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# White Cloud Dam Disposition Feasibility Study

White Cloud Dam, White Cloud, Michigan

**Submitted to:**
The City of White Cloud
12 North Charles Street
White Cloud, Michigan 49349

**Submitted by:**
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Project 2302435

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# 1. Introduction

## 1.1 Background

The City of White Cloud owns and operates the White Cloud Dam (the Dam) on the White River in White Cloud, Michigan. The Dam is classified as a High Hazard dam by the Michigan Department of Environment, Great Lakes, and Energy (EGLE) Dam Safety and currently has a poor condition rating. The condition assessment categories are outlined in Table 1-1.

**Table 1-1: Condition Assessment Category**

| Condition | Description |
|---|---|
| Satisfactory | No existing or potential dam safety deficiencies are recognized. Acceptable performance is expected under all loading conditions. |
| Fair | No existing dam safety deficiencies are recognized for normal loading conditions. Rare or extreme hydrologic and/or seismic events may result in a dam safety deficiency. |
| Poor | A dam safety deficiency is recognized for loading conditions that may realistically occur. Remedial action or further investigations and studies are necessary to determine risk. |
| Unsatisfactory | A dam safety deficiency is recognized that requires immediate or emergency remedial action for problem resolution. |
| Not Rated | The dam has not been inspected, is not under State jurisdiction, or has been inspected but, for whatever reason, has not been rated. |

In recent years, numerous deficiencies have been identified during dam safety inspections by EGLE and structural inspections by the City’s engineer (Holland Engineering, formerly OMM, Inc.). These deficiencies range from uncertainty about spillway capacity, concrete and steel deterioration, seepage on the downstream side of the embankment, and woody vegetation growing on the embankment. Additionally, there have been several recent overtopping events reported along the overflow spillway and a failure of stoplogs on the principal spillway. These deficiencies have led to the Dam receiving a poor condition rating from EGLE Dam Safety as of the latest inspection report, dated December 22, 2022.

Over the life span of the Dam, the Dam has faced numerous deficiencies that were addressed through reconstruction and modification. In 1975 and 1986, the City was forced to cut through the embankment during high flow events to keep the spillway structure from failing and to protect municipal pump houses that at the time were used as the main City water source. As recent as 2014 and 2018, overtopping of the Dam occurred, resulting in overflow damage, and required necessary repairs.

In 2019, the Michigan Department of Natural Resources (MDNR) conducted a fisheries survey report of Lake White Cloud. Findings from the report conclude the Dam blocks fish passage and significantly increases the water temperature of the White River downstream of the Dam. The MDNR recommended the White Cloud Dam be removed to allow the White River to return to a free-flowing state.

In 2022, Trout Unlimited prepared a preliminary dam removal feasibility study. This study explored the design considerations associated with dam removal and river restoration and provided perspectives for restoration aesthetics and options for recreation and natural resource improvements. Unfortunately, this document did not provide financial evaluations for dam-in or dam-removal scenarios.

Given the poor condition rating and the challenges of maintenance and upkeep on the Dam, The City of White Cloud (the City) has recently engaged in discussions and meetings regarding a long-term plan for the structure. Out of these meetings, there was a desire to continue maintaining the Dam but uncertainty regarding the life-cycle costs and liability for maintaining versus removing the Dam.

## 1.2 Purpose

To better inform the City and community on anticipated costs, the City requested and was awarded funding through the EGLE Dam Risk Reduction Grant Program (DRRGP) to perform this dam disposition feasibility study. Through this study, the City will gain a greater understanding of the current condition of the Dam, as well as long-term risks, liabilities, and life-cycle costs for three potential scenarios: dam-in under current dam regulations, dam-in under proposed dam regulation changes, and dam removal.

## 1.3 Scope of Work

A summary of the scope of work performed by GEI for the White Cloud Dam Disposition Feasibility Study is presented below:

- Review available engineering reports and data documenting the configuration, construction, and performance of the existing dam and appurtenant facilities.
- Perform a desktop assessment for wetlands, threatened and endangered species, and cultural resources.
- Desktop assessment for factors that can impact the cost of implementing a dam removal project include infrastructure, like bridges and utilities, and real estate.

- Perform onsite analysis of the impoundment including a bathymetric survey, sediment quantification, sediment sampling, and characterization within the impoundment.
- Preform subsurface exploration program including geotechnical borings within the dam embankment.
- Preform structural field investigation including concrete coring and ground penetrating radar (GPR).
- Perform analysis of seepage and slope stability of the existing earthen dam embankment.
- Preform structural stability and strength analysis on existing concrete spillway.
- Perform hydrologic analysis and update current hydraulic model for existing hydraulic capacity of the dam and evaluate stoplog operation scenarios.
- Engineering Analysis / Feasibility Analysis for dam repair and dam removal.
- Develop Channel and Floodplain Conceptual Design for a dam removal alternative.
- Develop preliminary cost estimates for dam repair and dam removal alternatives.

## 1.4 Authorization

GEI performed engineering consulting services for the City of White Cloud – Project No. 2302435 (Project), with the work authorized by the City of White Cloud by means of the Professional Services Agreement dated May 30th, 2023.

## 1.5 Personnel

The following GEI personnel were primarily responsible for performing the engineering analyses for this report:

| | |
|---|---|
| Project Principal: | Dan DeVaun, P.E. |
| Water Resources Engineer: | Janeen McDermott, P.E. |
| Geostructural Engineer: | Morgan Carden, P.E. |
| Structural Engineer: | Richard Price, P.E. |

## 1.6 Elevation Datum

Elevations (El.) listed herein are referenced to the North American Vertical Datum of 1988 (NAVD88).

## 1.7 Limitation of Liability

The professional services completed in preparing this report of repair and removal alternative concepts were performed in a manner consistent with the level of care and skill ordinarily exercised by members of the engineering profession currently practicing in the same locality and under similar conditions as this project. No other representation, expressed or implied, is included or intended, and no warranty or guarantee is included or intended in this report, or any other instrument of service.

# 2. Site Description and Existing Conditions

## 2.1 Location

The White Cloud Dam is located at the southeast corner of the City of White Cloud. Two city parks are set on the edge of the impoundment near the Dam. The City of White Cloud Rotary Park is located at the toe of the right[1] downstream embankment of the Dam. There is a public boat launch located on the right upstream embankment of the Dam, and a public beach with a dock is located North of the impoundment and accessible from James Street. The impoundment is primarily used for swimming and boating. South State Street runs along the crest of the Dam (Figure 2-1).

