safety are maintained under normal pool conditions but are not provided at all sections under flood pool conditions, and supplemental sliding resistance was provided by the SOE wall as shown in Figure 2.3.
This supplemental sliding resistance is provided by the tie-back anchors and the steel soldier piles with sufficient embedment to prevent kick-out of the toe.

During rock anchor construction, all soldier pile support in rock above the bottom of excavated anchor pits has been ignored. In order to assure that the requisite factors of safety are maintained throughout the construction period the design assumes that anchor pit excavations are performed only during normal pool conditions and that each pit is immediately backfilled with concrete after rock excavation. Should a flood event be predicted during anchor pit excavation, the rock removal will be halted and open pits will be temporarily backfilled with concrete. Design loads on the SOE wall are shown in Figure 2.4.
Under this condition the rock anchors are installed and the SOE wall is designed to provide lateral support of the weighting berm plus supplemental sliding resistance to maintain sliding factor of safety during a 100 year flood. The flood stabilization load has been applied as a distributed load with a triangular distribution. All components of the SOE have been designed for this additional load.

2.3.2. Wall loads
The earth pressure loads on the SOE have been estimated as active pressures following the methods described in EM 1110-2-2504 “Design of Sheet Pile Walls.” The effect of the berm slope (β) behind the SOE is included. This sloped berm will increase the active pressure coefficient used to determine loads on the wall. The soldier pile and louvered lagging system was assumed to be free draining, which was verified by piezometers installed in the berm and monitored during construction, as are the natural and berm fill materials at the top of rock, with no water pressure on the wall.

Prior to removal of the toe of the berm, the sliding safety factors for the 100 year flood were met for all load cases. Once the toe was removed, the safety factor was reduced and additional support was needed to maintain the required safety factor:

<table>
<thead>
<tr>
<th>Design Section</th>
<th>Maximum required sliding stabilization load for FS=1.5, kips/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstream, 30’ anchored wall</td>
<td>18.2</td>
</tr>
<tr>
<td>Upstream, 22’ anchored wall</td>
<td>5.0</td>
</tr>
<tr>
<td>Upstream, 12’ cantilever</td>
<td>-</td>
</tr>
<tr>
<td>Downstream, 27’ anchored wall</td>
<td>22.8</td>
</tr>
<tr>
<td>Riverside, 32’ anchored wall</td>
<td>32.9</td>
</tr>
</tbody>
</table>

The wall loads and resisting forces have been developed on a per foot of width basis. Proportioning of the active and passive earth pressures has been performed based on the relative width of soil engaged per pile. The full sliding stabilization load has been applied to the pile, and the results of the per foot analysis are multiplied by the actual pile spacing for design calculations.
2.3.3. Design methodology
The design was performed in accordance with EM 1110-2-2504 by the Teng Free Earth Method for both the cantilever and braced design sections.

Construction Stages
A staged analysis including four stages for the anchored wall sections and three stages for the cantilever section was performed. The pile bending, shear, and toe stability were checked at each stage of excavation for both the normal pool (Load Case 1) and flood condition (Load Case 2). The stages analyzed for the anchored wall sections are:

- Stage 1 – Initial cantilever: An initial cantilever stage with an interim excavation subgrade is made to reach the tieback level. Active soil pressures act on the cantilevered wall, with increased soil pressures due to the sloping berm behind the wall.
- Stage 2 – Install tie-backs and excavate to top of rock: Excavation to the top of rock after wales and tiebacks have been installed with active soil pressures and the tieback load applied to the wall, and the required distributed flood load added for Load Case 2.
- Stage 3 – Rock anchor pit excavation: Excavation of anchor pits in front of the SOE soldier piles for normal pool only. All rock resistance between the SOE wall and the anchor pit to the deepest point in the pit has been ignored during this stage. Calculated embedments start at the bottom of the anchor pit.
- Stage 4 – Rock anchors installed: Final condition after anchor blocks have been constructed and anchors are installed. Resistance to the top of rock has been reestablished by the concrete anchor blocks and anchors. No additional resistance from the anchor force has been included in assessing the required toe embedment.

Pile design and embedment
Calculations of wall bending moment, shear and required embedment follow the Teng Free Earth Method. The soldier piles have been designed to resist the maximum moment and shear. Calculated toe embedment at each stage has been increased to meet the required safety factor (2.0 corresponding to usual loading, fine-grained soil, Q - Case) from EM 1110-2-2504. In the embedment calculations, the safety factor has been applied to the passive resistance in accordance with the EM and the required embedment is determined by the Teng method. Embedments in the excavated rock anchor pit stage were developed from the bottom of the anchor pits regardless of the condition in front of and adjacent to the pile under consideration.
**Wall component design**

The wale is analyzed as a continuous beam with multiple loads from the piles and tiebacks. Tiebacks were sized in accordance with PTI recommendations and according to manufacturer's recommendations for stress limits.

Bond lengths were determined in accordance with PTI. An ultimate bond stress of 100 psi was used for tieback design. Wale seats, welding, and bearing plate calculations have been provided for each component of the wale support system, were designed in accordance with the 13th edition of AISC.

Lagging used was generally timber, designed in accordance with the 2001 Edition of the American Wood Council’s National Design Specification (NDS) for Wood Construction. The load is assumed to arch from the piles to the lagging, and a triangular pressure distribution has been used to model this effect. The lagging was designed as a simple beam spanning from soldier pile to soldier pile. The lowest levels of lagging will use steel sections instead of timber because of the higher loads on the lagging due to the additive sliding resistance under the 100 year flood load. Soil arching was not included due to the stiffness of the steel and the load is applied as a uniform load instead. See Figure 2. for a photograph of the SOE wall in place.

*Figure 2.5 - Photo of Downstream SOE*
2.4. **Cofferdam Tie-in to existing dam**

Protection of existing USACE structures was a very important component of the design of the cofferdam. The design of the cofferdam included several components that were required to tie the cofferdam into one of the piers of the existing dam.

2.4.1. **Closure Caisson**

On the upstream side of the dam, a gap exists between the upstream edge of Pier 8 and Cell 6 of the cofferdam. This gap was fitted with a rolled ¾” thick steel plate closure caisson 9 feet in diameter, which was driven to rock. The closure caisson was secured between the cell and pier face by two levels of wire ropes fastened to the caisson and attached to Pier 9 of the dam. Tension on the wire ropes and water pressure on the exterior of the caisson provide the force to seal the gaps between the caisson, cofferdam, and existing dam. Timber wedges were used as required to improve the seal.

