November 6, 2014

Ms. Judith Hansen, Superintendent
Kingston Water Department
111 Jansen Avenue
Kingston, NY 12402

Subject: **Cooper Lake Dam & West Dike – Preliminary Engineering Phase 1 Technical Memorandum (Schnabel Reference 14925003)**

Dear Ms. Hansen:

SCHNABEL ENGINEERING OF NEW YORK (Schnabel) is pleased to present this Technical Memorandum to the Kingston Water Department Board of Water Commissioners (KWD) in accordance with the scope of work for Preliminary Engineering Phase 1 of the Improvements to Cooper Lake Dam & West Dike project, dated June 10, 2014, and amended August 12, 2014. Schnabel and our subconsultants CDM Smith and Hunt Underwater Specialties (Hunt) have substantially completed Tasks 1 through 4 corresponding to the Records Review, Site Visit, Intake Tower Inspection and Structural Evaluation, and Review of the Engineering Assessment (EA) for Cooper Lake Dam & West Dike completed by Malcolm Pirnie/ARCADIS in August 2012 (Revised April 2013).

The findings from these efforts are detailed in this Technical Memorandum. This Technical Memorandum begins with a Summary of Findings followed by detailed discussions focusing on the Intake Tower Evaluation, and our independent review of the EA; and concludes with an updated engineer's opinion of probable construction costs (EOPCC) for specific elements of the proposed project.

SUMMARY OF FINDINGS

The Schnabel team has substantially completed Tasks 1 through 4 in accordance with the scope of work for Preliminary Engineering Phase 1 of the Improvements to Cooper Lake Dam & West Dike project. The following items represent the substantial findings of this work.

1. An underwater inspection of the intake tower was performed which concluded that the condition of the intake tower is suitable for further consideration as a long-term component of the water supply system. Specific related activities and findings are detailed below.
    a. This underwater inspection was facilitated by a change order allowing for the installation of an insertion valve on the raw water transmission main. The valve was installed on September 4, 2014, and utilized to isolate (stop) the flow in order to provide a safe underwater working environment.

b. The underwater investigation proceeded through eight separate mobilizations beginning on September 19 and concluding on October 28, 2014.
c. The investigations included the assessment and removal of inoperable gates (gates, stems, and operators). The intake and outlet gates at the lowest elevation of approximately 1068 ft were both found to be partially open. This finding indicates that the low-level outlet has been charged (i.e., under full headwater pressure) through the embankment, most likely to at least one (presumed) closed valve in the manhole complex at the downstream toe. This is a significant finding, and it is our recommendation that the KWD needs to remedy this condition as soon as possible.
d. Gate components were removed from the intake pipes located at elevation 1086 ft, 1095 ft, and 1068 ft. Gate components were removed from outlet pipes at elevations 1068 ft and 1086 ft.
e. The investigation included the collection of five core samples (three from the walls and two from the floor). The wall core samples indicate that the lower portion of the intake tower is constructed of bluestone masonry, and the upper portions are constructed of concrete. Three of the concrete core samples were analyzed and found to be of sufficient strength for the tower to be considered for continued use. The floor thickness was also investigated and determined to be greater than 42 inches.

2. A review and refinement of the Hydrologic and Hydraulic modeling presented in the EA was conducted through an independent assessment. Our updated model was based on a recently released 2-meter Digital Elevation Model (DEM) covering portions of Ulster County including the Cooper Lake watershed area. The findings of the revised modeling are consistent with the EA, and confirmed the conclusion that the existing spillway has inadequate capacity to convey the regulatory SDF without overtopping of the Main Dam and West Dike embankments.

3. Two additional spillway capacity alternatives were investigated in lieu of the labyrinth presented in the EA. Based on our analysis, we are recommending advancing the concept of raising the crest of the Main Dam and West Dike one foot to EL 1109.0 while maintaining the existing normal pool elevation. This alternative will take advantage of the increase in the surcharge storage capacity in Cooper Lake and significantly reduce the size of the constructed spillway at the Main Dam. The concept assumes the Main Dam will be an earthfill raise and the West Dike would be raised via the installation of a short parapet. The new spillway would consist of a concrete-encased 36-inch diameter reinforced concrete pipe (RCP), a drop inlet spillway with a weir length of 4 ft, and a USBR Type VI impact basin to provide energy dissipation prior to discharging to the downstream drainageway.

4. An independent review of the Geotechnical Evaluations presented in the EA was performed for the Main Dam and West Dike. We performed slope stability analyses for the maximum embankment sections using the SLOPE/W computer program, a product of GeoStudio 2012, which employs various two-dimensional limiting equilibrium methods of analysis. Specific related activities and findings are detailed below.
    a. Our opinion of a representative drained (effective) angle of internal friction assumed for the embankment fill materials for the Main Dam and West Dike differs from that presented in the EA (the EA presented a value of 34 degrees for each). While selection of the higher value presented in the EA may be justified in some locations, the variability

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of the embankment soil type and densities encountered in the borings, and the very limited shear strength laboratory data presented in the EA, do not justify its use as a representative value of the entire Main Dam and West Dike.

b. Significant previous geotechnical explorations have been conducted at the Main Dam and West Dike. While these have provided substantial and valuable information, none has included appropriate or adequately documented laboratory shear strength testing to establish a technically defensible effective angle of internal friction.
c. In consideration of the variability and uncertainty associated with the effective angle of internal friction, we evaluated the sensitivity of the factors of safety (FOS) with the Main Dam slope stability model by varying the drained friction angle of the embankment fill within a reasonable range of values. Consistent with the overall findings from the EA, the results of our analyses confirmed that the Main Dam (existing configuration) is stable but does not meet NYS DEC stability criteria for the normal pool condition.
d. The EA recommended buttressing (or flattening) the downstream slope of the Main Dam from 1.9H:1V to 2.5H:1V to address inadequate FOS for slope stability. We agree that flattening the downstream slope is an appropriate method to achieve the requisite minimum FOS for slope stability at the Main Dam, but that the slope should be further flattened to 3.0H:1V to accommodate the variability and uncertainty in appropriate shear strength design parameters.
e. The EA also recommended the installation of a new filtered drainage blanket and toe drain system for the Main Dam. We agree that this is a necessary requirement considering no engineered internal drainage system is known to exist.
f. It is our opinion that the piezometric levels used in the EA's West Dike stability model represent conditions in the foundation soils rather than the embankment, and that the low piezometer readings may not represent pore water pressures for the upper zone of embankment soils, which control slope stability. Therefore, our slope stability analysis considered a higher piezometric level based on the water levels encountered during the 1984 drilling program, and more typical for an embankment constructed without an internal drainage system.
g. We evaluated the sensitivity of the FOS with the slope stability model for the West Dike by varying the friction angle of the embankment fill within a potential range of values, but with a more representative piezometric level. The results of the sensitivity analysis indicate that the West Dike (existing configuration) is stable but does not meet NYS DEC stability criteria for the normal pool condition. This represents a significant difference from the findings presented in the EA which suggested no upgrades to the West Dike were necessary aside from leveling the crest to EL 1108.
h. As part of the next phase of this project, it is recommended that conceptual design alternatives be developed to improve slope stability of the West Dike. This may consist of slope flattening and/or the installation of an internal drainage system.
i. As part of the next phase of this project, it is recommended that a targeted field exploration program be performed to collect additional data at the Main Dam and West Dike. This field program is intended to establish a cost effective and defendable set of design parameters (primarily shear strength data for both the Main Dam and the West Dike), and to confirm representative embankment soil piezometric levels for the West Dike. These additional activities coupled with the previous investigations will allow us to

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determine an appropriate and representative basis of design for the Main Dam and the West Dike.

5. An Engineers Opinion of Probable Construction Costs (EOPPC) was developed as part of the EA consistent with a Class 5 estimate as defined by the Association for the Advancement of Cost Engineering (AACE). Class 5 cost estimates are typically accurate on the low side between -20 percent and -50 percent, and on the high side between +30 percent and +100 percent. Independent EOPCC were developed for the recommended spillway capacity alternative and Main Dam embankment stability improvements. Our estimates were advanced to a Class 4 estimate as defined by the AACE. Class 4 cost estimates are typically accurate on the low side between -15 percent and -30 percent, and on the high side between +20 percent and +50 percent. Specific cost findings are summarized below:
    a. The point cost estimate for the recommended spillway capacity improvements (incremental Main Dam raise, the West Dike parapet wall, and the new spillway system and energy dissipation structure) is $1,270,000. This represents an estimated savings of approximately $580,000 relative to costs estimated for the spillway upgrading alternative recommended in the EA.
    b. The point cost estimate for the Main Dam stability improvements (flattening slope to 3:0H:1V and toe drain system) is $930,000. This represents an estimated reduction of approximately $270,000 relative to costs estimated in the EA despite our recommendation to further flatten the slope to 3H:1V. This reduction is likely due to the advancement of project detail and incorporation of unit rates consistent with recent or ongoing projects we have been involved in within the northeast.
    c. No advancement of cost considerations was performed for the intake structure: refurbished or new. These evaluations will be performed as part of the next phase of work.
    d. No costs were developed for remedial measures necessary to bring the West Dike into regulatory compliance. These evaluations will also be performed as part of the next phase of work.

INTAKE TOWER INSPECTION AND STRUCTURAL EVALUATION

The EA identified outlet works improvement items necessary to restore the functionality of the raw water intake system, improve drawdown capabilities (in compliance with the dam safety regulations), and to eliminate a potential failure mode related to the aged piping that passes through the main embankment. The outlet works design concept presented in the EA was contingent on the utilization and modification of the existing intake tower, as well as the intake piping that extends into the reservoir. As all of these components have been in service for well over 80 years, it was critical to assess the condition of these facilities and their ability to provide continued long-term service to the KWD before endorsing their use or considering a complete replacement of the facility. The primary unknown condition was the structural integrity of the intake tower and intake piping, instigating the need to assess their long term viability for continued service. Schnabel subcontracted with Hunt, a specialty diving contractor, to perform underwater investigations and demolition as specified in our scope of work. Through the investigations performed to date, we have concluded that the condition of the intake tower is suitable for further consideration as a long-term component of the water supply system. Further evaluation on modifying this structure will be included in the work associated with Preliminary Engineering Phase 2.

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The investigation proceeded through eight separate mobilizations beginning on September 19 and concluding on October 28, 2014. For convenience we have provided commentary on these activities following the tasks identified in the scope of work.

Preparation of a Safe Work Plan

The intent of this work was to develop and document the operational requirements for safely performing the underwater work. Early on in the investigation it was determined that there was no reliable method to isolate the flow from Cooper Lake in order to provide a safe diving environment. This required that the KWD execute additional contracts for the selection and installation of an insertion valve on the raw water transmission main. The insertion valve was purchased from and installed on September 4, 2014, by Garrison Enterprises from Vineland, New Jersey, with the site work performed by Baker Brothers Excavating from West Hurley, New York. Engineering was performed by Schnabel. This valve was subsequently operated by KWD staff to isolate flow from Cooper Lake and to rapidly “flush” the water in the intake tower to improve underwater visibility. A pipe coupon was removed during the installation of the valve. Visual inspection of the coupon showed that the cast iron pipe is in very good condition. The insertion valve installation is shown as Photo 1.