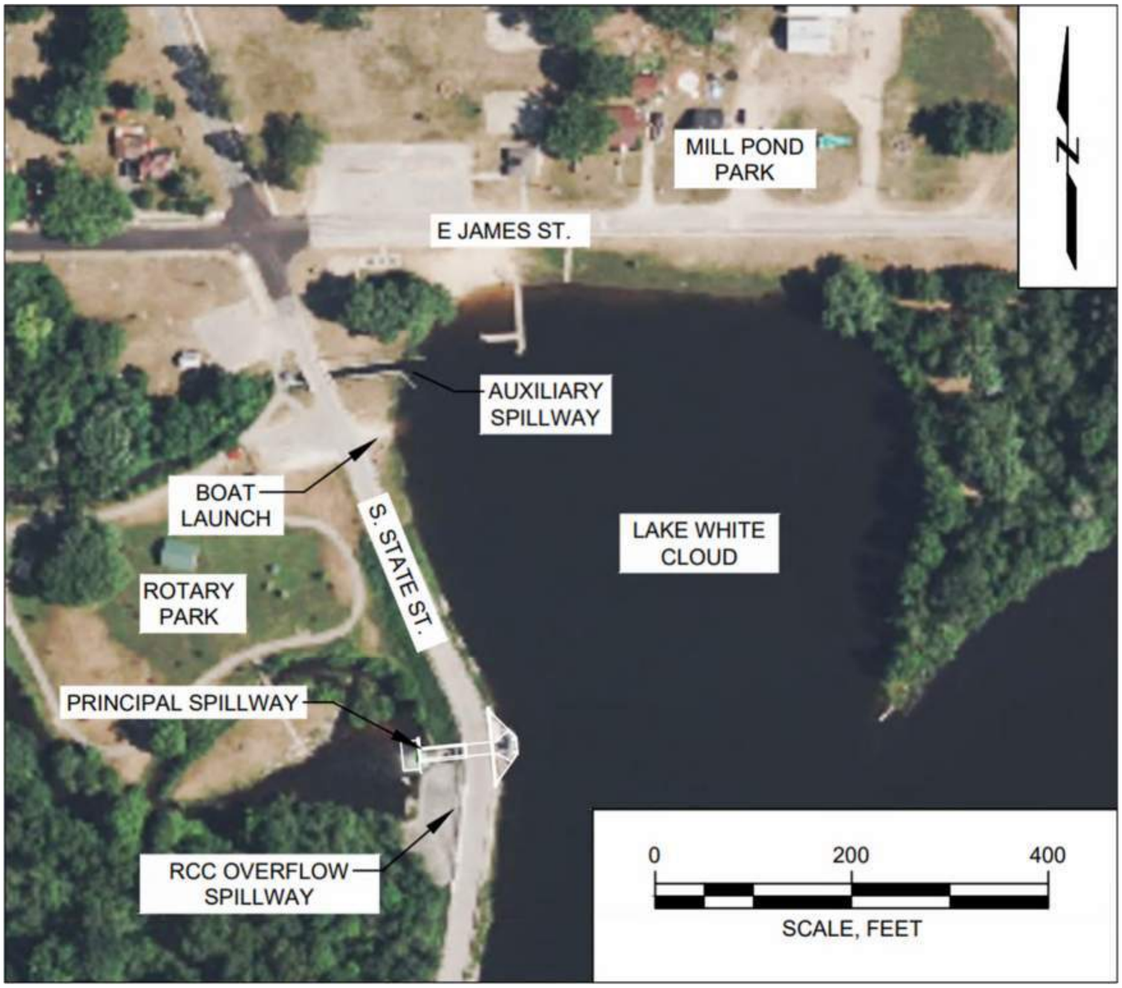

Figure 2-1: White Cloud Dam Location

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[1] 'left' and 'right' directional notations when referring to dam structures is from the viewpoint of looking downstream.

## 2.2 Dam Structure

The White Cloud Dam is a 950-foot-long and roughly 19-foot-high earthen embankment with one concrete spillway with stoplogs and three gates, an auxiliary spillway with stoplogs and a gate, and a roller compacted concrete (RCC) overflow emergency spillway.

**Table 2-1: Key Existing Project Data – White Cloud Dam**

| Parameter | White Cloud Dam |
|---|---|
| Dam Crest El. (feet) | 849 |
| Dam Crest Width (feet) | 30 |
| Normal Operating Headwater El. (feet) | 845 |
| Embankment Length (feet) | 950 |
| Right Embankment Upstream / Downstream Slopes (H:V) | 2.5:1 to 5:1 / 2.5:1 |
| Dam Structural Height (feet) | 18.9 |
| Dam Hydraulic Height (feet) | 18.9 |
| **Principal Spillway** | |
| Principal Spillway Type | Broad Crested Weir w/ 3 Gates |
| Principal Spillway Invert El. (feet) | 835.23 |
| Spillway Crest Length (feet) | 16 |
| Center Gate Sill Elevation (feet) | 835.23 |
| Center Spillway Width (feet) | 10’-1” |
| South Wing Gate Sill Elevation (feet) | 839.13 |
| South Wing Spillway Width (feet) | 8’-1” |
| North Wing Gate Sill Elevation (feet) | 839.13 |
| North Wing Spillway Width (feet) | 8’ |
| **Overflow (RCC) Spillway** | |
| Overflow Spillway Type | RCC |
| Overflow Spillway Length (feet) | 140 |
| Overflow Spillway Crest El. (feet) | 847 |
| Overflow Spillway Embankment Upstream/Downstream Slopes (H:V) | 5:1 / 2.5:1 |
| **Auxiliary Spillway** | |
| Auxiliary Spillway Type | Channel w/ Gate |
| Auxiliary Spillway Crest Length (feet) | 8 |
| Auxiliary Spillway Invert El. (feet) | 835.43 |
| Parapet Flood Wall El. (feet) | 850.3 to 852.9 |

The Dam was originally constructed in 1872. In 1910, the Dam and spillway structure were destroyed by a flood and the Dam was reconstructed the same year. The embankment crest was later increased by 3 feet in 1975 and the auxiliary spillway added in 1978. During a major flood in 1986, the City cut the left embankment south of the principal spillway to avoid the spillway failing. This resulted in significant erosion of the left embankment and significant dewatering of the impoundment, requiring the Dam to be rebuilt. In 1990 the left embankment was rebuilt and an RCC overflow spillway was added, per the drawing package for the 1989 reconstruction of the Auxiliary Spillway by OMM titled “White Cloud Dam Reconstruction” included in Appendix A.

## 2.3 Spillways

The principal spillway is located near the center of the embankment with three lift gates and stoplogs. The center gate is 10 feet wide, and the two side gates are 8 feet wide. These gates have slide gates and stoplog bays at each opening. Downstream of the gates, flow passes through a 10-foot-wide culvert, under the road and down to the river. To the left of the principal spillway is a 140-foot-long overflow spillway constructed of RCC. An 8-foot-wide auxiliary spillway is located North of the right embankment of the Dam. The auxiliary spillway is controlled by a slide gate and stoplogs on the downstream side of the roadway (and crest) before discharging to a chute conveying flow downstream to the confluence of the White River.

A parapet wall exists along the upstream side of the dam crest to prevent overtopping of the Dam during flood events. There is a break in the parapet wall at the RCC overflow spillway and immediately left of the auxiliary spillway for a boat launch into the impoundment. During flooding events, the Dam’s Emergency Action Plan states sandbags are to be installed at the boat launch to prevent overtopping.