2.4.2. **Waterstop Plate**

In the downstream portion of Pier 8 of the existing dam the top of concrete drops below the 100 year flood elevation. A stiffened waterstop plate has been provided to maintain water tightness of the cofferdam to the same elevation as the downstream cellular cofferdam of 623.5, about 3 feet above the 100 year flood level. It is supported with wales, rakers, and struts, and the bottom of the plate is welded to an angle bolted to the dam concrete through a neoprene pad to seal against potential leakage.

On the downstream side, where the waterstop plate meets the cellular cofferdam, an angle has been welded to the outside of the plate, and a polyethylene pipe with timber wedges has been used to provide the seal to the cells.

2.4.3. **Existing Dam Stability / Stress Analysis**

Construction of the cofferdam and dewatering of its interior creates an unbalanced water pressure condition not originally designed for Pier 8 of the existing dam. This load could impact the stability of Pier 8 to rotation or sliding, and will induce additional stresses in the pier. Analyses were performed to confirm that the existing dam remains stable under these conditions.

For the stability analysis, Pier 8 was conservatively evaluated considering the pier as a gravity structure independent of the adjacent sill slabs in Bays 7 and 8. The pier was analyzed using the USACE computer program “Three-Dimensional Stability Analysis/Design (3DSAD) Program.”

The geometry of the pier footing and stem up to elevation 601 is input in the program, and the corresponding weights are computed by the program. The stem weight above elevation 601 is computed by the user and input in the program. Additional inputs in the
program include initial uplift, vertical and lateral water loads (transverse and longitudinal), vertical and lateral (transverse) berm loads, and transverse wind load, along with other loads taken from the original USACE design stability calculations. They include crane bridge, crane reaction, gate reactions, gate buoyancy, machinery load, ice force, and silt reactions.

Three pool conditions were considered in these calculations, a normal pool condition, a 100-year flood condition and a construction condition. The stability calculations determined that a weighting berm was required on the cofferdam side of Pier 8 in order to maintain base compression requirements during powerhouse construction. Sliding of the Pier was not found to be an issue.

The existing Willow Island Dam was designed as a gravity structure, and was not designed for lateral loading from one side of the structure. As such, the dam is minimally reinforced, and a stress analysis needed to be performed to confirm the structure could tolerate the bending caused by high water on one side of the pier. The stress analysis was performed at the base of the stem at the top of rock for all three cases and at the subsequent upper construction joints affected by the specific loading conditions. These construction joints are at El 573 and El 583 for Case 1 (normal pool conditions) and at El 573, El 583 and El 593 for Case 2 (100 year flood conditions).

The reinforcing is checked following the procedures contained in the Corps of Engineers Manual EM 1110-2-2104 “Strength Design for Reinforced-Concrete Hydraulic Structures.” The material properties of the existing Pier 8 are $f_y = 40$ ksi, $f'_c = 3,000$ psi, $E_c = 3,000$ ksi. The forces computed for the determination of axial force and moment in the pier stem include the weight of the stem, water load, berm load and moment caused in the stem by Cell 9 internal pressure.

These checks found that the dam has sufficient strength to resist the loads imposed by the plant construction.

2.5. Emergency Flooding Control Structure
The cofferdam has been designed to maintain a dry excavation with surrounding water levels up to the 100 year flood level. An emergency flood gate was developed to be installed between Cell Nos. 13 and 14 that will allow filling of the cofferdam from the downstream side if needed to prevent potential damage due to overtopping if water levels are ever predicted to exceed the 100 year flood levels.

The floodgate consists of two primary components: a floodgate and spillway. The floodgate was designed as a lowered section between Cells 13 and 14 on the downstream side of the cofferdam, with a bridge over the top to allow construction traffic on all cells. The gate itself consists of timber needle beams with a steel H-pile for initial
opening. If the cofferdam needed to be flooded, the needle beams can be removed from above by an excavator. The spillway was made of grouted rip-rap to the top of rock to prevent erosion during filling. See Figure 2. for a photograph of the flooding structure.

![Figure 2.6 - Emergency Flooding Structure under construction](image)

### 2.5.1. Design Methodology

The emergency flood gate has been designed structurally in accordance with industry accepted standards, including relevant American Wood Council, American Concrete Institute, and American Institute of Steel Construction Standards, such that each component, including the gate, overpass bridge, base slab, timber and steel needle beams, support pipe and internal grouted rip rap spillway can resist the potential loads it may be subject to.

Further, the flood gate and spillway have been designed hydraulically so that the entire cofferdam can be filled to three quarters full in 4 to 12 hours (based on Contract requirements) if, during the construction period, a flood exceeding the 100-year event is predicted to occur. The highest water level that allows filling in 4 hours has been determined for several different river rates of rise from 0.1 to 1.0 feet per hour, rates that were selected based on a review of historical water level readings at the site.

Flow through the floodgate has been treated as open channel flow over a broad crested weir with square approach sidewalls and tail water effects. As such, unimpeded (no tail water effects) flow through the gate is determined based upon the following empirical relationship:

\[
Q = 2.65 \cdot (L - 2 \cdot 0.1 \cdot H) \cdot H^{1.5}
\]
Where:

\[ Q = \text{flow rate, cfs} \]
\[ L = \text{weir width, ft. (21.0 ft)} \]
\[ H = \text{depth of flow, ft.} \]

**2.5.2. Filling computations**

The preceding flow rate computations through the floodgate coupled with cofferdam volume calculations allowed the determination of the rate of filling the cofferdam. Computations proceed in the following order:

1. Assume a rate of rise for the river.
2. Select the headwater level at which the flood gate will be opened.
3. For each discrete time interval:
   a. Compute the average headwater level and flow rate into the cofferdam. If the water level inside the cofferdam is high enough to induce tail water effects.
   b. Compute the volume of water flowing into the cofferdam through the floodgate.
   c. Determine the new water level within the cofferdam.
4. The time to flood the cofferdam will be the length of time it takes for the interior water level to reach the 100 year flood level, or the length of time it takes for the interior and exterior water levels to equilibrate.

**2.5.3. Rip-Rap spillway**

The spillway has been designed to allow flow from the flood gate invert into the top of rock with as few transitions as possible. Flow velocities have been calculated based non-uniform open channel flow principles. Based on the results of the flow calculations down the spillway into the cofferdam, much of the flow down the spillway will have a velocity too high to use ungrouted rip rap to surface the spillway. A 30 inch thick grouted rip rap layer with a maximum stone size of 24 inches was used to resist erosion.