Photo 1: Installation of Insertion Valve

Visual Inspection, and Removal and Disposal of Equipment

The intent of this work was to perform a preliminary underwater visual inspection to identify the general condition of the intake structure, position of the gates, and presence and depth of sediment in advance of other activities. Due to the restricted access, this task was completed in conjunction with the removal and

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disposal of the equipment. Removal of the equipment at the upper levels was required to safely access the lower levels of the structure. Although successful, demolition progressed slower than anticipated because of the many obstructions within the tower. An additional half-day delay occurred on October 16, 2014, when the KWD distribution staff assisting with valve operations at the Main Dam were called away to address a water main break.

The stems, operators, gates, and guides at approximate elevation 1095 ft (intake) and 1086 ft (intake and outlet) have been removed. These were found to be in the full-open position as was expected. All three gates were circular in shape with a diameter of 20 inches, and generally at their anticipated elevation and location. There remains a remnant of the gate's flanged connection on each that, in the interest of time management, was not removed. Future removal of these is likely to reveal a flanged connection to the inlet pipe that could be utilized.

The intake and outlet gates at the lowest elevation of approximately 1068 ft were both found to be open by approximately six inches. It was our expectation was that these would both be closed. This finding indicates that the low-level outlet has been charged (i.e., under full headwater pressure) through the embankment, most likely to at least one (presumed) closed valve in the manhole complex at the downstream toe. This is a significant finding and it is our recommendation that the KWD needs to remedy this condition as soon as possible. Because both were found to be open, they could be removed without creating a changed condition within the intake tower or the embankment. Both operators were able to be slightly rotated, resulting in a bowing of the gate stem. Although a significant effort was made to close the downstream gate in an effort to eliminate the hydraulic connection between the reservoir and the low-level outlet pipe, neither gate could be restored to operation. Because the outlet gate could not be restored to operation or closed, it was removed along with its operator, stem, and gate guides in preparation for a more permanent closure alternative. As shown in Photo 2, the low-level outlet gate is rectangular in shape with a width of approximately 24 inches and height of approximately 26 inches. This rectangular opening transitions into a circular pipe.

Photo 2: Removed Low-Level Outlet Gate

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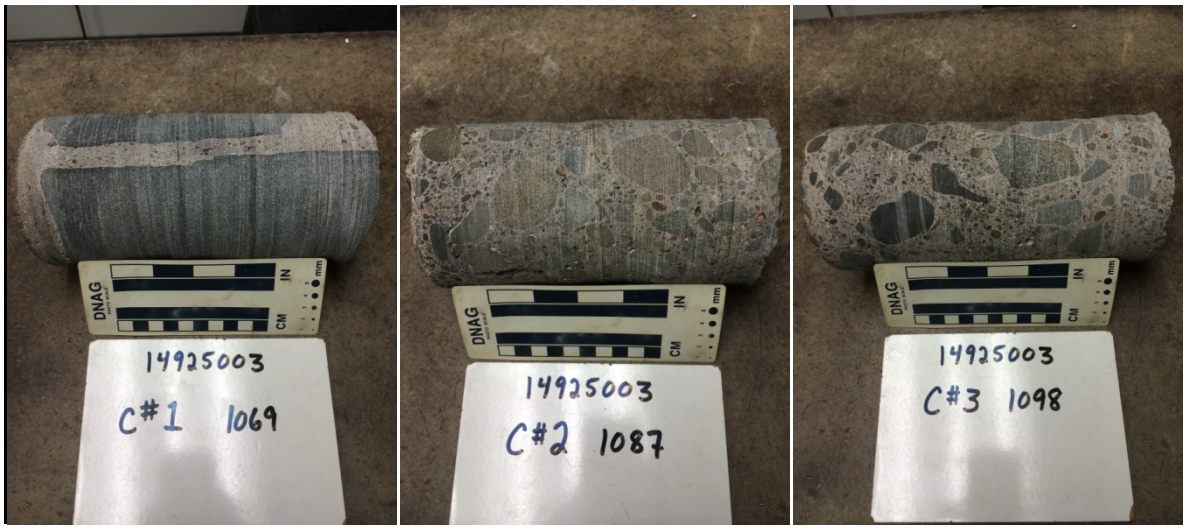
Although the low level outlet has been charged for many years, additional work should commence to plug it as soon as possible. The operator and stem for the lowest level intake pipe have also been removed, but in the interest of time management, the rectangular gate and gate guides remain in place. Future removal of these will likely reveal a flanged connection to the inlet that could be utilized.

Collection and Analysis of Core Samples

The upper portion of the tower is concrete with an internal diameter of 5'-6" at the top. Based on internal and external measurements, the upper intake tower walls are approximately 21-inches thick. The lower portion of the tower is masonry with an internal concrete skim coat, and was measured to be 5 ft in diameter at the bottom. It is likely the masonry walls are slightly thicker than the upper concrete sections, accounting for the reduced internal diameter.

The intent of this task was to collect three, 10-inch long and 4-inch diameter cores from the walls of the structure in order to assess the quality of its concrete and masonry. Based on our review of drawings of the structure, the original walls were anticipated to be masonry and 2-ft thick up to elevation 1095±, with a lightly reinforced concrete structure above (which is visible from the dam). Because physical measurement of the upper structure indicated a wall thickness of 21 inches (less than anticipated), the cores were reduced to 8-inches long. Cores were initially collected at elevations of approximately 1069 ft, 1087 ft, and 1098 ft. These cores are shown in Photos 3, 4, and 5.

Photos 3, 4, and 5: Concrete Cores Taken from the Internal Walls of the Intake Structure



The core samples confirmed that the lowest level is masonry (bluestone), and the mid and upper levels are concrete. No steel was encountered within the cores. The masonry/concrete interface occurs somewhere between elevations 1069 ft and 1087 ft, which is lower than was anticipated. All three cores showed the materials to be in very good condition with the aggregate (and large masonry stone) well bonded with the cement paste and mortar. At several locations adjacent to the bluestone masonry stones in Core #1, there appeared to be two distinct layers of mortar. The finer layer may indicate a grouting program sometime after original construction. Also, the aggregate in Core #2 is rounded, while the aggregate in Core #3 is angular, possibly indicating two separate past tower raisings or repair efforts. Careful examination of each also reveals the internal skim coat present throughout the structure. This

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concrete facing (skim coating), which appears to exist throughout the interior, obscures the exact location of the masonry/concrete interface.

Following removal of the core samples from the walls of the intake structure and preliminary structural analyses (see Assessment for Dewatering below), two additional core samples were taken from the floor. These cores are shown in Photos 6 and 7.

Photos 6 and 7: Photos of Concrete Cores Taken from the Floor of the Intake Structure

Similar to the wall samples, both floor core samples showed the materials to be in very good condition with the aggregate well bonded with the cement paste. All five samples were sent to Schnabel's lab in West Chester, Pennsylvania, for examination, and three were subject to compressive strength testing. The results of this testing indicate that the concrete is suitable for further consideration as a long-term component of the water supply system. Table 1 summarizes the details of the core samples collected.

Table 1: Intake Tower Concrete/Masonry Core Sample Summary

Sample No.	Location	Maximum Size of Aggregate/ Masonry	Unit Weight (pcf)	Unconfined Compressive Strength (psi)	Notes
C #1 – 1069	Taken from interior wall at location 1 o'clock(1) and elevation 1069 ft	Masonry stones >8"	Not tested	Not tested	Masonry stone was larger than core sample. Sample preserved.
C #2 – 1087	Taken from interior wall at location 1 o'clock(1) and elevation 1087 ft	1-inch rounded	149.9	7,880	
C #3 – 1098	Taken from interior wall at location 7 o'clock(1) and elevation 1098 ft	angular	147.8	9,440	
F-1	Taken from floor at 2 ft from inner wall at location 4 o'clock(1)	1.5 inches, angular	148.4	4,990	
F-2	Taken from floor at 1.5 ft from inner wall at location 10 o'clock(1)	1.5 inches, angular	Not tested	Not tested	Sample preserved.

(1) This reference is used to location the position of the core sample within the circular section. 12 o'clock is aligned with the window in the gate house.

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No documentation has been discovered that details the thickness of the intake tower floor or the composition of the subgrade material. A smaller, 1.5-inch diameter drilled hole was advanced through the floor of the intake tower at the location of core sample F-1 in an attempt to determine its thickness and the nature of the subgrade. This hole was advanced to a depth of 42 inches below the floor before the activity was stopped. Assuming the tower is founded on glacial till rather than bedrock (based on nearby borings), this appears to indicate that the concrete floor of the intake tower is at least 42-inches thick.

The three core holes in the tower walls have been patched with underwater grout; however, due to time limitations, the two core holes in the floor slab have not been plugged. It is recommended that the holes be patched after unwatering of the tower.

Removal and Disposal of Sediment

No significant sediments were discovered or removed. The intake tower had small amounts of very fine sediments on the floor that were mobilized by diver activity. Some demolition materials and items that dropped into the intake were subsequently removed.

ROV Inspection of the Raw Water Line

The intent of this task was to perform an ROV inspection of the raw water intake piping (openings) within Cooper Lake. From the visual inspection, it could be seen that these pipes are significantly tuberculated. Based on the condition of the cast iron gates and the condition of the coupon removed from the raw water transmission main, it is anticipated that the intake piping is in good condition beneath the tuberculation; therefore, we did not consider a need to perform an ROV inspection at this time. Because the gate to the outlet pipe at elevation 1068 ft has been removed, the KWD should consider performing an ROV of this line either as part of a future dry inspection or independently under wet conditions.

Assessment for Dewatering

The intent of this task was to perform a desktop assessment of the structure in consideration of performing a dry inspection. A preliminary structural analysis was performed that indicated the intake tower walls, assuming their condition are as represented by the cores, should be adequate to handle the hydrostatic pressure stresses associated with dewatering of the tower. Because of the intake tower's cylindrical shape, the maximum stresses are compressive and relatively small at around 50 psi. Both the concrete and masonry walls should be able to handle these compressive stresses. Initially there was some concern about the ability of the floor to withstand the uplift forces and the shear stresses around its perimeter. This concern prompted the collection of the core samples within the floor and the investigation of the floor's thickness. Assuming the floor is uniformly 42-inches thick and its condition is as represented by the cores, it would be adequate to resist the uplift forces and shear stresses.

At this point we have not identified a definitive way to plug the inlet piping in order to dewater the structure. As was mentioned earlier, there remains a remnant of each intake gate's flanged connection that was not removed. Future removal of these is likely to reveal a flanged connection to the inlet pipe that could be utilized for this purpose.

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Locating and Cleaning of Intake Screens

The intent of this task was to locate, clean, and mark the intake screens within the reservoir. In the interest of time, this work was not performed.

Dry Inspection

The intent of this task was to plug the active inlet piping for the purpose of dewatering the structure and to perform a dry inspection of the intake tower. This would have allowed the most comprehensive inspection of the structure. This activity was not performed, but it is recommended that the KWD continue advancing the intake tower inspection activities to accomplish the dry inspection.