## 2.4 Impoundment

The Dam impounds approximately 50 acres at the normal water surface El. 845. The reservoir has a maximum depth of approximately 16 feet. The drainage area for the reservoir is 94.52 square miles.

## 2.5 EGLE Inspection Site Deficiencies

The EGLE Dam Safety Division conducted an inspection on May 25, 2022. EGLE determined that the White Cloud Dam's stability and structural condition were in Fair condition.

Under normal loading conditions, no dam safety deficiencies were identified. Rare or extreme hydrologic and/or seismic events could potentially lead to safety concerns. However, the dam is considered to have inadequate spillway capacity, as evidenced by recent overtopping and near overtopping events. Therefore, the dam overall is considered to be in Poor condition. The following actions were recommended in order of priority:

1. **Insufficient Hydraulic Modelling:** Provide an updated hydraulic analysis of the Dam's hydraulics after the recent overtopping and near overtopping events.

2. **Principal Spillway Left Gate Repair:** Repair the principal spillway left gate so that it can be operated as soon as reasonably possible.

3. **Spillway Concrete and Metal Repairs:** Complete the spillway concrete and metal repairs that have been recommended since the 2015 OMM report within 1 year. Concrete deficiencies that should be repaired are located throughout the principal spillway from the upstream piers between spillway gates downstream to the undermining concrete walls adjacent to the overflow spillway. Minor concrete deficiencies exist at the auxiliary spillway walls, although the most significant holes and cracks were repaired in 2021. Additionally, the steel support beams at the principal spillway are in poor condition and should be repaired or replaced.

4. **Vertical Extension of the Walls of the Auxiliary Spillway:** Develop and implement a plan for vertical extension of the walls of the auxiliary spillway between the road and pedestrian bridges within 1 year to provide freeboard during the design flood. This plan should be guided by the updated hydraulic analysis and the Operation and Maintenance Plan. **This item has been completed in the August of 2024.**

5. **Remove Vegetation from Embankment:** Trees and brush should be removed to 10 feet beyond groins and from the upstream water surface to 10 feet beyond the downstream toe of slope, including any vegetation growing within spillway features. Remove the vegetation growing within and adjacent to the overflow spillway, adjacent to the principal spillway downstream concrete walls, right of the principal spillway outlet on the downstream slope, and adjacent to the auxiliary spillway downstream walls. The trees and brush growing along the auxiliary spillway downstream walls provides minimal temperature reduction for the flowing water, and the City should consider removing them to reduce the risk for concrete impairment due to the trees.

6. **Inspection for Seepage:** The slope of the downstream embankment right of the principal spillway between the principal spillway and the Rotary Park is steeper than other portions of the embankment. When this portion of the embankment is cleared, monitor for any seepage or stability issues. Report any issues to the Dam Safety Unit and implement any follow-up repairs as recommended. This part of the slope is also susceptible to tailwater erosion. Consider armoring against erosion at the downstream toe of slope. Additionally, wet areas were observed on the downstream slope right of the principal spillway at the boundary of routine embankment mowing. These areas should be monitored as routine embankment maintenance/mowing occurs. Any changes should be reported to the Dam Safety Unit. If seepage persists or worsens, it should be investigated.

7. **Remove Sandbagging as part of the Emergency Action Plan:** The City's previous EAP called for sandbagging area of the boat launch, where the upstream vertical wall terminates, during times of high impoundment levels. The updated EAP says sandbagging is an emergency action to be completed by city staff, although the specifics are not included. While this is a good temporary measure, the Dam should be modified to provide appropriate freeboard without relying on sandbagging.

8. **Monitor Wall for Changes in Alignment and Position:** Monitor the concrete wall at the top of the upstream slope for changes in alignment and position.

9. **Monitor Wave Erosion:** Some wave erosion was observed on the upstream slope in areas without armoring. Monitor these locations for further erosion and repair, as necessary. Consider installing rip rap armoring along the entirety of the upstream slope similar to what was placed near the left end of the embankment.

10. **Auxiliary Spillway Repair:** Fill and armor the eroded area at the auxiliary spillway's left downstream wall. Consider armoring the existing slope at the auxiliary spillway inlet walls where erosion has previously occurred.

11. **Install Signage:** While it is not required by EGLE regulations, it is recommended that signage and floating barriers (booms) be installed upstream of the spillways to warn and redirect swimmers and boats away from the hazards.

12. **Install Staff Gauge:** Consider installing a staff gage that can help City staff easily monitor and record impoundment levels. A gage would also be helpful when operation of gates and/or stoplogs as needed.

13. **Update Operation and Maintenance Plan:** Provide an updated Operation and Maintenance Plan (O&M Plan) to the Dam Safety Unit by January 30, 2023. The current O&M Plan lacks the detail required to provide the dam operator specific guidelines on safe operation to avoid overtopping.

## 2.6 GEI Site Observations

To assess the current conditions at the site, GEI reviewed available reference information, including the condition assessments performed by EGLE (2022) and provided design drawings. In addition, GEI conducted a site visit on November 7, 2023. Photos from the GEI site visit are provide in Appendix B. The following sections summarize the current condition of the various structures at the site as observed by GEI.

### *2.6.1 Summary of Field Inspection Findings*

In general, the field inspection found the White Could Dam in overall poor condition. The following items were identified and considered noteworthy during the inspection:

1. Significant pavement cracking along the crest of the Dam. This could be indicative of settling along the embankment.
2. Rotation of flood wall at top of left earthen embankment with possible erosion at the toe of the flood wall.
3. Deterioration of roller compacted concrete (RCC) at overtopping section of left embankment.
4. Deterioration of steel bracing on the upstream and downstream sides of the principal spillway and possible brace missing on the upstream side of the principal spillway.
5. Concrete deterioration of the principal spillway.
6. Seepage breakout at the toe of the right earthen embankment starting at the interface with the abutment and extending along the majority of the right embankment toe.
7. Trash rack at auxiliary spillway is currently not in use.
8. Significant deterioration of bridge beams over the primary spillway.

### *2.6.2 Crest of Dam*

There was significant cracking of the pavement observed along the crest of the Dam. The cracking in the pavement over the left embankment could be due to the expansion and contraction of the roller compacted concrete supporting the pavement or due to settlement of the earthen embankment supporting the RCC. A visible dip in the pavement is present at the interface of the principal spillway and right embankment.

There was rotation of portions of the flood wall located adjacent to the roadway on the upstream side of the embankment. This rotation is possibly caused by erosion of the toe of the flood wall along the upstream embankment, rip rap was present on the upstream slope; however, there appeared to possibly be voids beneath the rip rap.

### *2.6.3 Left Embankment*

The upstream side of the left embankment has rip rap armoring along a portion of the embankment; however, a section of the flood wall and the over topping (RCC) section of the embankment lacked rip rap armoring. The RCC on the downstream embankment face shows signs of significant deterioration. Due to the visual deterioration of the RCC, it appears that a surface treatment was not completed at the time of the installation to protect the exposed face of the RCC. In addition, vegetation is growing on the face of the RCC. The toe of the RCC directly adjacent to the outlet for the principal spillway has visible undermining present.