**2.5.4. Spillway depth**

The spillway depth will be controlled by the depth of flow and the freeboard required to prevent wave action, air bulking, splash and spray from overtopping the spillway. Freeboard has been developed according to recommendations from the U.S. Bureau of Reclamation, and is a function of the flow velocity and depth. The recommended freeboard in feet is:

\[ \text{Freeboard} = 2.0 + 0.025 \times v^{3/4} \]
We have conservatively developed the containment berms on the sides of the spillway assuming critical flow from the narrower 21 foot wide section of the spillway at the top, transitioning to normal flow 100 feet down the spillway. Based on those flows and required freeboard, the spillway will include 8 foot tall side berms at the top and transition to 4 foot tall side berms at the base toe of the spillway.

2.6. Instrumentation
An extensive instrumentation system was installed on site in order to confirm the performance of the cofferdam during construction. The instrumentation system had three goals: to confirm the safety and stability of the cofferdam by monitoring movements throughout the site, to confirm the stability of the site and dewatering system performance by monitoring water levels, and to confirm the performance of the rock anchors by monitoring load within them.

The majority of the instruments were automatically monitored by data loggers with data uploaded to a web server for remote access to data on a near real-time basis throughout the construction.

2.6.1. Movement monitoring
Movement monitoring on the project included the use of an automated motorized total station to monitor movement of dozens of prisms mounted on the cellular cofferdam, SOE, rock anchor blocks, the embankment cofferdam, and the existing dam structure. Eight in-place inclinometer (IPI) installations were installed at critical locations around the site as well, including at the top of the rock excavation, through one of the cells, and within the embankment. Finally, four biaxial tilt meters were installed on the existing dam to monitor potential movements during rock excavation.

A detailed finite element analysis of the cofferdam was used to establish expected levels of movement as excavation and construction progresses. Results of this analysis were used to establish threshold levels for survey monitoring prisms, in place inclinometers, and tilt meters.

2.6.2. Groundwater monitoring
A total of forty vibrating wire piezometers were installed around the site to confirm the performance of the dewatering system and to confirm that required drawdown to maintain slope stability was met during construction. About ten of these piezometer installations included isolated shallow (soil groundwater) and deep (rock) piezometers to monitor water levels below the top of rock in addition to soil water levels.

The results of slope stability analyses were used to determine threshold values for piezometers, and 3D finite difference modeling of the site and surrounding area were utilized to develop estimates of dewatering system pumping quantities. Due to the
potential variability inherent in construction dewatering modeling, no threshold values were set on pump flow rates.

An illustration of the deformation and groundwater monitoring instrument locations is included in Figure 2.

![Figure 2.7 - Locations of groundwater and deflection monitoring instruments](image)

### 2.6.3. Load monitoring

In lieu of installation of load cells on select rock anchors around the site, load monitoring was performed through the use of individual elasto-magnetic load sensors installed on three of the fifty-nine strands within each of the thirty rock anchors. The use of these sensors allowed partial monitoring of all rock anchors instead of just a select few. Monitoring of all rock anchors was greatly desired due to the high variability of rock stratigraphy and the potential risk that an individual anchor could be completely bonded within weaker claystone or indurated clay.

Anchor load sensors were set to alarm if a change in load of more than 10% was observed, indicating possible movement of the rock mass or loss of bond capacity.
3. Cofferdam Installation
3.1 Sheetpile Cells and Connecting Arcs
This work consisted of a steel sheet pile structure having eight upstream cells with six connecting arcs and a closure caisson and eight downstream cells with seven connecting arcs. Also included were the weighting berm, which served to support the cofferdam structure and a sheet pile cut off wall to tie the cellular cofferdam to the landslide cofferdam soil bentonite slurry cut off wall.

3.1.1 Construction Methods
3.1.1.1 Preliminary Considerations
Prior to the start of sheet pile installation, the following preliminary work was required;

- Preconstruction Survey to document the existing conditions of USACE Facilities at the Willow Island Lock and Dam. This included a detailed survey on Piers 8 and 9 along with the alignment of the Crane Bridge and Gate in Bay 8.

- Pier 9 Monitoring, the purpose of this plan was to ascertain that Pier 9 was not affected unfavorably by construction activities. Initial monitoring was to begin before the start of any construction activities.

- The final considerations prior to starting work was to survey existing conditions within the cofferdam footprint and establish, at a minimum, reference points for future Control Points. An underwater Bathymetric Survey was performed to identify locations of the dam structure and to profile the upstream and downstream riverbed.

- Design and fabrication of structural steel templates for both the cells and connecting arcs.

3.1.1.2 Temporary Cofferdam Sheeting
Initial excavation required Horizontal Control Points to be established in order to place the cells in their specified locations. Prior to setting templates and sheets a pre-excavation was performed at Cells 1 and 15. These cells are located along the riverbed on the upstream (Cell No. 1) and downstream (Cell No. 15) sides of the dam. The purpose of this excavation was to limit driving of flat sheets through a large amount of overburden. The maximum amount of overburden was limited to 30 feet. This excavation was also intended to remove any existing rip rap, cobbles, logs, or debris that would have prevented driving the sheets. This excavation was done with a Cat 345 excavator and the waste materials hauled with 35 Ton Articulating Trucks to a spoil area.
The next step was to prepare the upstream and downstream riverbed. Similar to the initial excavation, this step required removal of logs, cobbles and debris. Also included in this step was the placement of cell fill where the riverbed had less than 8 foot of overburden to the top of rock. This work was done with marine equipment. This included a Manitowoc 888 Crawler Crane with Clam on a Crane Barge with a Material Barge for disposal of debris and cell fill as required. An 800 hp Tugboat maneuvered the barges.

With preparatory work complete, the sheet pile on the first two 62.68 foot diameter cells was installed. Sheet pile specifications required PS27.5 and PS31 flat sheets, as well as fabricated wye connectors. The first step in setting the sheets required the setting and securing of the ring template. This template was secured to spuds that were driven through the overburden to bedrock. In the event the spuds were unable to maintain its position due to minimal overburden, the spuds and template would be moored to temporary supports. See Figure 3.1 for sheet placement on the ring template.

![Figure 3.1 Ring Template](image1)

![Figure 3.2 Initial setting of cells](image2)

With the template secured in place, the fabricated wyes were set on the template ring. Following the wyes, the balance of the sheets was laced together until all sheets were interlocked. The sheets were then driven in stages (6 foot maximum) to bedrock using a vibratory hammer and seated individually into bedrock utilizing an MKT 983 Air Hammer powered by a 750 CFM compressor. See Figure 3.2 for initial setting of cells.