REVIEW OF THE ENGINEERING ASSESSMENT (EA) FOR COOPER LAKE DAM & WEST DIKE

The Engineering Assessment (EA) for Cooper Lake Dam & West Dike was completed by Malcolm Pirnie/ARCADIS in August 2012 (Revised April 2013). The EA concluded that the Main Dam did not meet requisite factors of safety for slope stability, had inadequate spillway capacity, and did not have a reliable Low-Level Outlet (LLO). To address these deficiencies, the EA included conceptual design of remedial measures including:

- Leveling of the Main Dam crest and flattening of the downstream slope
- Leveling of West Dike crest
- Construction of a new labyrinth spillway, chute, and stilling basin
- New intake piping and valves/gates including LLO connected to existing intake tower
- Abandoning of existing outlet piping and valves

An Engineer's opinion of probable construction costs (EOPCC) was developed for the proposed improvements in accordance with a Class 5 construction cost estimate as defined by the Association for the Advancement of Cost Engineering. The point cost estimate was $4,300,000 for construction costs, $430,000 for engineering design and permitting, $430,000 for resident engineering services, providing a total cost of approximately $5,160,000.

Schnabel performed independent evaluations focusing on the hydraulics, hydrology, and geotechnical aspects of the Main Dam and West Dike as part of our review of the EA and to support development of a recommended approach toward achieving compliance with New York State Department of Environmental Conservation (NYS DEC) Dam Safety Regulations. As stated in our scope of work, our team utilized their collective experiences to challenge the conclusions of the EA. We have presented the basis of our opinion whether we concurred with previous findings or not and, where appropriate, detailed particular concerns with the previous analyses.

EA Hazard Class Assessment

The EA presented a section for Review of the Hazard Classification for Cooper Lake Dam. This section presented a summary of breach parameters and predicted peak flows resulting from a catastrophic failure of both the Cooper Lake Main Dam and the West Dike. Breach analyses were conducted for both Sunny Day and Wet Weather breach scenarios. The HEC-RAS unsteady flow hydraulic model was used to route the resulting breach hydrographs from each failure scenario through the downstream drainageway.

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The model results and inundation mapping indicate that numerous structures and critical infrastructure would be impacted from a failure of Cooper Lake Dam. Based on review of the dam breach models and inundation mapping, we concur with the EA that Cooper Lake Dam is appropriately classified as a Class C – High Hazard Dam.

EA Hydrologic and Modeling

The hydrologic analysis included in the EA was performed with the Corps of Engineers HEC-HMS model using the Soil Conservation Service (SCS) Unit Hydrograph methodology for runoff transformation, and the SCS Curve Number (CN) methodology to estimate hydrologic losses. The watershed was delineated on USGS 7.5-minute topographic mapping and found to be approximately 0.63 square miles. Soils and Land Use mapping were used to compute a composite SCS CN of 78. The SCS TR-55 methodology was used with USGS topographic mapping to compute a Time of Concentration (Tc) of approximately 0.35 hours.

As an existing Class C dam, Cooper Lake Dam is required to have a spillway capable of safely conveying the Spillway Design Flood (SDF) equivalent to 50 percent of the Probable Maximum Flood (PMF). The National Weather Service's Hydrometeorological Reports No. 51 and No. 52 were used to develop and distribute the Probable Maximum Precipitation (PMP) for Cooper Lake Dam. The 72-hour PMP rainfall depth computed was approximately 36.34 inches and the 6-hour PMP rainfall depth computed was approximately 25.36 inches. A stage-versus-discharge relationship was developed for the existing spillway for scenarios with and without the stop-logs in place. The HEC-HMS model was executed for the 6-hour and 72-hour 50 percent PMF storm events for scenarios with and without the spillway stop-logs in place. Note that the lowest points along the Main Dam and West Dike are approximately 1107.4 ft and 1107.2 ft, respectively. The model results from the EA are summarized in the Table 2.

Table 2: Summary of Hydrologic and Hydraulic Modeling Results for the Existing Conditions (from the EA)

Spillway Condition	Storm Event	Peak Elevation	Available Freeboard (ft)	Peak Inflow (cfs)	Peak Outflow (cfs)
No Stop-logs	6-hr SDF	1105.5	1.9	3,190	309
No Stop-logs	72-hr SDF	1106.0	1.4	3,285	438
With Stop-logs	6-hr SDF	1107.4	0.0	3,190	271
With Stop-logs	72-hr SDF	Overtop	N/A	3,285	404

The model results indicate that the existing spillway can safely convey the regulatory SDF without the stop-logs in place, but would overtop the Main Dam and West Dike with the stop-logs in place. As it is currently the goal of the KWD to operate the reservoir at the elevation established by the stop-logs in place, the modeling concluded that the existing spillway was inadequate to safely convey the SDF. A hydraulic analysis was also conducted of the existing spillway chute which concluded that it also would be overtopped during the SDF. Subsequently, the EA recommended the construction of a new spillway crest and spillway chute which is described in the following section.

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EA Proposed Spillway Design

The EA proposed spillway improvements included three primary components: Spillway Crest Modifications, Spillway Channel Reconstruction, and Energy Dissipation Structure.

Spillway Crest Modifications

The proposed spillway crest improvements include replacement of the wooden stop-log weir crest with a new concrete labyrinth spillway. The proposed labyrinth weir would have an overall width of approximately 30 ft and include two cycles. The crest of the spillway would have an elevation of 1105.6 to match the crest of the existing stop-log weir. The floor slab would have an elevation of 1102.0 which would provide an approach height of 3.6 ft. The conceptual design of the labyrinth weir used a magnification ratio (L/W) of 2.0 which provides a total weir length of 30 ft per cycle or a total length of approximately 60 ft. A stage-versus-discharge relationship was developed for the proposed labyrinth weir using empirical design curves for half round crest shapes established through research at the Utah State University. The proposed labyrinth weir had a maximum discharge of approximately 610 cfs at the proposed level crest of the dam at EL 1108.0. Reservoir performance during the SDF with the proposed labyrinth spillway is summarized in Table 3.

Table 3: Summary of Hydrologic and Hydraulic Modeling Results for the Spillway Improvements (from the EA)

Storm Event	Peak Elevation	Available Freeboard (ft)	Peak Inflow (cfs)	Peak Outflow (cfs)
6-hr SDF	1107.2	0.8	3,191	397
24-hr SDF	1107.6	0.4	3,277	503
72-hr SDF	1107.7	0.3	3,285	528

Spillway Channel and Stilling Basin Improvements

As noted above, a HEC-RAS hydraulic analysis of the existing spillway channel indicated that the side slopes would be overtopped during the SDF. As the proposed labyrinth spillway would increase outflows during the SDF, overtopping of the spillway channel would be exacerbated. Subsequently, the design presented in the EA included a new concrete spillway chute and stilling basin. The proposed spillway chute generally follows the alignment of the existing channel and has a rectangular cross section with a bottom width of 30 ft at the spillway crest, narrowing to a bottom width of 10 ft through the majority of the channel. The length of the channel is approximately 360 ft with longitudinal slopes varying from 0.08 ft/ft to 0.40 ft/ft. The channel then transitions to a 30-ft long stilling basin with a horizontal apron and end sill. Side wall height for the channel/stilling basin varies from approximately 3.5 ft to 8.2 ft. A HEC-RAS hydraulic model of the proposed spillway channel and stilling basin concluded that the proposed design can convey the peak flow from the 72-hr 50 percent PMF storm event (528 cfs) with a minimum of one foot of freeboard to the top of walls. Additionally, the modeling concluded that the proposed stilling basin would provide adequate hydraulic jump formation for all flows up to the SDF.

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EA Spillway Improvements Cost

An EOPCC was developed as part of the EA, consistent with a Class 5 estimate as defined by the Association for the Advancement of Cost Engineering (AACD). Class 5 cost estimates are typically accurate on the low side between -20 percent and -50 percent, and on the high side between +30 percent and +100 percent. The point cost estimate for the new spillway, spillway chute, and energy dissipation structure was reported as $1,850,000. This represents approximately 43 percent of the reported total construction cost estimate of $4,300,000.

SCHNABEL'S 2014 HYDROLOGIC AND HYDRAULIC ANALYSIS

The hydrologic and hydraulic modeling performed for the 2012 EA is consistent with the current standard of care for engineering analyses of watershed hydrology and spillway capacity assessment. The proposed labyrinth weir and spillway chute/stilling basin are a viable alternative to remediate the identified spillway capacity deficiency. However, the improvements proposed in the EA do represent a significant portion of the estimated construction cost. Therefore, the Phase 1 scope of work included an evaluation of additional alternatives with the goal of identifying more cost-effective solutions to remediate the spillway capacity deficiency. The two additional alternatives included:

- Raising the Main Dam and West Dike 1 ft to EL 1109 to provide additional flood surcharge storing, thereby reducing peak discharges and the required spillway upgrade at the Main Dam.
- Incorporating overtopping protection on portions of the West Dike to provide an auxiliary spillway to pass flows in excess of the 100-yr flood, thereby reducing the required spillway upgrade at the Main Dam.

As part of the evaluation, the HEC-HMS hydrologic model from the 2012 EA was refined. The watershed was re-delineated on finer resolution topographic data. The previous watershed had been delineated on USGS 7.5-minute topographic mapping with a contour interval of 20 ft. The NYSGIS Clearinghouse recently released a 2-meter Digital Elevation Model (DEM) covering portions of Ulster County including the Cooper Lake watershed area. This DEM was developed from various LiDAR datasets including the 2009 collection of LiDAR data in the New York City – West of Hudson Watershed. The 2m DEM was used to develop 1-ft contour interval topographic mapping for the area surrounding Cooper Lake. Based on this more accurate topographic mapping, the watershed draining to Cooper Lake was delineated and found to be approximately 0.60 square miles. This is slightly smaller than the 0.63 square mile watershed used in the 2012 EA. The SCS Curve Number and Time of Concentration were also revised based on the new mapping and found to be 81.5 and 0.89 hours, respectively; these compare with values of 78 and 0.35 hours used in the 2012 EA modeling. The stage-versus-storage relationship was also revised based on the more accurate topographic modeling. Additionally, the stage-versus-discharge relationship for the existing stop-log weir was revised based on a HEC-RAS hydraulic analysis. The revised model was executed for the 50 percent PMF storm event with durations of 6 hours, 24 hours, and 72 hours. The model results for existing conditions compared with the 2012 EA results are summarized in Table 4.

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Table 4: Comparison of Hydrologic and Hydraulic Modeling Results for the Existing Conditions

	2012 EA (Schnabel HMS Runs)				Schnabel 2014			
Storm	**Runoff Volume (AF)**	**Peak Inflow (cfs)**	**Peak Outflow (cfs)**	**Peak Elevation**	**Runoff Volume (AF)**	**Peak Inflow (cfs)**	**Peak Outflow (cfs)**	**Peak Elevation**
6-Hour	743.0	3,195	301	1,107.5	723.7	2,473	327	1,107.4
24-Hour	951.5	3,277	459	1,107.7	930.6	2,515	475	1,107.7
72-Hour	1,112.3	3,289	553	1,107.8	1,074.0	2,522	538	1,107.8

As can be seen in Table 4, the revised modeling for existing conditions results in reduced peak inflows to Cooper Lake; however, the peak outflow and reservoir elevations are very similar to the 2012 EA results. This is a function of the small watershed size relative to the size of the reservoir, and indicates that peak reservoir elevations are generally a function of runoff volume rather than peak inflows. The revised modeling is consistent with the 2012 EA, and confirms the conclusion that the existing spillway has inadequate capacity to convey the regulatory SDF without overtopping of the Main Dam and West Dike embankments.