### *2.6.4 Principal Spillway*

The principal spillway consists of a concrete structure with three gates with stoplog slots immediately downstream and a concrete chute with steel cross braces. The stoplogs experienced a failure of lower logs in August 2024, a majority of stoplogs were replaced at that time to address deteriorating conditions of the stoplogs. The concrete shows signs of deterioration throughout the spillway which includes spalling, delamination, and efflorescence. The steel braces from the concrete piers to the concrete spillway walls on the upstream side of the spillway have significant signs of deterioration and section loss. The cross brace between the concrete spillway walls on the upstream side appears to be missing; the 2019 EGLE inspections report photos show a cross brace; however, one was not visible during the GEI site observations. The steel cross braces in the downstream concrete chute have significant signs of deterioration and section loss.

### *2.6.5 Right Embankment*

The upstream side of the right embankment does not appear to have any rip rap armoring present. There is a flood wall adjacent to the roadway on the upstream side of the embankment. There is a break in the flood wall on the right embankment for a boat launch. The City reports that during a flood event sandbags are placed in the opening of the flood wall.

The downstream side of the right embankment has hydrophilic vegetation along the face of the embankment which indicates presence of high levels of moisture. Along the toe of the embankment there was an observed seepage outbreak that starts at the principal spillway and runs along the majority of the embankment toe toward the north.

### *2.6.6 Auxiliary Spillway*

At the time of the observations, the auxiliary spillway was flowing full and little of the concrete chute structure was visible. There were signs of concrete deterioration throughout the visible portions of the structure. The trash rack appeared to be tied in an upright position and was not in use. Additionally, since the auxiliary spillway gate control is downstream of the dam crest, the auxiliary spillway walls were recently raised in August 2024 to meet the same elevation as the parapet wall or higher.

## 2.7 Bathymetric Survey and Sediment Quantification

In October 2023, GEI conducted a bathymetric survey of the Lake White Cloud Impoundment and performed depth of refusal measurements within the upper reach of the impoundment to assess the depth of accumulated sediment.

The bathymetric survey covered all navigable waters within the impoundment, extending approximately 3,000 feet upstream of the Dam. Beyond this point, waterways were not navigable by boat. Using SonarMite hydrographic survey equipment, water depths were measured, collecting echo sounding data from the water surface to the bottom of the impoundment.

The survey provided information on existing ground elevations of the top of sediment within the impoundment, which were then converted to impoundment water depths, as depicted in Figure 2-2. The water depths on Figure 2-2 are based on a water surface elevation (WSE) of 844.9 feet measured onsite on the day of the bathymetric survey. Water depths will be subject to slight variations due to seasonal changes, operation of the spillway and storm occurrences.

Within the surveyed area, maximum water depths reach approximately 16 feet (or existing grade elevation of 829 feet) with the deepest sections found at the upstream toe of the spillway and within the community swimming area south of Pine Street. Of the survey points collected, the average depth was approximately 4.7 feet deep with 47% of the data collected recording water depths under 4 feet deep. However, a significant portion of the impoundment was not navigable by boat, and therefore, the shallowest areas could not be extensively surveyed. If elevations from this section were collected, the average depth would likely decrease. Surface elevations beyond the navigable area were estimated based on bathymetric data and depth of refusal site investigation.

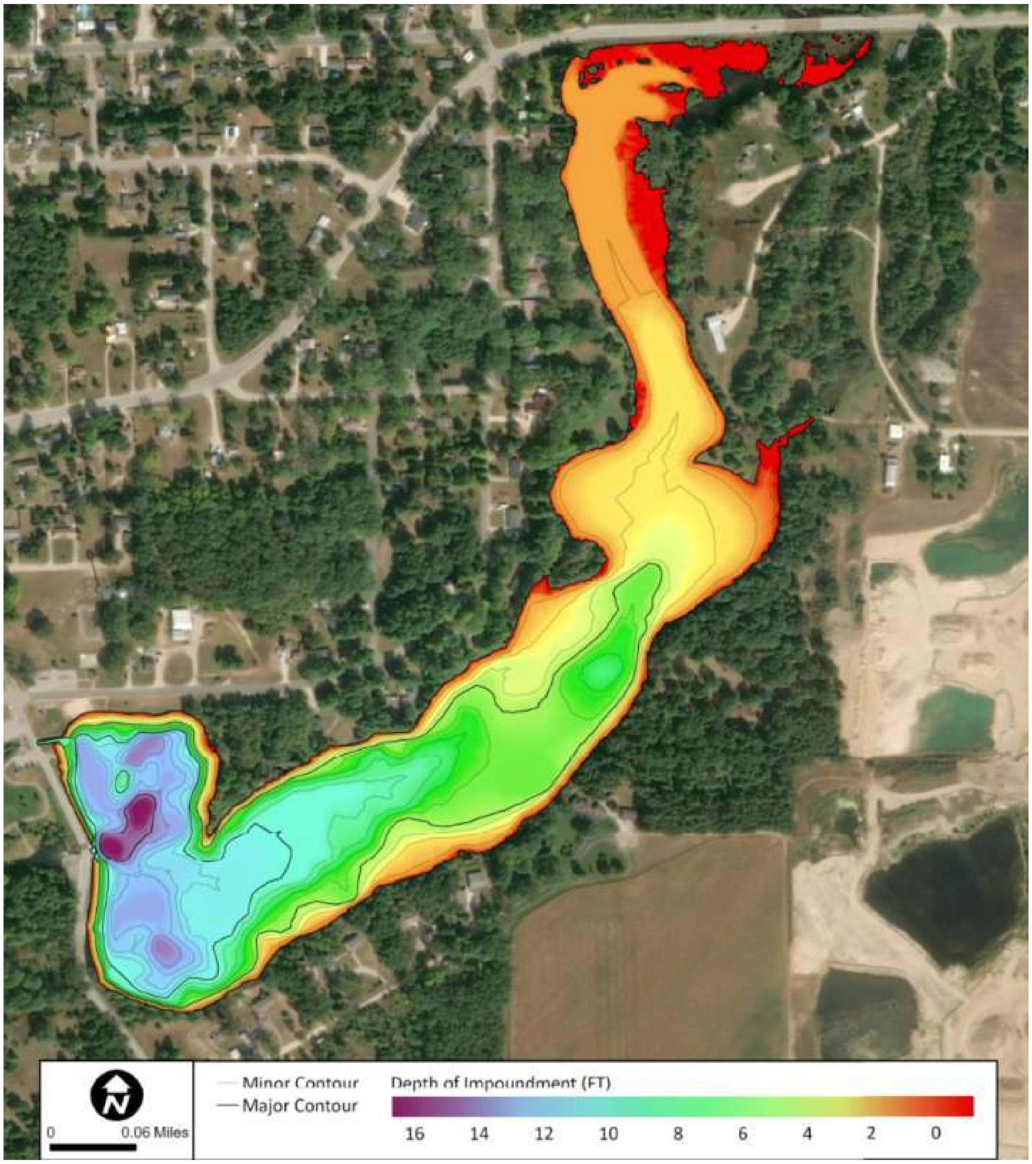


**Figure 2-2: Bathymetric Survey**

To further evaluate sediment accumulation within the impoundment, sediment depths below the existing grade were measured at four sample transects within the upper reach of the impoundment. Depth of refusal survey conducted at this time was limited to what was safely accessible without a boat, therefore, transects were not collected closer to the dam. At each transect, a 1-inch-diameter rod was inserted into the sediment until reaching the refusal point or hard bottom. Combining this method with the bathymetric survey provides insight into understanding sediment accumulation depths and aids in estimating the volume of sediment that would need to be managed during dam removal. Figure 2-3 provides visual representations of the sample transects and cross sections obtained from the depth of refusal survey.