This work was done with the marine equipment described above. This included a Manitowoc 888 Crawler Crane on a Crane Barge with a Material Barge that the sheets were stored on. An 800 hp Tugboat maneuvered these barges. See Figure 3.3 Cell Installation Equipment.
After two adjacent Cells were completed and backfilled, the Connecting Arc between the two cells was set. Similar to the Cells a template in the shape of the arc was used to support the sheets while interlocking. After lacing was complete, the sheets were driven to bedrock in the same stages (6 foot maximum) using a vibratory hammer. The same marine equipment used to install the cells was used for the connecting arcs.

Special consideration was given to Cells 8 and 9 and their connecting arc. These cells and arc partially rest on top of the Dam Structure (stilling basin) and care was taken to set and seal the bottom of the sheets without damage to the stilling basin. Similar to the riverbed, 8 feet of cell pre-fill was placed over top of the stilling basin. Burlap bags filled with bentonite were also used along with the pre-fill to help seal the bottom of the sheets. The same marine equipment as previously described was used to perform this work with the exception that the sheets were lowered to rest on top of the stilling basin and not driven with a vibratory hammer.
Cell drainage was an important consideration to allow water within the cells to drain. Weep holes were cut on the inboard face of the sheets. These holes were cut after backfill was completed and maintained by rodding to stimulate flow.

### 3.1.1.3 Cell Fill

The 62.68 foot diameter cells were filled prior to the connecting arcs. Clean sand cell fill material was placed utilizing a Manitowoc 888 Crawler Crane with a clam bucket on a Crane Barge. Material was unloaded from a barge with the clam bucket and dropped into the cells.

Once access became available from the landside to the top of the cells it was possible to deliver sand with 35 Ton Articulating Trucks. From a stockpile on land the sand was loaded into 35 Ton Articulating Trucks with a 4 cyd Loader. The trucks dumped the fill in the vicinity (reach) of a crawler crane with a clam bucket or a Cat 345 excavator to place in the cell.

Special consideration was given during filling the cells to ensure that silt did not accumulate in a layer within the cell when filling work was interrupted by more than 4 hours and the water level in the cell was above the sand. In this case the water was agitated with a vibratory probe to disperse the silt. When the level of the sand was above the water in the cell the layer of silt was removed with a clam or dispersed by agitating with the vibrating probe.

The connecting arcs were placed in the same manner as the cells with the exception that no connecting arc was filled until two adjacent full diameter cells are filled. The same equipment placing cell fill was used as well.

Similar to filling the full diameter cells, special consideration was given to ensure that silt did not accumulate in a layer within the connecting arc cell. Again, care was taken if filling work was interrupted by more than four hours when the water level in the cell was above the sand. In this case the water in the cell was agitated with a vibratory probe to disperse the silt. Likewise, if the level of the sand was above the water in the cell the layer of silt was removed with a clam bucket or dispersed by agitating with the same vibrating probe.

Special consideration was given to filling the downstream connecting arcs. In addition to the adjacent full diameter cells being filled, the downstream connecting arcs were filled until the water level in the same adjacent full diameter cells was confirmed to be at river elevation. This was accomplished by cutting weep holes in the sheets and verifying that the cells had drained to river elevation.
Special consideration was given to the instrumentation being installed in the cells. As noted above weeps were cut to control water (reduce to river level) in the cells. Consequently, the piezometers located in the cells were very important to verify in addition to visual inspection that the water had drained.

3.1.1.4 Closure Caisson
The Closure Caisson is located on the upstream side of the dam and serves as a seal between Cell No. 6 and Pier 8. The caisson was set with the Manitowoc 888 Crawler Crane on a Crane Barge and driven to bedrock with a vibratory hammer. The top and mid span of the caisson were moored to Pier 9 through an existing mooring ring and with the use of wire rope and turnbuckles. The caisson was then filled with the same sand used in the cells.

3.1.1.5 Weighting Berm
The underwater berm was placed with a Manitowoc 888 Crawler Crane and Clam located on the top of the Cells. The underwater weighted berm was the same well-graded sand material which was used in the cell fill. This sand was stockpiled on top of the cells within reach of the crane noted above. This stockpile was generated by using both the marine equipment on the river or the stockpile on land and delivered with the 35 Ton Articulating Trucks.

The underwater weighted berm placement began at the lowest elevation and was placed in three foot lifts parallel to the axis of the cellular cofferdam. The underwater weighted berm was placed in this manner to the designed elevation specified by MRCE. Berm placement above this elevation becomes the above water-weighted berm.

The 35 Ton Articulating Trucks delivered material from the stockpile and a dozer knocked down the piles into 12 inch lifts. An 84” Smooth Drum Compactor compacted the material with a minimum of two passes. Similar to the underwater work, the fill was placed parallel to the axis of the cofferdam until it reached the designed elevation.

3.2 Emergency Flooding Structure
The Emergency Flood Gate is intended to provide a means to fill the cofferdam with water if river levels are predicted to rise above the 100 year flood elevation, thus threatening to have water overtop the cofferdam structure. This gate will allow three quarters of the cofferdam to be filled in 4 to 12 hours and avoid uncontrolled overtopping of the cofferdam. The Emergency Flood Gate is located between Cell Nos. 13 and 14 on the downstream side of the dam and is located on top of the connecting arc between these cells. This gate consists of three major components. The first includes the concrete footing, support walls, and timber needles of the gate. The second
being the grouted rip rap spillway with containment berm and the third consisting of the equipment bridge.

3.2.1 Construction Methods

3.2.1.1 Flood Gate
Upon completion of Cells Nos. 13 and 14 the connecting arc cell was backfilled. The first step was constructing the concrete footing which supported the timber needles and side support walls adjacent to each cell. The footing work began with surveying existing conditions and then grading. The footing was formed using custom built wood forms and held in place by staking in the sand. A form was installed to create the box-out for the needle timbers at its base. Rebar was installed followed by placing the concrete. After the footing was cured, the two side walls were formed with custom built wood forms. The support channels for the timber needles were attached to the wood forms. Similar to the footing, after the forms were set, the rebar was installed followed by concrete placement. After the concrete had cured the timber needles were installed. The 12”x12” timber needles were set in the footing pocket against bituminous joint filler. The balance of the void was filled with bituminous concrete after all timbers had been set. The pilot needle was set first followed by the balance of the timbers. The batter on the timbers was created by a 16” diameter x 1/2” walled pipe embedded on each end in the side walls. 4” studs were installed to tie the side walls to the sheets. Finally, the timber needles were treated with mastic and building paper and also had wire rope lifting devices.