Additional Spillway Capacity Alternatives

Raise Crest of Main Dam and West Dike to EL 1109.0

This alternative is intended to increase the surcharge storage capacity in Cooper Lake by raising the Main Dam and West Dike embankments to EL 1109.0. This is one foot above the original design crest elevation of EL 1108.0. Based on the available survey data, the embankments would need to be raised a maximum of approximately 1.7 ft at the West Dike and 1.6 ft at the Main Dam. As significant fill is proposed at the Main Dam to flatten the downstream embankment slope to achieve adequate factors of safety for slope stability (see later discussion), an earthfill raise for the Main Dam was assumed. The West Dike could be raised via earthfill or through the installation of a short parapet. For the purpose of estimating the potential construction cost for this alternative, a concrete parapet was conservatively assumed for the West Dike.

Raising the dam crest by one foot while maintaining the existing normal pool elevation of 1105.6 provides approximately 161 acre-feet of additional flood storage, which is equivalent to approximately 5 inches of rainfall over the entire 384-acre (0.6 square miles) watershed. The raise of the peak pool elevation during the SDF also provides up to one foot of additional head to discharge runoff over the spillway, thereby increasing spillway capacity. This alternative was evaluated with the revised HEC-HMS model for the regulatory SDF and with a reduced 4 ft long spillway section. Table 5 summarizes the results for the proposed dam raising alternative.

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Table 5: Comparison of Hydrologic and Hydraulic Modeling Results for Raised Dam Crest

	2012 EA Proposed Labyrinth (EL 1108 dam crest) (Schnabel Runs)				Schnabel 2014 – Raised Dam Crest (EL 1109 dam crest)			
Storm	**Peak Inflow (cfs)**	**Peak Outflow (cfs)**	**Peak Elevation (ft)**	**Freeboard (ft)**	**Peak Inflow (cfs)**	**Peak Outflow (cfs)**	**Peak Elevation (ft)**	**Freeboard (ft)**
6-Hour	3,195	422	1,107.3	0.7	2,473	42	1,107.8	1.2
24-Hour	3,277	503	1,107.6	0.4	2,515	57	1,108.3	0.7
72-Hour	3,289	529	1,107.7	0.3	2,522	62	1,108.5	0.5

For the 24-hour 50% PMF storm event, our model results indicate a peak inflow of approximately 2,515 cfs, a peak outflow of approximately 57 cfs, and a peak reservoir elevation of 1108.3. This provides 0.7 ft of freeboard, which is greater than the 0.4 ft of freeboard provided by the proposed EA design for the same storm event. This alternative represents a substantial reduction in spillway crest length required and peak outflows from the reservoir during the SDF when compared with the labyrinth spillway option presented in the EA. While Cooper Lake Dam is not intended to serve as a flood control facility, this alternative would also reduce peak flows downstream of the dam during large storm events.

A hydraulic analysis performed with XP-SWMM indicates that a 36-inch diameter pipe would have adequate capacity to convey the 24-hour 50% PMF peak discharge of approximately 57 cfs. For the purposes of estimating the potential construction cost of this conceptual spillway design alternative, we assumed a concrete-encased 36-inch diameter reinforced concrete pipe (RCP), a drop inlet spillway with a weir length of 4 ft, and a USBR Type VI impact basin to provide energy dissipation prior to discharging to the downstream drainageway.

An EOPCC was developed for this alternative, consistent with a Class 4 estimate as defined by the AACD. Class 4 cost estimates are typically accurate on the low side between -15 percent and -30 percent, and on the high side between +20 percent and +50 percent. The point cost estimate for the incremental Main Dam raise, the West Dike parapet wall, and the new spillway system and energy dissipation structure is $1,270,000. This represents an estimated savings of approximately $580,000 relative to costs estimated for the spillway upgrading alternative recommended in the EA. Additional discussion on the potential cost savings from this alternative is included later in this Technical Memorandum.

Provide Auxiliary West Dike Spillway

The concept behind this alternative was to provide a new primary spillway at the Main Dam and an armored auxiliary spillway over the West Dike, with a crest elevation set at the computed peak reservoir elevation during the 100-year storm. All flows up to the 100-year storm would be conveyed through the primary spillway at the Main Dam; the new auxiliary spillway at the West Dike would begin to flow during events larger than the 100-year storm. The West Dike is a much smaller embankment than the Main Dam, and the armored auxiliary spillway would not require a long spillway chute like the proposed EA design. It was assumed that the existing embankments would be graded level to an elevation of 1108.0, consistent with the recommendation from the EA.

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The refined HEC-HMS model was used in an iterative process to analyze various configurations for the primary spillway and auxiliary spillway for the 100-year and 0.5 PMF 24-hour storm events. The spillways were sized to safely pass the 0.5 PMF storm event while providing a similar level of freeboard as the EA proposed design which was approximately 0.4 ft for the 24-hour storm event. Table 6 summarizes the results for the proposed auxiliary West Dike Spillway alternative.

Table 6: Comparison of Hydrologic and Hydraulic Modeling Results for Auxiliary West Dike Spillway

	2012 EA Proposed Labyrinth (Schnabel Runs)				Schnabel 2014 – West Dike Spillway					
Storm	Peak Inflow (cfs)	Peak Outflow (cfs)	Peak WSEL (ft)	Free-board (ft)	Peak Inflow (cfs)	Peak Outflow (cfs)			Peak WSEL (ft)	Free-board (ft)
						Primary Spillway (cfs)	Auxiliary Spillway (cfs)	Total (cfs)		
6-Hr	3,195	422	1,107.3	0.7	2,473	250	194	444	1,107.3	0.7
24-Hr	3,277	503	1,107.6	0.4	2,515	282	298	580	1,107.6	0.4
72-Hr	3,289	529	1,107.7	0.3	2,522	296	326	622	1,107.7	0.3

The proposed design includes a one-cycle labyrinth weir for the primary spillway. The labyrinth spillway would have an overall width of approximately 12 ft, an approach height of 3.6 ft, and a magnification ratio (L/W) of 3.5. Based on the HEC-HMS hydrologic model, this primary spillway would result in a peak reservoir elevation of 1106.5 and a peak discharge of approximately 73 cfs during the 100-year storm event. The HEC-HMS model was then modified with the proposed auxiliary spillway at West Dike with a crest elevation of 1106.5. The auxiliary spillway was modeled as a broad-crested weir with a weir coefficient of 2.65. Based on the model results, an auxiliary spillway length of 100 ft at the West Dike would result in a peak reservoir elevation of 1107.6 during the 24-hour 0.5 PMF storm event, providing 0.4 ft of freeboard to the dam crest. The combined peak discharge is approximately 580 cfs with approximately 282 cfs being discharged over the primary spillway, and 298 cfs being discharged over the proposed auxiliary spillway at West Dike. A logical location for the auxiliary spillway would be along the southern leg of the west dike, near where it turns 90 degrees towards the north. At this location, the dike is only about 7 ft in height, and a roadside ditch between the dike and Cooper's Lake Road could convey the majority of overflows to the west and north towards the Beaver Kill. The West Dike overflow would be armored with articulated concrete block (ACB). The primary spillway would still require a new concrete chute and energy dissipation structure. Based on a conceptual level HEC-RAS hydraulic model, a 5-ft wide rectangular concrete channel with side wall heights of approximately 5 ft would be adequate to convey the 282 cfs discharge during the SDF.

This alternative does appear to be a viable and cost-effective option when compared with the proposed spillway system presented in the EA. However, it was not further evaluated, and is not the recommended alternative for the following reasons:

- Provides less freeboard than the alternative of raising the Main Dam and West Dike to EL 1109
- Has the potential to flood Cooper Lake Road during significant West Dike overflows
- Requires approval of an overtopping protection alternative by NYS DEC
- Has the potential to exacerbate extreme flooding along the Beaver Kill above existing conditions
- Provides less potential cost savings than raising the Main Dam and West Dike due to the more significant primary spillway combined with overtopping protection at West Dike

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GEOTECHNICAL EVALUATION

EA Geotechnical Analyses

The EA presented stability analyses for both the Cooper Lake Main Dam embankment and the West Dike. Results for these analyses were compared to the required factors of safety (FOS) for various loading cases as defined by the NYS DEC. The analyses utilized the Morgenstern-Price slope stability to determine the factors of safety for circular-type failure surfaces for each of the loading cases analyzed. The slope stability program SLIDE™ (Rocscience, Inc., 1998-2009) was used to model the stability of the embankments. Engineering material properties for the embankment fill, foundation soils, and concrete elements were estimated based on the findings from previous subsurface investigations, laboratory test results, empirical correlations with published literature, and engineering judgment.

Note that according to the EA, stability questions were raised when operations and maintenance staff were attempting (in October of 2007) to access a valve on the water main that is located in one of the manholes at the downstream toe. According to the EA, "When the isolation valve access manholes were opened, it was discovered that the upper portions of the manholes had shifted, indicating movement within the embankment dam." Based on further informal site assessments in October 2007 and April 2008, it was concluded in the EA that the "main embankment dam was stable … however, the main embankment dam did not meet the level of stability required under NYS DEC." The EA further states that "shifting of the slope near the manholes and the steepness of the downstream embankment slope is a concern to the extent that the shallow sloughing is damaging and obstructing access to the isolation valves. This embankment movement is associated with the progressive creep of stone materials placed on the downstream embankment slope."

Four static load cases were evaluated: Case II – Sudden drawdown from maximum pool (from EL 1105.6 to 1065), Case IV – Partial Pool with Steady Seepage (EL 1086), Case V – Steady Seepage with Maximum Storage Pool (EL 1105.6), and Case VI – Steady Seepage with Surcharge Pool (EL 1108). Four different earthquake loading conditions (two each for static load Cases IV and V) were also evaluated. The earthquake loadings were analyzed using a pseudostatic method of analysis.

EA Main Dam Analysis

The soil properties and shear strength parameters used in the EA stability analysis for the Main Dam are presented in Table 7.

Table 7: Material Properties Utilized in EA – Main Dam

Material Description	Unit Weight (pcf)	STRENGTH PARAMETERS	
		Angle of Internal Friction (degrees)	Cohesion (psf)
Cooper Lake Dam Embankment Material	125	34	0
Cooper Lake Dam Embankment Material (Earthquake Load Cases Only)	125	32	200
Glacial Till (Foundation Soils)	145	45	0
Concrete	125	34	0

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Based on the analytical methods used and the material properties adopted, the EA concluded that the Cooper Lake Dam is stable (FOS > 1) for all load cases but failed to meet the required factors of safety for three of the static load cases: Case IV - Partial Pool with Steady Seepage, Case V - Steady Seepage with Maximum Storage Pool, and Case VI - Steady Seepage with Surcharge Pool.

EA Recommended Main Dam Modifications

Based on the findings of the Main Dam stability analyses, the EA considered modifications to increase the FOS associated with the embankment slope, and recommended regrading of the downstream embankment slope from the current 1.9H to 1V to 2.5H to 1V, and the installation of a filter drain blanket near the toe of the embankment. Combined, these modifications reportedly resulted in meeting the minimum FOS requirements for all load cases as described in the EA.

EA West Dike Analysis

The soil properties and shear strength parameters used in the EA stability analysis for the West Dike are presented in Table 8.