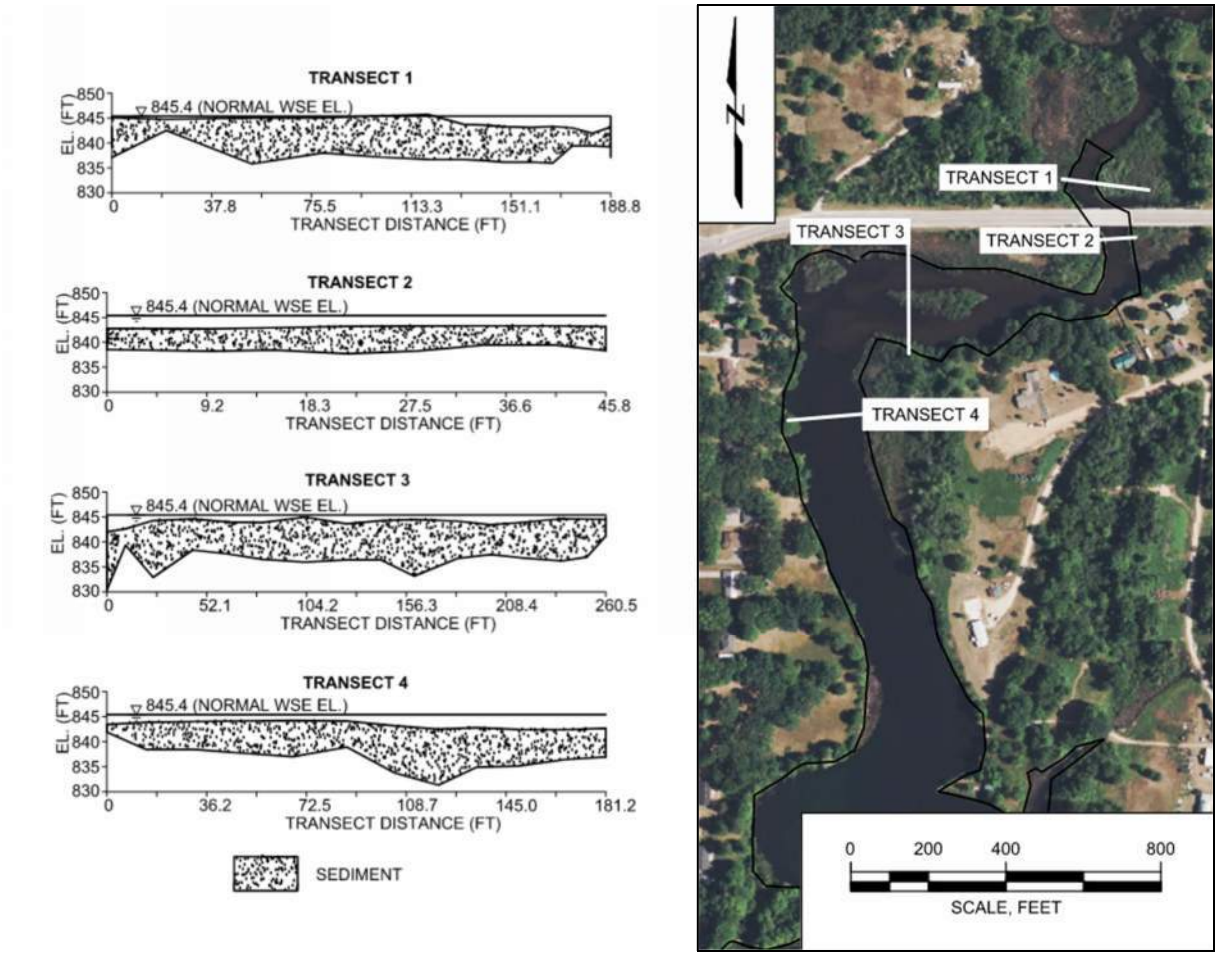


Figure 2-3: Depth of Refusal Cross Sections

In Transect 2, located downstream of the E. Pine Hill Ave overpass, the minimum existing grade elevation is 843.3 feet, with a minimum depth of refusal at 837.7 feet, showing approximately 5.5 feet of accumulated sediment at this location. To avoid impacts to the E. Pine Hill Avenue bridge, at this stage in design Transect 2 serves as an approximation of the tie-in point for a restored river channel following potential dam removal. It is possible that restoration design could target a tie in point further upstream, however grade control would be required at the E. Pine Hill Avenue bridge to avoid unintended impacts to the bridge foundation and additional data collection would be required upstream to determine where impounded sediment terminates. The depth of refusal at Transects 3 and 4 deepens further, measuring at 833.2 feet and 831.32 feet, respectively. The deeper refusal depths observed within Transects 3 and 4 likely indicate the location of the historic channel.

From the bathymetric survey and depth of refusal data collection, a three-dimensional model was created, in part, to estimate the sediment wedge located behind the Dam. As water carrying sediment enters the impoundment, it decelerates, causing sediment to settle out due to the reduced water velocity. This sediment gradually accumulates, forming a wedge-shaped configuration extending from the upstream end of the impoundment toward the Dam. A portion of the estimated sediment wedge in the Lake White Cloud Impoundment is depicted in Figure 2-4 below.