The above work was done with a 110 Ton Link Belt Crawler Crane positioned on top of Cell No. 14. Concrete, rebar, formwork timber needles were delivered to the crane from land across the top of the cells.

3.2.1.2 Concrete Arc Cap and Grouted Spillway
This portion of work was constructed in three phases:
1. The first phase focused the work on top of the connecting arc cell. The first step was to perform the grading of the cell backfill. Mirafi Filterweave 300 Filter Fabric was then placed over the arc area. Next, 18” of 304 aggregate was placed and compacted followed by the placement of 6x6 wire mesh. Then 4” slotted PVC pipe was augured vertically in a 5’ x 5’ grid. Each 4” pipe was tied to the wire mesh for support. In order to tie the arc cap to the grouted spillway, #6 and #5 rebar was installed. Finally, 12” of concrete was poured as a cap on the arc and finished. The concrete was placed using a 60 meter concrete pump truck.

2. The second phase focused on the spillway located on top of the weighting berm prior to unwatering. Similarly, the weighting berm was properly graded to an elevation of 609.0 prior to placing the composite geogrid filter fabric. This work was followed by
placing the Type C stone rip rap. The containment berm was placed to an elevation of 618.0 at the same time the grouted rip rap was placed and was held with temporary supports until the spillway was completed to rock. The rip rap was placed with a Cat 345 Excavator.

3. The third phase began upon completion of unwatering and continued with the weighted berm and rip rap placement as excavation to rock (elev. 558) progressed. Once the rip rap and containment berm were complete, the spillway was grouted using a 60 meter concrete pump truck and 3:1 grout mix.

3.2.1.3 Equipment Bridge
The Equipment Bridge provides access from Cell No. 13 to Cell No. 14 over the connecting arc cell which comprises the Emergency Floodgate Structure. This bridge rests on a support system fabricated on site and seated into the concrete in the connecting arc. The bridge consists of double stacked wooden timbers with a portable concrete highway barrier lining both sides to serve as fall protection as is common on top of all cells and connecting arcs. This work is constructed with the 110 Ton Link Belt Crawler Crane located on top of Cell No. 14.

3.3 Dewatering and Instrumentation
Construction required excavation immediately adjacent to the Willow Island Dam to a depth of about 100 feet below the normal water level (Elev. 602) in the upper pool of the Ohio River. Excavation was planned within a cofferdam consisting of a cellular cofferdam along the river and an earthen cofferdam on land that extended from the upper pool into the lower pool. The earthen cofferdam included a soil-bentonite cutoff wall penetrating to the top of the underlying bedrock to reduce groundwater flow through the excavation slopes and the impacts of variations in river stage due to flooding.

Dewatering was required to remove groundwater storage within the cofferdam, leakage through the cutoff wall and underlying bedrock, and precipitation falling within the excavation boundaries. Installation of a system of dewatering wells was planned to lower groundwater levels to several feet above the top of rock (about Elev. 558) within the excavation and as otherwise necessary to maintain stability of excavated slopes. Dewatering well screens are 12 inch diameter slotted steel pipe with solid steel pipe risers. Surficial supplemental systems, including finger and toe drains, sumps, and/or other dewatering devices were installed to further draw down groundwater to the surface of the rock and below as the excavation continued to final grade at Elev. 500, about 60 feet below the top of rock.

After completion of the riverside cofferdam and soil bentonite wall, the dewatering system consisted of fourteen (14) deep wells and nine (9) standard wells installed
around the inside perimeter of the cofferdam as well as in Cell 5 on the upstream side. These deep wells draw down the ground water within the confines of the excavation and associated bentonite wall. The standard wells extend through the overburden soil and are seated approximately seven (7) feet into bedrock. The deep wells extend to depths between elevation 494 and 498. The system is capable of discharging up to 3,000 gallons per minute (gpm) to the designated discharge point. The water is collected and discharged through a Dewatering System Treatment Plant (DSTP), in accordance with the WVDEP permit. Dewatering is a continuous operation without interruption. Trained personnel and standby equipment are available to maintain the dewatering system throughout the duration of construction. A 10% surplus of all pumping equipment is maintained on-site for back-up in case of failure. In addition, to provide back-up power in the event of a service disruption, a diesel powered Caterpillar C27, 750 EKW, 938KVA generator is provided.

3.3.1 Construction Methods
3.3.1.1 Initial Unwatering
The initial unwatering removed trapped river water located within the cofferdam. This unwatering was performed using trailer mounted pumps and the discharge was filtered with a sediment control bag and discharged back to the Ohio River.

Prior to unwatering the excavation to elevation 580, the following work items were completed:

- All piezometers were operational
- Emergency floodgate between cells 13 & 14 were installed
- Landside slurry trench and sheet pile cutoff wall were complete
- Landside cofferdam was completed to elevation 624.5
- Bay 8 Closure, except for rakers to bottom of Bay 8, was completed
- Underwater cell weighting berm was complete to elevation 580

Additionally, during the initial unwatering to elevation 580, piezometers installed in the cells were monitored to ensure the water in the cofferdam and saturated cell fill did not exceed the normal pool elevation. Should this have occurred, unwatering would have been stopped until water levels within the cell fell back to normal pool elevation.

Final unwatering occurred once the weighting berm was complete to elevation 600. After the weighting berm was completed, the cofferdam was unwatered to elevation 558, the approximate top of rock.

3.3.1.2 Dewatering to Bedrock
After the weighting berm and impervious fill were constructed to elevations 600 and 624 respectively, dewatering wells were placed in the slopes and atop of the weighted berm.
and landside cofferdam. Wells were constructed using a Gus Pech bucket auger drill rig and down hole hammer into the rock formation. Wells consisted of a twenty four (24) inch bore hole advancing to bedrock. If the auger drill rig reaches refusal before reaching design depth, the down hole hammer was used to drill to depth. The verticality was maintained through monitoring the work pipe with a four foot level. Drilling was achieved using clear water as drilling additive, and depth was confirmed by measuring the work pipe from known elevations. The contractors QC verified depths and provided a written report to the Owner’s Engineer. The soil cuttings from the drilling were contained within the excavation area, as well as discharge from pump tests. A twelve (12) inch slotted well screen and twelve inch steel casing, with welded joints was installed. Centralizers were used to insure the casing was centered in the bore hole. Upon completion of the screen and casing installation, the filter sand was placed between the well screen and drilled hole, up to a depth approximately five feet below ground surface. A one foot thick bentonite seal was placed above the sand, and the remainder of the hole filled with a cement-bentonite grout. The depth of the screen, filter sand, and bentonite seal was taped for known elevations.