Table 8: Material Properties Utilized in EA – West Dike

Material Description	Unit Weight (pcf)	STRENGTH PARAMETERS	
		Angle of Internal Friction (degrees)	Cohesion (psf)
West Dike Embankment Material	125	34	0
West Dike Embankment Material (Earthquake Load Cases Only)	125	32	200
West Dike Foundation Material	120	32	0

Based on the analytical methods used and the material properties assumed, the EA concluded that the West Dike met the minimum FOS requirements for all load cases evaluated.

EA Embankment Modifications Cost

Similar to the EA recommended spillway improvements, an AACD Class 5 EOPCC was developed as part of the EA. The point-cost estimate for the embankment regrading and installation of filter drain was reported as $1,200,000. This represents approximately 28 percent of the reported total construction cost estimate of $4,300,000.

SCHNABEL'S 2014 GEOTECHNICAL ANALYSES

Schnabel performed an independent geotechnical evaluation of the Main Dam and West Dike as part of our review of the EA, and to support development of a recommended approach toward achieving compliance with NYS DEC dam safety regulations. Data from available topographic surveys and several past geotechnical investigations listed below were utilized to support our review and independent stability analysis of the Main Dam and West Dike:

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- *Engineering Assessment Report for Cooper Lake Dam and West Dike* (EA) prepared by Malcolm Pirnie/ARCADIS, dated August 2012 (Revised 2013)
- *Stability and Seepage Report for Cooper Lake Dam* prepared by Malcolm Pirnie, Inc., dated October 2009
- Geotechnical Investigation Summary Letter prepared by Mueser Rutledge Consulting Engineers (MRCE), dated May 4, 1988
- *Report on Proposed Improvements to Cooper Lake Reservoir* prepared by Charles A. Manganaro Consulting Engineers (CAMCE), dated April 1985
- Geotechnical Investigations Summary Letter prepared by MRJD, dated August 6, 1984
- *Phase I Inspection Report – Cooper Lake Dam and West Dike* prepared by O'Brien and Gere Engineers, Inc., dated July 27, 1978

Analysis Methods and Load Conditions

Similar to the EA, we performed slope stability analyses for the maximum embankment sections of the Main Dam and West Dike using the SLOPE/W computer program, a product of GeoStudio 2012, which employs various two-dimensional limiting equilibrium methods of analysis. The analyses considered a circular slip surface using the Morgenstern-Price method of analysis that satisfies both force and moment equilibrium.

Subsurface strata and material parameters selected for these analyses were based on previous site investigations and laboratory test results by others, empirical correlations, and our engineering judgment. Additional discussion related to the material parameters selected for each structure is included in the sections below. Existing embankment geometry was developed from the available survey data, and drawings and reports documenting the various modifications completed since the dam's original construction in the late 1800s. Reasonable piezometric levels throughout the embankment were estimated based on observed piezometer readings obtained during normal pool conditions.

For the purpose of this Phase 1 study and evaluating a suitably precise rehabilitation concept appropriate for developing the EOPCC herein, limited stability analyses (consisting of one static and one earthquake loading condition) were performed for the downstream slope consisting of:

- <u>Normal Pool (Case V- from EA):</u> Models the stability of the dam with the steady state phreatic surface observed within the embankment for the normal reservoir storage pool level (water surface elevation 1105.6 ft).

- <u>Earthquake (Pseudostatic Analysis):</u> Models the stability of the dam with the long-term phreatic surface within the embankment for the normal reservoir storage pool (EL 1105.6) when subjected to seismic loading. The peak ground acceleration was obtained from the USGS Earthquake Hazards Program. The return period used for the analysis was 2,475 years (2% in 50 years). A peak ground acceleration of 0.072g (derived from USGS data available in 2008) was used for the seismic analysis.

Main Dam Review and Stability

Material parameters (and ranges) for our independent analysis of the existing Main Dam were estimated based on our review of the numerous previous geotechnical investigations and analyses, empirical

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correlations, laboratory testing, and our experienced engineering judgment. These values are summarized in Table 9.

Table 9: Material Properties Utilized by Schnabel for Slope Stability Analysis – Main Dam

Material Description	Drained (Effective) Angle of Internal Friction (degrees)	Drained (Effective) Cohesion (psf)	Moist Unit Weight (pcf)
Embankment Fill Material (USCS Group Symbol SM, SC, ML, CL)	26 to 34	0	125
Foundation Material- Glacial Till (USGS Group Symbol GM, SM, ML)	32	0	135
Concrete	45	10,000	150

1. USCS = Unified Soil Classification System
2. SM = Silty Sand; SC = Clayey Sand; ML = Silt; CL = Lean Clay; GM = Silty Gravel

Our opinion of a representative drained (effective) angle of internal friction for the embankment fill material differs from that presented in the EA (the EA presented a value of 34 degrees). While selection of the higher value presented in the EA may be justified in some locations, the variability of the embankment soil type and densities encountered in the borings, and the very limited shear strength laboratory data presented in the EA, do not justify its use as a representative value of the entire embankment.

Apart from the EA, two previous geotechnical explorations were performed to characterize the Main Dam and West Dike as part of the previously proposed raise of the dam and dike. Test borings were advanced and piezometers installed in June 1984 by Mueser, Rutledge, Johnston & DeSimone (MRJD) Consulting Engineers as summarized in the April 1985 CAMCE report. Limited laboratory testing (natural moisture content and grain size distribution) was performed during this phase, with no laboratory shear strength testing. However, the MRJD report (included as an appendix of the CAMCE report) cautioned that “fill materials in the Main Dam are predominantly loose and soft” based on the test boring data. Recommendations for additional geotechnical work were provided by MRJD.

Based on the recommendations by MRJD in the 1985 CAMCE report, a more extensive geotechnical investigation was performed by Mueser Rutledge Consulting Engineers (MRCE) between August and October 1987. This work is summarized in the May 4, 1988 MRCE report. Additional laboratory testing was performed to characterize the Main Dam and West Dike fill and foundation soils, including several unconsolidated undrained (UU) triaxial compression tests (to characterize undrained shear strengths), and one consolidated undrained (CU) triaxial compression test, presumably performed to characterize the undrained and drained shear strength of the Main Dam embankment fill. CU tests were performed on two specimens, each tested at two different confining stresses, which were obtained from a single sample of the Main Dam embankment (Sample 6S from a depth interval from 11 to 13 ft in Test Boring B-110U).

There are several inconsistencies in the MRCE (1988) report that make it difficult to interpret the characterization of shear strength of the CU test data. First, on page 8 of the report, the narrative describes the compressive strengths of the Main Dam embankment fill as 5 to 7 tons per square foot (tsf). These values correspond to the deviatoric stresses (σ_1-σ_3) at failure, and the curves from tests performed on Specimens 1 and 2 are reversed (mislabeled) on the stress-strain curve shown on the upper portion of

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Plate 1. It is also not clear that these samples were backpressure saturated prior to testing based on the data presented. If they were not, artificially higher shear strengths would be manifest from the test data. Also, the Mohr Diagram on the lower part of Plate 1 does not indicate if the normal stresses correspond to total stress or effective stress, and nowhere is the friction angle of 38 degrees that was obtained referred to as a "drained" friction angle. Based on the data provided, this would be necessary to have definitive confidence that the friction angle reported is indeed a "drained" or "effective" friction angle (while one would presume this should be the case). Finally, Sample 6S was an undisturbed sample obtained between Standard Penetration Test (SPT) Samples 5D (above) and 7D (below), with corresponding SPT N-values of 8 and 16, respectively. This implies that Sample 6S that was tested is not necessarily representative of the "predominantly loose and soft soils" that were of initial concern.

Upon review of the soil laboratory testing performed for the EA, a CU triaxial compression test was performed, but it was performed on only one *specimen* (obtained from an undisturbed *Sample* U-1 in Test Boring GB-2) at only one confining stress (11.8 psi confining stress). The drained angle of internal friction was obtained from this single test by forcing the failure line tangent to the Mohr's circle and extending it through the origin. This represents an incomplete triaxial test, lacking a sufficient number of specimens to account for the uncertainty associated with the variability in the material testing of this sample, and an insufficient number of samples tested to account for the uncertainty associated with the variability in the soil conditions within the embankment dam, which based on soil classifications and SPT N-values is quite large.

In addition, it is quite possible that the deformation of the downstream manholes could be symptomatic of a creep along a potential failure surface within the embankment itself. Limiting equilibrium methods of analysis do not consider the deformations required to mobilize the shear strengths adopted in the model. For coarse-grained soils (SM & SC), low levels of deformation/shear strain are typically required to mobilize peak/design-level shear strengths, but quite large deformations (shear strains in excess of 5 to 10 percent or much more) may be required to mobilize peak/design drained strengths in fine-grained soils (ML & CL). It is possible the displacements at the toe of the dam are a manifestation of deformations that have occurred over a relatively long period of time as the softer fine-grained soils have deformed to mobilize strengths required for limiting equilibrium. Such deformations are necessary to mobilize strength and do not imply that a true slope failure (i.e., a large mass-wasting of the materials above the critical failure surface) will occur. Note that based on the model developed for the Main Dam, the slope is just stable (i.e., FOS ~ 1) for a drained angle of internal friction of about 26 degrees for the embankment fill material.

In consideration of this variability and uncertainty, we evaluated the sensitivity of the FOS with the slope stability model by varying the drained friction angle of the embankment fill within a reasonable range of values. Consistent with the overall findings from the EA and previous reports by MRJD and CAMCE, the results of the sensitivity analysis confirmed that the Main Dam (existing configuration) is stable but does not meet NYS DEC stability criteria for the normal pool condition for a drained friction angle less than 26.

The EA recommended buttressing (or flattening) the downstream slope of the Main Dam from 1.9H:1V to 2.5H:1V to address inadequate factors of safety (FOS) for slope stability. We agree that flattening the downstream slope is an appropriate method to achieve the requisite minimum FOS for slope stability at the Main Dam.

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Similar to the analysis for the existing configuration of the embankment described above, we evaluated the sensitivity of the slope stability model for a flattened downstream slope and a raised top of dam (proposed in conjunction with the replacement spillway to achieve adequate spillway capacity). The primary intent of this analysis was to estimate the required slope to achieve adequate factors of safety considering a range of drained friction angles for the embankment fill. Assuming that the embankment fill has a representative drained friction angle of 26 degrees, the results of the sensitivity analysis indicate that the downstream slope will need to be flattened to approximately 3.0H:1V to meet NYS DEC stability criteria for the normal pool and earthquake loading conditions. We are not advocating that such a low drained friction angle is necessarily required to drive the development of a final design; however, we feel that there is currently insufficient data to justify a less conservative evaluation. The recommended baseline design concept presented herein considers flattening the downstream slope to 3.0H:1V, which will nominally increase construction costs, but provide for a more reliable safety upgrade and improve ease of maintenance. In addition, this is consistent with NYS DEC's recommendation for minimum embankment slopes.

In addition to improving slope stability, the EA recommended installation of a new filtered drainage blanket and toe drain system. We agree that this is a necessary requirement considering no engineered internal drainage system is known to exist.

West Dike Review and Stability

Material parameters (and ranges) for our independent analysis of the West Dike were estimated based on our review of the numerous previous geotechnical investigations and analyses, empirical correlations, laboratory testing, and our experienced engineering judgment. These values are summarized in Table 10.