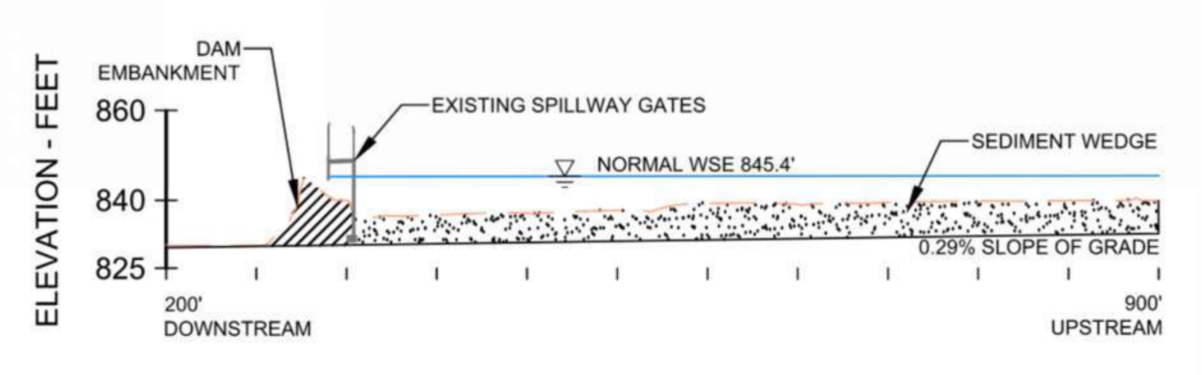


**Figure 2-4: Portion of Estimated Sediment Wedge within Impoundment**

The total volume of sediment within the impoundment can be estimated from an Annual Sediment Yield Rating Curve for the State of Michigan developed by the United States Army Corps of Engineers (USACE). This estimate is based on the watershed drainage area at a given location. With the Lake White Cloud impoundment receiving drainage from a surrounding watershed of 95 square miles, it is projected to accumulate approximately 10,250 cubic yards of sediment annually. Based on there being a dam at this location for over 100 years, the impoundment could potentially contain 1,158,000 cubic yards of sediment or more. However, the reliability of this estimate is compromised due to uncontrolled sediment release during earthen embankment breaches in 1975 and 1986. The exact amount of sediment released during these events is not easily quantifiable, making the total sediment volume within the impoundment uncertain.

If dam removal is chosen as the preferred alternative, additional depth of refusal measurements will be necessary in the middle and lower reaches of the impoundments to accurately determine the location of the historic river channel. These additional measurements will also help to determine a more accurate estimate of impounded sediment contained by the Dam.

Current preliminary design of a restored river channel and floodplain estimates approximately 130,000 cubic yards of material would be needed to be excavated to create the restored conditions. This is discussed in further detail in Section 4.2.

## 2.8 Sediment Sampling and Characterization

Five sediment samples were collected from the impoundment using soil samplers with diameters of 4 inches and 2 inches. These samples were collected from areas with slow moving water where fine sediments were predicted to occur. A composite sample of each core was placed in an amber glass bottle for lab analysis for metal and chemical concentrations and sieve analyses. Figure 2-5 illustrates the coring locations for the five samples. All five samples contained more than 98 percent sand (refer to Table 2). Since contaminants typically do not adhere to sand, and sand is relatively easy to remove and relocate during dam removal and river restoration construction, these findings are significant. If dam removal is pursued, further sampling and characterization within the middle and lower impoundment may yield different results as it is likely finer grain sediment may be present further into the impoundment. Therefore, it will be important that additional sediment sampling be completed if dam removal is selected as the preferred alternative.

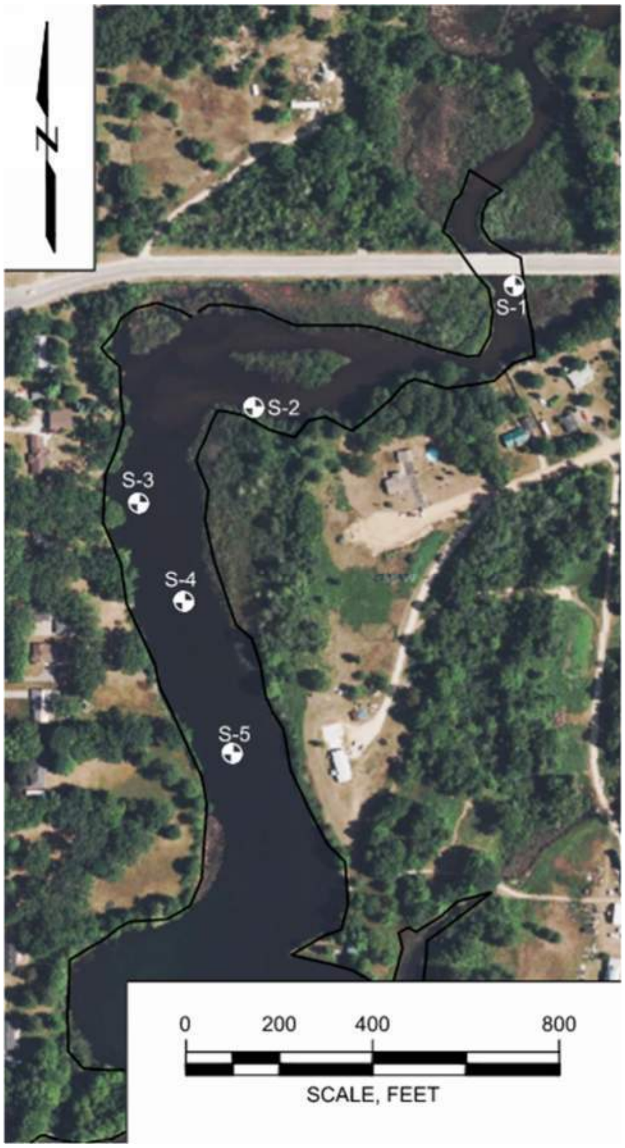


**Figure 2-5: Sediment Sampling Locations**

**Table 2-2: Site Sediment Characterization**

| | S-1 | S-2 | S-3 | S-4 | S-5 |
|---|---|---|---|---|---|
| **Gravel** | 0.6% | 1.1% | 1.8% | 0.5% | 0.3% |
| **Sand** | 96.7% | 97.0% | 96.1% | 91% | 98.6% |
| **Fines** | 2.7% | 1.9% | 2.1% | 8.5% | 1.1% |

The EGLE's Part 201 Program regulates soil contamination and cleanup standards for exposed sediment in dam removal projects. The criteria for direct contact to exposed sediment are determined by land use, specifically whether residential or non-residential land use will exist. In the case of the sediment from the Lake White Cloud Impoundment, the sediment samples were compared to residential direct contact criteria (DCC), the stricter of the two DCC, due to the proximity of the project area to residential areas.

Each sediment sample was analyzed for metals, polychlorinated biphenyls (PCBs), and semi-volatile organic compounds, also known as Polynuclear Aromatic Compounds (PNAs) or Polycyclic Aromatic Hydrocarbons (PAHs). The levels of metals and chemicals in all soil samples were found to be below the residential criteria, indicating that the exposed sediment does not pose a risk to human health. Consequently, there is no need for special disposal considerations or final restoration measures for the sediment. Detailed sediment characterization and analysis results for heavy metals and chemicals can be found in Appendix C.

## 2.9 Survey of Parapet Wall

GEI worked with Holland Engineering to complete a survey of the top of wall of the flood wall (i.e., parapet wall) in October 2024. The elevations of top of wall ranged from 850.3 to 851.1 feet, except for near the auxiliary spillway where the top of wall was between 852.4 to 852.9. The low point in the wall was identified at elevation 850.3 near the south end of the floodwall extent. As noted in previous sections, there are areas of the parapet wall that are showing rotation and cracking.