Inside the casing pipe, a pump capable of pumping 150 gpm at 100 foot total dynamic head, with a three (3) inch PVC riser pipe was lowered into the well. A suspension cable of polypropylene rope at least five feet longer than the pump column was attached to the pump and affixed to the top of well. Pumps were hardwired to electrical drops and included a backup generator should a power outage occur. The development of each well was accomplished by using air lift method and over pumping until discharge was visually clear. After development, the well head was installed, consisting of the port for water sampling and rossum sand tester, 0 to 150 GPM flow meter, and gate valve. The water was pumped to a fourteen (14) inch HDPE discharge line that was connected to the DSTP. Once treated the water was discharged into Cow Creek.

After well construction was complete, a 6 hour step drawdown test was performed on the pumping well to evaluate well performance and pumping rates for the constant test. Following the recovery of groundwater levels from the step drawdown test, a constant rate pumping test was performed for 72 hours to determine that the means and methods employed met the minimum well efficiency requirement of 65%.

These wells allowed the excavation to be performed in the dry. The wells drew the water elevation inside the excavation down to approximate top of bedrock (elevation 558). In addition, the groundwater was maintained three (3) feet below the toe of slope in the general excavation. If the measured water level did not meet the above stated criteria, the excavation would have been stopped, until dewatering draw down obtained required elevation. Once the overburden soil excavation was complete, the operating
wells maintained seepage that occurred from the riverside and landside cofferdam during rock excavation as well as Contract W3, powerhouse construction.

### 3.3.1.3 Dewatering below Bedrock
With the deep wells controlling seepage entering the excavation from the river and landside cofferdams, localized sumps were installed to remove any additional leakage and precipitation falling within the excavation boundaries. A toe drain collector was located at the top of rock elevation and bottom of overburden slope around the landside portion of the excavation. The toe drain was constructed with drainage stone and a berm of clay material that directed flow to catch basin sumps. The lower dewatering system consisted of open sumps and deep wells as the excavation of rock proceeded from elevation 558 to elevation 500. The first step of constructing these local sumps was to excavate the sump hole using a CAT 345 excavator, in a lower lying area. Once excavated, a 7.5 HP submersible pump, or larger if needed, was piped through 3" HDPE pipe to the top of the cofferdam excavation. The sump was backfilled with #57 gravel. The discharged water from the rock sumps was not treated through the DSTP, instead, the effluent was filtered with a sediment control bag and discharged back to the Ohio River. The pumps were operated as necessary to allow work to be performed in the dry. Due to the amount of groundwater encountered multiple sumps were required and were installed as the excavation progressed. Once the excavation was completed to elevation 500, a pair of eight (8) inch core holes advancing an additional five (5) feet into bedrock were installed. Two 7.5 HP submersible pumps were piped to the top of the cofferdam excavation. These were installed and hardwired to permanent power. These were situated in the lowest part of the excavation and self-operate as needed to maintain the excavation in a dry condition.

### 3.4 Rock Anchors
The anchor locations were selected based on analyses in DDR 7 and DDR 12 to provide deep seated stability of the rock excavation while maintaining the stability of the cells. Rock anchors were included in all locations where stability calculations showed safety factors below the required minimums. Rock anchors were placed along the north (riverside), east (upstream), and west (downstream) sides of the rock excavation to improve safety factors for slope stability in rock and fault block support. In front of the dam, no rock anchor load was needed for sliding stability. The anchors have been designed to carry the fault block load. See Figure 1.2 for the rock anchor layout.

#### 3.4.1 Construction Methods
**3.4.1.1 Anchor Block Excavation**
The Willow Island Hydroelectric Project required the excavation and removal of approximately 1,500 cubic yards of rock for construction of the rock anchor blocks. The
primary and initial efforts for excavation of the blocks included line drilling the perimeter and breaking down the rock with a percussion hammer fitted to a CAT 345 excavator. All material was excavated in accordance with contract drawings to achieve the lines and grades shown on the Drawings. The excavated material was loaded into a Volvo 35 ton articulated dump and hauled to spoil area 5.

3.4.1.2 Anchor Block Construction
This phase involved constructing the concrete anchor block and recess at each anchor location prior to the drilling subcontractors mobilization to that anchor location. With the aid of a crane or CAT excavator, pre-tied reinforcing steel was installed according to the plan drawings. With the completion of reinforcing steel, EFCO Plate Girder forms were placed to meet the upper slope dimensions of the anchor block as shown on the plan drawings. The bottom and sides of the anchor block were cast against the clean rock face of the block excavation. The concrete placement method utilized a tremie pipe. Forming, rebar erection, and concrete placement was assisted by a 60 ton Grove crane or a CAT 345 excavator. A sub-bearing plate / trumpet assembly was cast in place at each anchor location. Proper layout of this sub-bearing plate ensured the accuracy of the plan location of the top of the drill hole and hole alignment. Sleeved trumpets were attached to the EFCO plate girder form to ensure stability while placing concrete. Concrete was then placed, forms stripped, and concrete spray cured. Trash pumps were used to remove any water entering the excavated anchor block location prior, during, and after concrete placement.

3.4.1.3 Rock Anchor Installation
The installation of the rock anchors was subcontracted to an experienced specialty subcontractor. Access to the work areas was accomplished through utilizing a combination of ramps, benches, and temporary shoring. Installation of anchor holes started with drilling the two (2) pre-production anchors and then moving to the production anchor locations. Rotary-percussive drilling methods with air and/or water flush were utilized. See Figure 3.4 Rock Anchor Installation photo. The pre-production and production anchors were drilled, installed, and tested per the methods outlined in specification. Upon completion of testing, lock off the anchors, perform lift–off tests and restressing were performed as necessary. Engineering oversite provided monitoring and inspection as required during the installation process.
3.5 Fault Exposure During Excavation

The site geology contains a fault that was identified during initial design. The fault is located under the USACE dam structure was encountered during construction of the Willow Island Dam. Since geotechnical investigation could not completely delineate the location and condition of the fault zone, it required further investigation and engineering once the fault was encountered and exposed. As the rock excavation advanced the fault zone became visible due to rock plane dipping and loose broken rock in the zone. See Figure 3.5.