Table 10: Material Properties Utilized by Schnabel for Slope Stability Analysis – West Dike

Material Description	Drained (Effective) Angle of Internal Friction (degrees)	Drained (Effective) Cohesion (psf)	Moist Unit Weight (pcf)
Embankment Fill Material (USCS Group Symbol SM, SC)	25 to 34	0	125
Foundation Material – Glacial Till (USCS Group Symbol GM, SM, ML)	32	0	135

1. USCS = Unified Soil Classification System
2. SM = Silty Sand; SC = Clayey Sand; GM = Silty Gravel; ML = Silt

Similar to the Main Dam and in consideration of the lack of adequate laboratory shear strength data and variability in soil type and density in the fill used to construct the approximately 1500-ft long dike, our opinion of a representative drained (effective) angle of internal friction for the embankment fill material differs from that presented in the EA (the EA presented a value of 34 degrees). In addition, it is our opinion that the piezometric levels used in the EA stability model represent conditions in the foundation soils rather than the embankment. The existing piezometer located at the crest of the dam (GB-3 WD) was installed with a tip elevation 30 ft below grade (in possible foundation soils), with a screened interval between 22.7 ft and 40 ft below grade, and indicates relatively low piezometric levels at various pool levels. We agree with the findings from the EA that the low water levels measured in Piezometer GB-3 WD may be a result of higher permeability foundation soils. MRJD presented a similar conclusion for the

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pre-existing crest piezometer in their 1987 Geotechnical Investigations Summary Letter. The two piezometers installed on the crest of the West Dike by MRJD in 1984 were set with tip elevations at 20 ft and 26.5 ft below ground surface (also in possible foundation soils) with a screened interval between 34 ft and 13 ft below grade. However, the low piezometer readings may not accurately represent pore water pressures for the upper zone of embankment soils, which tend to drive slope stability. Therefore, our slope stability analysis considered a higher piezometric level based on the water levels encountered during the 1984 MRJD drilling program, and more typical for an embankment constructed without an internal drainage system. The water levels measured in the midslope piezometer appeared reasonable considering the depth of standing water at the toe of the dike.

Again, we evaluated the sensitivity of the FOS with the slope stability model for the West Dike by varying the friction angle of the embankment fill within a potential range of values, but with what we believe is a more representative piezometric level. With these higher piezometric levels, the results of the sensitivity analysis indicate that the West Dike (existing configuration) is likely currently stable but does not meet NYS DEC stability criteria for the normal pool condition within the range of friction angles evaluated.

Our findings for the West Dike represent a significant difference from those presented in the EA. The EA did not consider any upgrades to the West Dike aside from leveling the crest to EL 1108. As part of the next phase of this project, we will develop conceptual design alternatives to upgrade slope stability at the West Dike, which may consist of slope flattening and/or an internal drainage system. Similar alternatives were considered by MRCE in their evaluation of dam raising concepts. Furthermore, we are recommending that a targeted field exploration program be performed to collect additional data at the Main Dam and West Dike. This field program is intended to establish a defendable record of laboratory shear strength data for both the Main Dam and the West Dike, and to confirm representative embankment soil piezometric levels for the West Dike. These additional activities coupled with the previous investigations will allow us to determine an appropriate and representative basis of design for the internal friction for the Main Dam and the West Dike.

ENGINEER'S OPINION OF PROBABLE CONSTRUCTION COSTS

Overview of Recommended Baseline Design Concept

The recommended concept includes raising the top of dam 1 ft to EL 1109 and constructing a replacement spillway that is significantly smaller than the spillway and chute proposed in the EA. Considering the existing crest width of the Main Dam and that the downstream slope will need to be flattened to achieve adequate factors of safety for slope stability, the top of dam could be raised with earthfill. We recommend raising the West Dike with a concrete parapet, although the design concept could be refined during final design to consider an earthfill raise or small concrete curb.

The replacement spillway will consist of a 4-ft by 4-ft box-drop inlet spillway located in about the same location of the existing spillway on the right abutment. Spillway flow will be discharged through a 36-inch diameter reinforced concrete pipe (RCP) to the existing stream channel at the toe of the dam. Several pre-cast manholes will be installed at various locations to accommodate grade changes and to provide access points to the conduit. We recommend encasing the conduit in concrete, particularly within the footprint of the embankment, to improve backfill adjacent to the conduit and to prevent failure modes associated with erosion of embankment materials along and/or into the conduit. A USBR Type VI impact basin will be constructed at the conduit outlet to provide energy dissipation.

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As mentioned above, the downstream slope of the Main Dam will be flattened to improve slope stability. We recommend a 3.0H:1.0V slope for ease of maintenance and to accommodate uncertainties in the existing material properties. The existing riprap will need to be removed prior to flattening the slope with earthfill. A toe drainpipe, chimney, and blanket drain system will also be constructed at the Main Dam to improve seepage control. The drainage system will be connected to a precast concrete pit equipped with a monitoring weir.

Other miscellaneous items considered with this concept include raising the existing piezometers and removing existing manholes located on the downstream slope. Schematics of this proposed alternative are presented in Figures 1 – 5. Figure 1 presents a plan view of the proposed improvements at the Main Dam. Figure 2 presents a sectional view of the proposed earthfill at the Main Dam. Figure 3 presents a profile view of the proposed spillway and storm sewer system. Figure 4 presents a section view and details of the improvements at West Dike, and Figure 5 presents a plan view of the improvements at West Dike.

The details of two significant elements of the baseline design are still outstanding. In particular these are the Outlet Works and the West Dike stability improvements. Both of these elements have been advanced during this phase of the work but neither has been resolved. Resolution of these two items will be part of the Phase 2 Preliminary Engineering services.

Comparison of Probable Construction Costs

The EOPCC contained herein are to be considered "Class 4" estimates, according to the categories of cost estimates defined by the AACE International, which are summarized in Tables 11 and 12. These estimates are advanced from those presented in the EA which were considered "Class 5."

Table 11: AACE Accuracy Matrix for Estimating Classes

ESTIMATE CLASS	Primary Characteristic	Secondary Characteristic			
	LEVEL OF PROJECT DEFINITION Expressed as % of complete definition	**END USE** Typical purpose of estimate	**METHODOLOGY** Typical estimating method	**EXPECTED ACCURACY RANGE** Typical variation in low and high ranges	**PREPARATION EFFORT** Typical degree of effort relative to least cost index of 1
Class 5	0% to 2%	Concept Screening	Capacity Factored, Parametric Models, Judgment or Analogy	Low: -20% to -50% High: +30% to +100%	1
Class 4	1% to 15%	Study or Feasibility	Equipment Factored or Parametric Models	Low: -15% to -30% High: +20% to +50%	2 to 4
Class 3	10% to 40%	Budget Authorization , or Control	Semi-Detailed Unit Costs with Assembly Level Line Items	Low: -10% to -20% High: +10% to +30%	3 to 10
Class 2	30% to 70%	Control or Bid/ Tender	Detailed Unit Cost with Forced Detailed Take-Off	Low: -5% to -15% High: +5% to +20%	4 to 20
Class 1	50% to 100%	Check Estimate or Bid/Tender	Detailed Unit Cost with Detailed Take-Off	Low: -3% to -10% High: +3% to +15%	5 to 100

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Table 12: AACE Class 4 Estimate Description

CLASS 4 ESTIMATE	
Description: Class 4 estimates are generally prepared based on limited information and subsequently have fairly wide accuracy ranges. They are typically used for project screening, determination of feasibility, concept evaluation, and preliminary budget approval. Typically, engineering is from 1% to 15% complete, and would comprise at a minimum the following: plant capacity, block schematics, indicated layout, process flow diagrams (PFDs) for main process systems, and preliminary engineered process and utility equipment lists. **Level of Project Definition Required:** 1% to 15% of full project definition. **End Usage:** Class 4 estimates are prepared for a number of purposes, such as but not limited to, detailed strategic planning, business development, project screening at more developed stages, alternative scheme analysis, confirmation of economic and/or technical feasibility, and preliminary budget approval or approval to proceed to next stage.	**Estimating Methods Used:** Class 4 estimates virtually always use stochastic estimating methods such as equipment factors, Lang factors, Hand factors, Chilton factors, Peters-Timmerhaus factors, Guthrie factors, the Miller method, gross unit costs/ratios, and other parametric and modeling techniques. **Expected Accuracy Range:** Typical accuracy ranges for Class 4 estimates are -15% to -30% on the low side, and +20% to +50% on the high side, depending on the technological complexity of the project, appropriate reference information, and the inclusion of an appropriate contingency determination. Ranges could exceed those shown in unusual circumstances. **Effort to Prepare (for US$20M Project):** Typically, as little as 20 hours or less to perhaps more than 300 hours, depending on the project and the estimating methodology used. **ANSI Standard Reference Z94.2-1989 Name:** Budget estimate (typically -15% to + 30%). **Alternate Estimate Names, Terms, Expressions, Synonyms:** Screening, top-down, feasibility, authorization, factored, pre-design, pre-study.

We developed unit costs for the individual cost items considered in the EOPCC based on our experience with recently completed construction projects of similar magnitude and complexity and published construction cost data. Explanations for items considered are listed below:

- ***Mobilization and Demobilization:*** The cost for mobilization and demobilization includes the contractor's cost to mobilize equipment and personnel, acquire bonds and insurance, provide field offices, and other miscellaneous costs. Mobilization and demobilization were estimated to be about 10 percent of the total construction cost.

- ***Dewatering and Control of Water:*** This includes costs for dewatering the applicable work areas and control of surface water during construction. The costs presented assume that the lake will remain operational during construction.

- ***Erosion and Sediment Control:*** This includes erosion and sediment control measures (e.g., silt fence, construction entrance, erosion control mat, etc.) to meet local and state construction stormwater regulations.

- ***Stripping:*** Stripping (removal of vegetation, debris) of the site required to place fill and construct other structures.

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- ***Clearing and Grubbing:*** Removal of woody vegetation (shrubs, trees, etc.) at the site required to place fill and construct other structures.

- **Structure Removal:** Removal of existing structures (e.g., concrete spillway, manholes, etc.) and disposal of material at an off-site location.

- ***Excavation:*** Excavation includes removal of soils and rock required to flatten the downstream slope of the Main Dam and for construction of the replacement spillway, toe drain, parapet and other structures.

- ***Select Fill:*** The cost of select fill includes the materials to be used to construct the Main Dam drainage system and as riprap bedding.

- ***Earthfill:*** This includes soil fill to flatten the downstream slope of the Main Dam and for backfilling the replacement spillway, parapet and other structures. Some borrow fill will be required.

- **Plastic Pipe:** This includes slotted and solid walled plastic pipe (e.g., HDPE, PVC, etc.) used for the Main Dam drainage system.

- **Precast Concrete Structures:** This includes precast concrete structures for the seepage monitoring pit, manholes, drop box-inlet replacement spillway, and impact basin.

- ***Topsoil and Seed:*** This includes establishing permanent turf in disturbed areas of the site.

- ***Concrete:*** Concrete quantities include the spillway conduit encasement and West Dike parapet. The cost of concrete includes concrete, formwork, pumped delivery, and miscellaneous related items.

- ***Reinforcing Steel:*** This includes costs for purchasing and installing all reinforcing steel for concrete. For the purpose of estimating, we assumed 130 pounds of steel per cubic yard of concrete, which is considered typical for hydraulic structures.

- ***Misc. Unidentified Items:*** This includes items that have not yet been identified or not considered to be of significant enough cost to break out separately.