## 2.10 Geotechnical Investigations

Due to the lack of historical subsurface information, a subsurface exploration and laboratory testing program were required to establish existing subsurface conditions and aid in the analysis of the existing dam condition. GEI prepared a Geotechnical Data Report (GDR) for the dam summarizing the results of the field investigation and laboratory testing. The GDR is included in Appendix D.

Soil boring locations were selected to sample and characterize the various embankment and foundation soil strata of geotechnical interest at each dam location. Subsurface explorations were performed at the dam in October 2024, under the oversight of a GEI field professional. Subsurface investigations included standard penetration (SPT) borings at the crest of the dam.

Laboratory testing of representative disturbed SPT samples was completed at GEI’s Marquette, Michigan, geotechnical laboratory. GEI prepared a Geotechnical Data Report (GDR) for the dam summarizing the results of the subsurface investigations and laboratory testing.

The intent of the subsurface explorations was to:

- Define the depth of existing embankment fill and define the elevation at the top of the foundation material.
- Collect Standard Penetration Test (SPT) samples to develop a characterization of the subsurface stratigraphy within the embankment and foundation soils layers.
- Classify and define the engineering properties of the constituent embankment fills and foundation soils

The subsurface investigation consisted of three test borings to depths ranging from 40 to 50 feet. The foundation soils were encountered at elevations ranging from 827.5 to 840 feet and generally consisted of fine to medium grained silty sand with trace clay. The foundation soils were overlain by existing embankment fill consisting of a combination of fine to coarse grained sand and some silt. Groundwater was encountered during sampling between elevation 840 to 838 or 9 to 11 feet below the crest of the dam. The boring locations are shown in Figure 2-6 and in the GDR provided in Appendix F.

**Figure 2-6: Boring Location Diagram**

Based on the findings in Boring B-01 the 1989 OMM plans for the reconstruction of the left embankment do not appear to document actual construction conditions. The plans depict a clay core directly beneath the existing roadway and the soil boring indicates a subsurface profile consisting mainly of granular backfill. The proposed clay core was never encountered.

## 2.11 Structural Investigations

Nondestructive techniques were employed in our exploration of the primary spillway slab, wing walls, and side walls to determine steel reinforcing spacing and concrete coverage on October 24, 2024. The method performed was Ground Penetrating Radar (GPR), as described below. Concrete coring was also performed to test for concrete compressive strength at select locations on the spillway slab and wing wall. Figure 2-7 illustrates the locations and coverage of the GPR profiles and concrete cores performed at the primary spillway on October 24, 2024. This data collection was not possible in the Auxiliary Spillway because the gates would not close and there was active flow in the spillway.

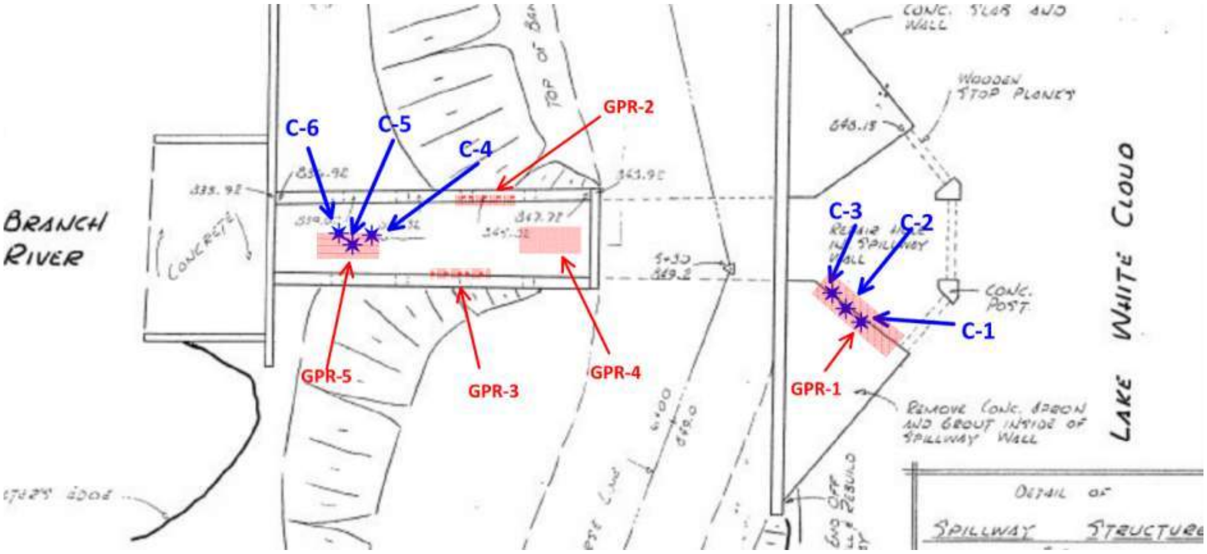


**Figure 2-7: NDT Testing Locations**

### 2.11.1 *Ground Penetrating Radar*

GPR is described in the American Society for Testing and Materials (ASTM) publication D6432, Standard Guide for Using the Surface Ground Penetrating Radar Method for Subsurface Investigation.

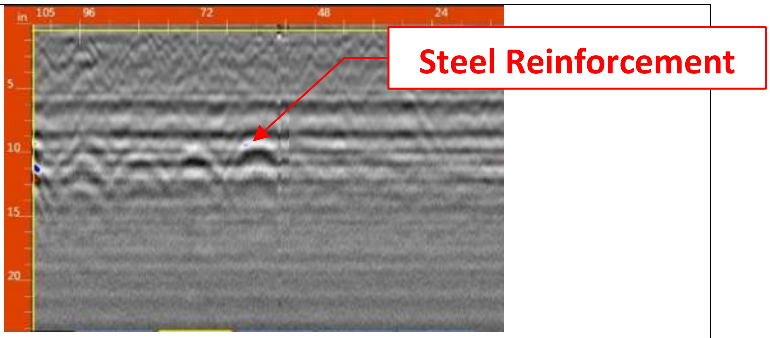
GPR functions by emitting brief impulses of electromagnetic energy (a microwave) from a transmitter coil, and then “listening” for reflections or echoes of those impulses. Different materials transmit the impulses at different speeds and strengths, depending on their dielectric properties. As the impulse travels through the substrate and encounters the interface with a material of differing dielectric properties, some of the impulse energy may be reflected or refracted, in the same way that light is reflected or refracted at the interface between air and water.

The reflected signals are recorded and processed as the antenna passes over the ground, to provide a vertically oriented cross-sectional “slice” of the ground over which the antenna has passed to show the different reflecting layers, and so help pinpoint areas of interest or changes in conditions.