The loose rock was removed and the rock surfaces exposed back to solid rock in the fault zone. The cleaning process extended back into the excavated face sufficiently to ensure any unconsolidated material was removed. During initial excavation and fault cleanout, small movements of the rock immediately above the fault were observed by the instrumentation system, and the two rock anchors above lost about 15 percent of their prestress as the rock they were founded on was undermined. To alleviate concern that blocks of rock could loosen during the ensuing construction period before the area is backfilled, rock bolts and shotcrete were installed prior to filling the remaining fault gap with concrete (movement and anchor loads stabilized once the fault repair was complete). Concrete forms were placed perpendicular to the rock cut face and tied to the rock bolts. See Figure 3.6.
Figure 3.6 Fault with rock bolts and shotcrete

Once the fault was stabilized, excavation in the area proceeded to plan grades for the intake of the powerhouse. As the powerhouse concrete placement advances, the concrete will be placed against the rock and fault face, further stabilizing the rock. See Figure 3.7

Figure 3.7 Excavation near stabilized fault zone
4. Cofferdam Removal
4.1 Cofferdam Removal

At the time of this writing, the cofferdam at the Willow Island Hydroelectric Project is providing 100 year flood level protection and an emergency flooding structure is in place on the downstream side of the cofferdam to enable emergency flooding in accordance with the project Temporary Construction Emergency Action Plan (TCEAP) in the event that a larger flood event is predicted. Once powerhouse construction reaches the “Powerhouse Watertight Milestone”, the level of flood protection provided by the cofferdam will be sequentially lowered in stages to allow removal of weighting berms, SOE wall and tiebacks, rock anchors, removal of Bay 8 fill, inspection of Bay 8 spillway, closing of Bay 8 gate, and completion of the remaining excavation for the approach and tailrace channels. Final removal of the temporary cofferdam will be performed after the cofferdam’s interior has been filled in a controlled manner by pumping river water into the upstream and downstream sides of the cofferdam. Reduced design river levels have been established for the cofferdam to maintain USACE safety factors for each stage of cofferdam decommissioning.

Figure 4.1 March 2014 aerial of site

The sequence of cofferdam decommissioning, including the completion of remaining excavation work and controlled filling of the cofferdam’s interior will be accomplished in
eleven stages of work. Should river levels be predicted to exceed the design river levels, controlled filling will be provided by high capacity pumps both upstream (U/S) and downstream (D/S) of the powerhouse in accordance with the revised project specific Temporary Construction Emergency Action Plan (TCEAP).

Stage I thru Stage III reflects the completion of excavation within the cofferdams while 100 year flood protection remains in place. Stage IV is the work needed to obtain the “Powerhouse Watertight Milestone”. Stages V and VI reflect the extent of additional excavations performed with reduced design river levels for the cofferdam. Stage VII reflects the excavation in Bay 8 with further reduced design river levels. Stage VIII reflects the partial removal of D/S embankment, D/S rock anchors and D/S tailrace rock excavation under further reduced design river levels. Stage IX reflects the controlled filling of the cofferdam and completion of the remaining weighting berm excavations. Stage X reflects removal of cells and completion of approach and tailrace channels after the cofferdam is filled with water. Stage XI reflects the final excavation work after removal of the cofferdam cells.

The sequence includes modifications to the cofferdam to allow excavation and grading work to proceed in the dry to the extent practical.

4.2 Cofferdam Removal Sequence

Once the powerhouse and closure structures are completed above the 100 year flood level, cofferdam decommissioning will commence. Bulkheads will be installed in the intake bulkhead slots, emergency closure gates closed, and the roof hatches will be complete. During cofferdam decommissioning, the powerhouse itself, after obtaining Powerhouse Watertight Milestone, will provide separation of the upstream and downstream pools. Controlled filling of the cofferdam will be performed with high capacity pumps. If necessary, these high capacity pumps would also be used for controlled filling of the cofferdam in advance of rising river levels in the event they were predicted to exceed the cofferdam design level during any given stage of cofferdam decommissioning.

The cofferdam will be removed in eleven stages, summarized as follows:

4.2.1 Stage I
Complete landside closure structure, which fills the gap between the new powerhouse structure and the existing dam structure.

4.2.2 Stage II
Perform dredging outside of the cofferdam for approach and tailrace channels to within 20’ of the cellular cofferdam. Inside the cofferdam controlled rock excavation to the
project neat lines occurs, and final grading, together with geogrid and riprap placement, will be completed on the landside of the cofferdam as well.

4.2.3 Stage III
Continue rock excavation of the intake channel on the upstream side of the powerhouse. On the downstream side, rock excavation will be completed to the plan limits.

4.2.4 Stage IV
At this stage in the decommissioning the powerhouse is fitted with intake bulkheads and emergency closure gates and roof hatches are ready for installation. Upon completion of the landside and riverside closure structures and the powerhouse verified as watertight the Powerhouse Watertight Milestone is reached.

4.2.5 Stage V
At this stage in the decommissioning the cofferdam will only provide protection to river EL 609 U/S and EL 607.5 D/S. Portions of the cellular cofferdam support berm will be removed on both the upstream and downstream sides, together with the U/S SOE walls and tiebacks. Interim support berm configurations and cell stability are established. Rock anchor foundations will be abandoned in place having tendons cut off and the remaining anchor block will be filled with concrete to the final grade.

At this point, the dewatering wells that are in the cell support berms will be removed, along with all instrumentation in the berms. Dewatering and water level monitoring will continue through the wells and piezometers in the cells, and any water infiltrating into the site will be managed by sumping.

4.2.6 Stage VI
On the upstream side of the cofferdam, any remaining sheet pile within the excavated footprint will be removed or cut off to final elevation, including the existing sheet pile obstruction. In addition, the Cytek outfall pipe tie-in will be installed at this time.

On the downstream side, the embankment cofferdam will be removed down to EL 610 to allow removal of the SOE wall and tiebacks.

4.2.7 Stage VII
The design river levels are reduced to EL 602 U/S and EL 600.5 D/S. The Bay 8 support berm will be removed, the Bay inspected and the gate closed.

4.2.8 Stage VIII
The design river levels are further reduced to EL 602 U/S and EL 594.7 D/S. The downstream embankment is lowered to EL 600, the downstream rock will be detensioned and anchor blocks removed and the remaining tailrace rock excavation
completed in the dry. Final clean up excavation and rip rap placement will be performed prior to the controlled filling.

4.2.9 Stage IX
Controlled water filling of the remaining cofferdam is performed (EL 602 U/S and EL 582 D/S) and the performance of the landside closure, riverside closure and powerhouse intake bulkheads and emergency closure gates evaluated.