- ***City Index:*** This factor covers the costs associated with the inflated cost of labor and materials due to the location of your site relative to national averages. A City Index of 124% was utilized based on RSMeans Building Construction Cost Data 2014; City Index for Kingston, NY; Site & Infrastructure Divisions.

- ***Contingency:*** A contingency of 30 percent of the construction cost was included in the total cost to provide for margins of error in the study level cost and quantity estimates, to account for numerous smaller items not specified above, to accommodate a reasonable amount for field changes during construction, and to recognize unknowns related to bidding climate and material availability.

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The EA presented a Class 5 EOPCC based on the major project elements. For comparison, our EOPCC was broken out similarly but advanced to a Class 4 estimate. Table 13 presents a comparison of the common elements and provides an update on the anticipated construction cost.

Table 13: Summary of Opinion of Probable Construction Costs in 2012 US Dollars

Item	EA EOPCC (2012 dollars)		Updated EOPCC (2014 dollars)	
	Notes	EA Point Estimate	Notes	Updated Point Estimate
Main Dam Embankment Modifications	Includes regrading of Main Dam and West Dike crest to elevation 1108 ft, regrading downstream slope to 2.5H:1V, and filter	$1,200,000	Includes regrading of Main Dam and West Dike crest to EL 1108 ft, regrading downstream slope to 3H:1V, and filter	$930,000
West Dike Embankment Modifications	None identified	$ 0	Stabilization measures required but not yet identified	To be determined in Phase 2
Outlet works	Includes new outlet piping, valves, gates	$1,250,000	Recommended outlet works based on rehabilitation of existing or construction of new intake structure	To be determined in Phase 2
Spillway Capacity	Includes new labyrinth spillway, spillway chute, and energy dissipation structure	$1,850,000	Includes an earthen raise of the Main Dam from EL 1108 ft to EL 1109 ft, new primary spillway and RCP conveyance, installation of parapet wall along West Dike	$1,270,000
Proposed Construction Total	This is a Class 5 cost estimates which is typically accurate on the low side between -20 percent and -50 percent, and on the high side between +30 percent and +100 percent	$4,300,000	This is a Class 4 estimate which is typically accurate on the low side between -15 percent and -30 percent, and on the high side between +20 percent and +50 percent	To be determined in Phase 2

Please note that our independent expectation of construction costs for the embankment modifications is less than that presented in the EA despite our recommendation to further flatten the slope to 3H:1V. This is likely due to the advancement of project detail and incorporation of unit rates consistent with recent or ongoing projects we have been involved in within the northeast. The revised approach for achieving spillway capacity represents a saving of approximately $580,000 in comparison to the proposed concept identified in the EA. Note that this estimate includes a conservative design for a concrete parapet to raise the West Dike to EL 1109. More detailed engineering design may identify alternate configurations that could lead to additional cost savings for the recommended spillway capacity improvements. Additional project costs associated with the baseline West Dike stability improvements and the Outlet Works will be determined in the Phase 2 Preliminary Engineering Services.

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Thank you for the opportunity to perform these important engineering services. Please contact me directly with any questions or concerns regarding the information presented in this memorandum.

Sincerely,

SCHNABEL ENGINEERING OF NEW YORK

Gregory J. Daviero, PhD, PE
Principal

KJR:GJD:JRY:SAR:hcf

Attachment:
(1) Figures 1-5

Distribution:
Kingston Water Department (email only)
Attn: Ms. Judith Hansen

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ATTACHMENT 1

FIGURES

Figure 1: Main Dam Modifications – Plan
Figure 2: Main Dam Modifications – Typical Sections
Figure 3: Main Dam Modifications – Spillway Profile
Figure 4: West Dike Modifications Typical Section
Figure 5: West Dike Modifications Plan

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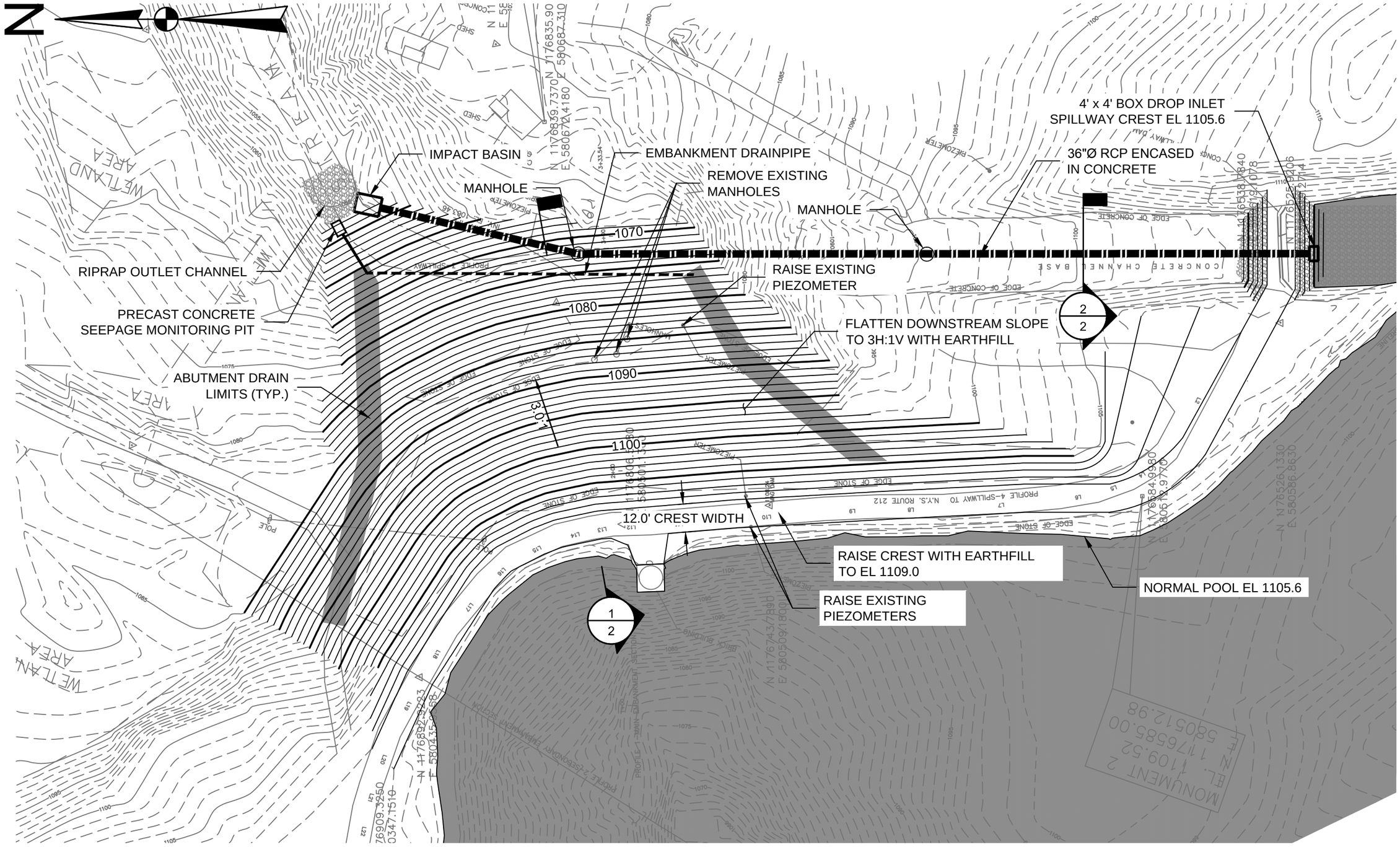

THE SOURCE OF THE SURVEY INFORMATION IS THE BRINNER AND LARIOS, P.C. SURVEY TITLED "TOPOGRAPHIC MAP OF THE CITY OF KINGSTON WATER SUPPLY COOPER LAKE RESERVOIR" AND DATED SEPTEMBER 24, 2008.

SCALE: 1"=40'

0 40' 80'

Schnabel ENGINEERING

IMPROVEMENTS TO COOPER LAKE DAM AND WEST DIKE

PRELIMINARY ENGINEERING PHASE 1

CITY OF KINGSTON

PROJECT NO. 14925003

MAIN DAM MODIFICATIONS - PLAN

FIGURE 1

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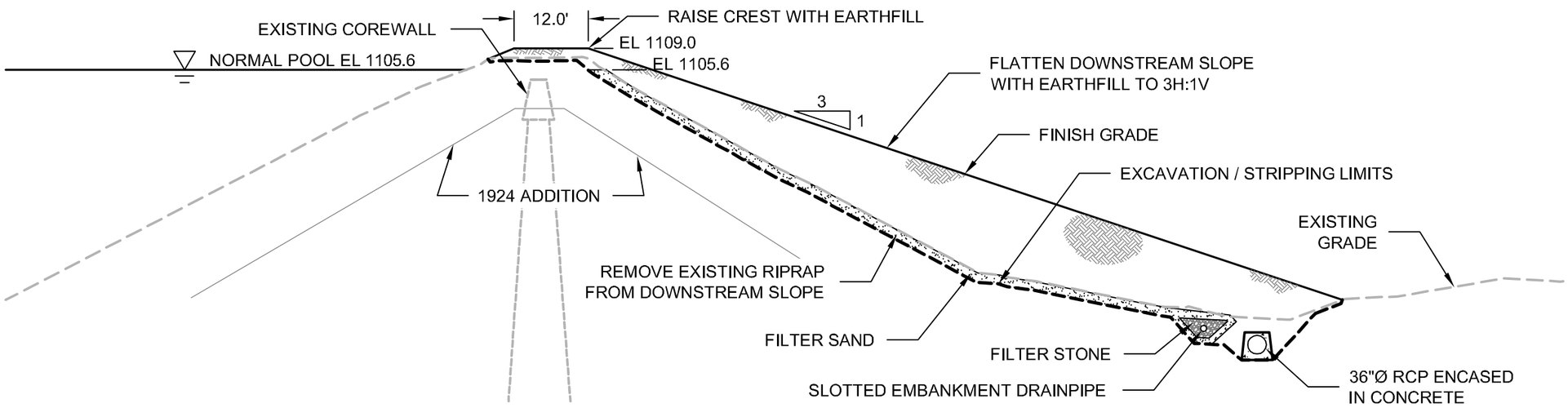

1 TYPICAL SECTION

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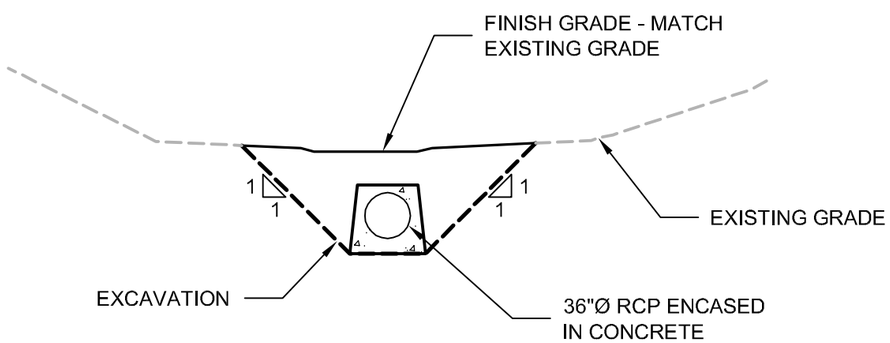

2 36"Ø RCP CONCRETE ENCASEMENT

SCALE: 1"=10'