4.2.10 Stage X
After the controlled water filling is complete, water levels inside the cofferdam will be equalized to match existing downstream river elevation. The Cells will be removed, beginning at the dam tie-in cells and working toward shore. Equipment on top of the cofferdam will remove cell fill and disassemble the cells.

4.2.11 Stage XI
Once the cells are removed, any remaining underwater excavation will be completed, and slopes dressed with rip rap and geogrid where needed. At this point the powerhouse will be ready for final commissioning and operation.

4.3 Design Criteria
The design criteria used in developing the cofferdam removal staging sequence include:

4.3.1 River water levels
The design river levels were selected to provide the requisite USACE factor of safety during each stage of decommissioning.

If the river levels are predicted to exceed the design values, the cofferdam will be filled to equalize the water level on the interior and exterior of the cofferdam. The filling will be accomplished using external pumps (three upstream and three downstream) as described in the section below.

If the cofferdam needs to be filled during decommissioning, a maximum upstream to downstream head differential of 20 feet will not be exceeded in accordance with the Contract criteria. The filling will be controlled and inside the cofferdam the downstream pool level will not rise above the upstream pool during filling.

4.3.2 Controlled Filling of Water
Controlled filling will be performed with six 3,700 GPM pumps (three U/S and three D/S). The discharge hoses will run continuously to the bottom of the excavation and will be secured during filling. Procedures for controlled filling in the event of a predicted exceedance of design water levels are included in the modified TCEAP.
4.3.3 Slope stability
Analyses were performed in accordance with the US Army Corps of Engineers’ Engineering Manual EM 1110-2-1902 Slope Stability. These calculations address several conditions that develop as part of the decommissioning. Prior investigation of slope stability was addressed in DDR 12 Rock Anchors and DDR 14 100 Year Flood Protection. The minimum factor of safety against slope stability failure for these temporary conditions is 1.3.

4.3.3.1 Reduced Weighting Berm Soil Stability
During Stages V through VII of cofferdam decommissioning, design river levels are reduced from 100 year flood levels to EL 609 upstream and EL 607.5 downstream. At this time, the cellular support berm will be partially removed. Slope stability analysis results of the reconfigured support berm slopes demonstrates that required safety factors are met both upstream and downstream.

4.3.3.2 Downstream embankment cofferdam reduced to EL 610
Stage VI of the decommissioning process includes the removal of the downstream SOE wall. The embankment cofferdam behind this wall will be cut down to EL 610 (design water level at this time is EL 607.5) to allow SOE decommissioning while maintaining stability. Stability analyses have been performed using limit equilibrium methods to demonstrate that both soil and rock remain stable in this configuration.

4.3.3.3 Downstream embankment cofferdam reduced to EL 600
In the final steps of decommissioning before the cofferdam is flooded (Stage VIII), downstream anchor foundations will be decommissioned and final tailrace channel rock excavation will be completed. Anchor foundations will be removed during excavation and the anchors cutoff at the final tailrace grade Design river levels are further lowered to EL 594.7 downstream during this portion of the work. Stability analyses have been performed for both the soil slopes and final rock cut to confirm that required factors of safety are met.

4.3.4 Pier 8 Stability
During the decommissioning analysis of Pier 8 and Bay 8, we have conservatively neglected any resistance provided by the spillway and are treating Pier 8 as an independent structure. The stability analysis investigates the stages of berm removal during decommissioning to establish a sequence of removal and when the tainter gate can be closed. Analyses conform to the criteria described in the USACE EM 1110-2-2100 (Stability Analysis of Concrete Structures).
4.3.5 Pier 8 Stress
The reinforcing was checked based on the procedures contained in the USACE EM 1110-2-2104 (Strength Design for Reinforced Concrete Hydraulic Structures). Following the EM method at Pier 8, stress analyses were performed at each construction joint. Our analysis of the Pier 8 construction joints found the known reinforcing to have sufficient capacity to resist the calculated stresses.

4.3.6 Cell stability
Due to the addition of active pumping to all cells that were not fully drained during the powerhouse construction, we have modified the assumed water level within cells for the temporary stability analysis. The modified water levels conform to the observed water levels established by the piezometer data. Cell water levels have been successfully maintained at near the top of rock since the start of active pumping. For most cases, the decommissioning analysis uses a design cell water level of EL 570.0. The two cells (13 and 14) flanking the downstream emergency flooding spillway have both shown higher water levels during operation. In those cells, a water level of EL 580 has been assumed.

Interior support berms have been reduced to the extent possible while still providing the USACE required safety factors under the different design river levels at various stages. We have prepared three analysis sections to demonstrate cell stability during decommissioning. They are upstream, downstream and downstream at cells 13 and 14. See calculation sections 3.5.1 through 3.5.6 for the stability analysis at each section for the two design river levels and berm configurations.

4.3.7 Downstream Berm and Cutoff Wall
The embankment cofferdam and soil bentonite cutoff wall are the landside continuation of the cellular cofferdam. The present configuration of the embankment overlaps the proposed rock excavation for the tailrace channel. The embankment will be lowered to allow additional excavation to proceed in the dry. The modification will be executed in the final steps prior to controlled filling. The berm modification will begin in stage VI and be completed in stage VIII.

4.4 Decommissioning Criteria
Similar construction methods and equipment utilized for installing the cells will be used for removing them. The design approach is to allow as much excavation work as possible to be accomplished in the dry during the staged decommissioning of the cofferdam structure.
4.4.1 Removal Stages
Stages I – III excavation to plan grades and rip rap placement occurs in the dry using hydraulic excavators, dozers and articulating trucks.

Stages VI – VIII excavation of portions of weighted berm and cell support berm material occurs within the cofferdam protection using hydraulic excavators, dozers and articulating trucks. Localized pumping of water will be used to keep the area dewatered during decommissioning of the pumping system.

Stages IX – XI begins with controlled water filling of the cofferdam subsequent to the powerhouse becoming water tight. After the cofferdam is watered up, removal of cell fill and sheet pile cells will commence.

4.4.2 Removal Methods
The cells will be removed beginning at the dam tie-in cells working towards shore both the U/S and D/S portions, thereby maintaining access along the top of the cells back to shore. A Cat 349 hydraulic excavator will be used to remove sand down to the river water level, loading it into articulating dump trucks for transport to the spoil area. At this point the same cell template used for constructing the cells will be inserted back into the cell to ensure it is stable while removing the remaining cell fill with a 250 Ton crawler crane and clam bucket. The sheet pile will be removed from outside the template using a 250 Ton crawler crane. The connecting arcs will be removed before the leading circular cell is. Sheet pile will be loaded onto a material barge in the river for disposition.

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