0 10' 20'

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IMPROVEMENTS TO COOPER LAKE DAM AND WEST DIKE
PRELIMINARY ENGINEERING PHASE 1
CITY OF KINGSTON
PROJECT NO. 14925003

MAIN DAM
MODIFICATIONS - TYPICAL
SECTIONS

FIGURE 2

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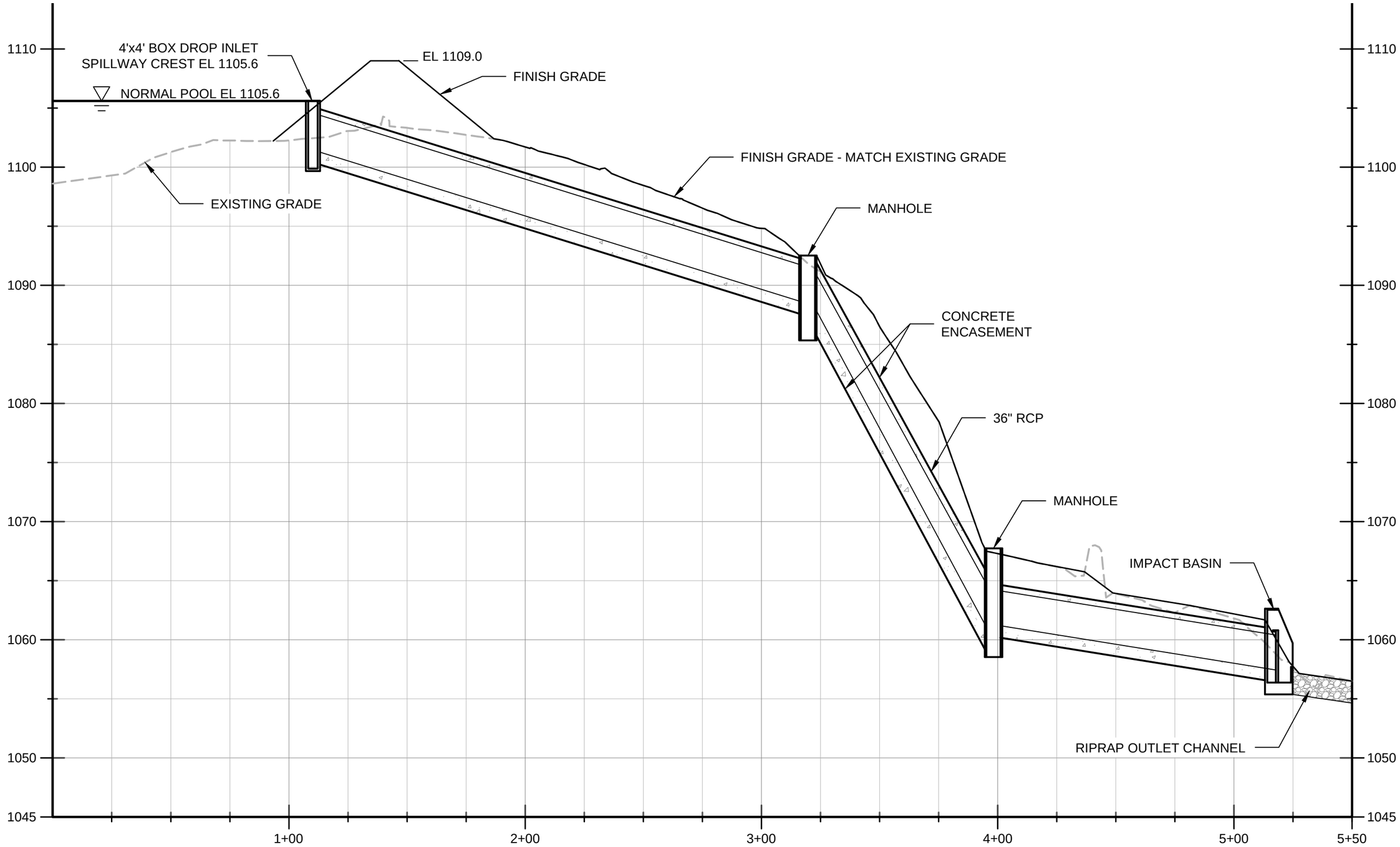

VERTICAL
SCALE: 1"=8'
0 8' 16'

HORIZONTAL
SCALE: 1"=40'
0 40' 80'

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IMPROVEMENTS TO COOPER LAKE DAM AND WEST DIKE
PRELIMINARY ENGINEERING PHASE 1
CITY OF KINGSTON
PROJECT NO. 14925003

MAIN DAM
MODIFICATIONS - SPILLWAY
PROFILE

FIGURE 3

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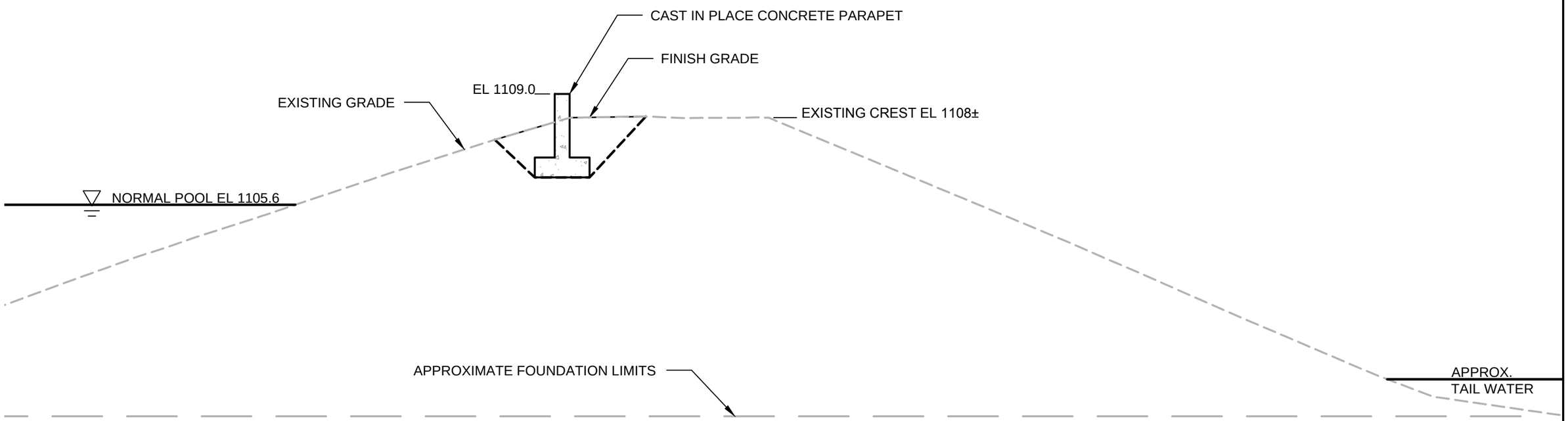

1 TYPICAL EMBANKMENT SECTION

SCALE: 1"=5'

0 5' 10'

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IMPROVEMENTS TO COOPER LAKE DAM AND WEST DIKE
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CITY OF KINGSTON
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WEST DIKE MODIFICATIONS
TYPICAL SECTION

FIGURE 4

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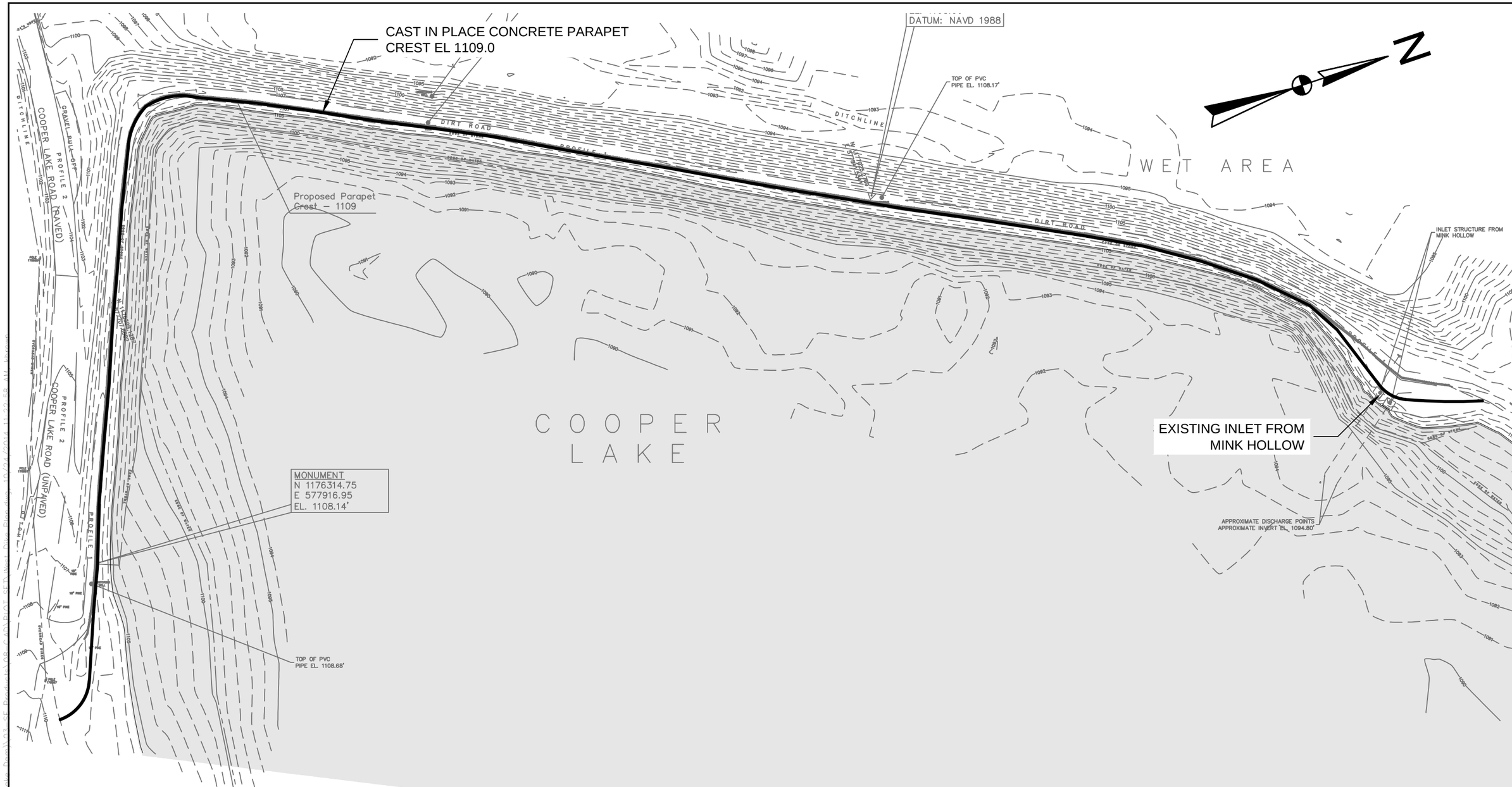

1 WEST DIKE PLAN

SCALE: 1"=80'

0 80' 160'

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IMPROVEMENTS TO COOPER LAKE DAM AND WEST DIKE
PRELIMINARY ENGINEERING PHASE 1
CITY OF KINGSTON
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WEST DIKE MODIFICATIONS PLAN

FIGURE 5

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