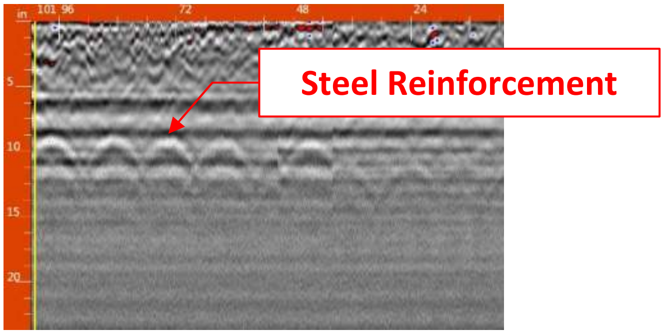
GPR transects were performed on the spillway slab and wingwalls with some additional passes performed over downstream vertical side walls. Several data transects were collected in both the vertical and horizontal axis at five locations to determine steel reinforcement pattern and concrete coverage.

The steel reinforcement in the upper portion of the spillway side walls are visible in the data profiles as a series of small, inverted parabolas at test location Nos. 2 and 3 (Figure 2-8). Horizontal steel reinforcing bars were observed to have 8 to 9 inches of concrete cover and were observed to be on a 12-inch spacing 48 inches above the spillway slab. A potential lower mat of horizontal steel reinforcement is sometimes visible at about 11 to 12 inches depth from 0 to 48 inches above the spillway slab . Vertical steel reinforcement was not observed in the spillway wing and side wall areas scanned during this investigation. The spillway side walls are estimated to be 12 inches thick at the locations tested during this investigation.

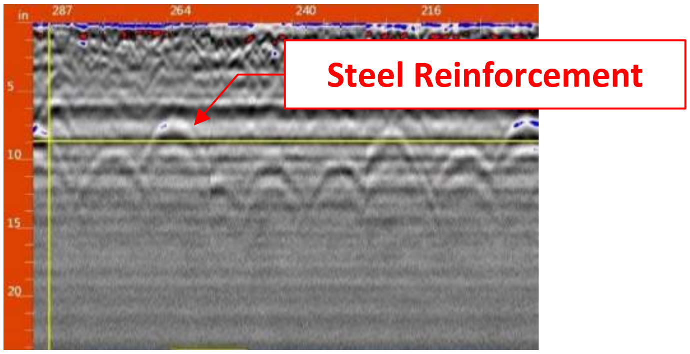
The steel reinforcement in the spillway slab was observed to be constructed on a 12-inch by 12-inch pattern with 9 to 10 inches of concrete cover at both GPR test location Nos. 4 and 5. The spillway slab is estimated to be 12 inches or greater along the tested portions of the spillway slab. The GPR profiles performed at this site did not identify areas with a characteristic voided response below the spillway slab.



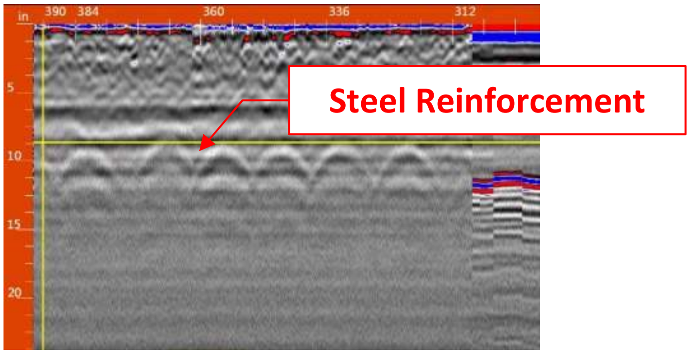
*Data Example No. 1 GPR Location No. 2*



*Data Example No. 2 GPR Location No. 3*



*Data Example No. 3 GPR Location No. 4*



*Data Example No. 4 GPR Location No. 5*

**Figure 2-8: GPR Data Examples**

### *2.11.2 Concrete Compressive Strength Testing*

In addition to the GPR scanning, concrete cores were collected for compressive strength testing. Three cores were collected on the dam spillway wingwall, and three cores were collected from the spillway slab. Prior to compressive strength testing the cores were documented and photographed (Appendix B).

The nine core samples were tested for compressive strength testing per ASTM standard C-39. The samples were delivered to a material testing laboratory for concrete compressive strength testing. The cores were saw cut and capped to provide a tested sample with a 2:1 length/diameter ratio. If the core sample lengths were less than a 2:1 length/diameter ratio, the ASTM correction factor was applied to the test sample results.

The tested compressive strength varied from 2,770 psi to 13,550 psi with an average of 9,722 psi for the south spillway wingwall and 6,740 psi for the spillway slab. The test data is summarized Table 2-3 below and in Appendix G of this report.

**Table 2-3: Compressive Strength Results**

| Sample No. | Sample Location | Date Tested | Compressive Strength (psi) |
|---|---|---|---|
| C-1A | South dam wingwall face | 11/06/2024 | 13,550 |
| C-1B | South dam wingwall face | 11/06/2024 | 11,030 |
| C-2A | South dam wingwall face | 11/06/2024 | 11,200 |
| C-2B | South dam wingwall face | 11/06/2024 | 2,770 |
| C-3A | South dam wingwall face | 11/06/2024 | 12,530 |
| C-3B | South dam wingwall face | 11/06/2024 | 7,250 |
| C-4 | Dam spillway slab | 11/06/2024 | 7,170 |
| C-5 | Dam spillway slab | 11/06/2024 | 6,730 |
| C-6 | Dam spillway slab | 11/06/2024 | 6,320 |

The core holes and borescope holes were filled with Sika Top 123 Plus, a two-part polymer non-shrink grout with a 28-day strength of 6,000 psi. This grout has been used by GEI on multiple spillways and has been approved for spillway patching by the Federal Energy Regulatory Commission.

### *2.11.3 Structural Investigation Summary*

The primary takeaways form the site observations and nondestructive testing are as follows:

- Limited horizontal steel reinforcing was observed on the dam wingwalls 48 inches and higher above the spillway slab. No vertical reinforcement was observed at the locations tested.

- A 12-inch by 12-inch steel reinforcement pattern was observed on the spillway slab with a slab thickness of approximately 12 inches. No voiding was observed in the GPR data sets that are summarized in this survey.
- Concrete core samples collected in the field reveal different aggregate composition in spillway wingwall and spillway slab, corroborating the different era dam concrete repair theory.

This information was used to establish the modeling parameters for the structural stability modeling of the primary spillway, which is discussed in Section 3.3.

## 2.12 Environmental Considerations

A desktop analysis of the project area for environmental considerations was conducted and determined several sensitive ecological resources, including wetlands and protected plant and animal species, occur within or near the project area. The presence of these sensitive ecological resources were identified through review of various state and federal agency databases and evaluated potential impacts to sensitive ecological resources resulting from dam removal or rehabilitation, and implications for the project.

### *2.12.1 Wetlands*

Reviews of aerial imagery and state and federal agency databases indicate the presence of wetlands along the Lake White Cloud impoundment and directly upstream along the South Branch of the White River. Based on data obtained from National Wetlands Inventory (NWI), EGLE, and Web Soil Survey (WSS)-databases, it is likely there are wetlands adjacent to the White River upstream of Lake White Cloud for the entire project reach. Aerial imagery and database maps indicate the wetlands are restricted to a narrow corridor along the White River (Figure 2-9).

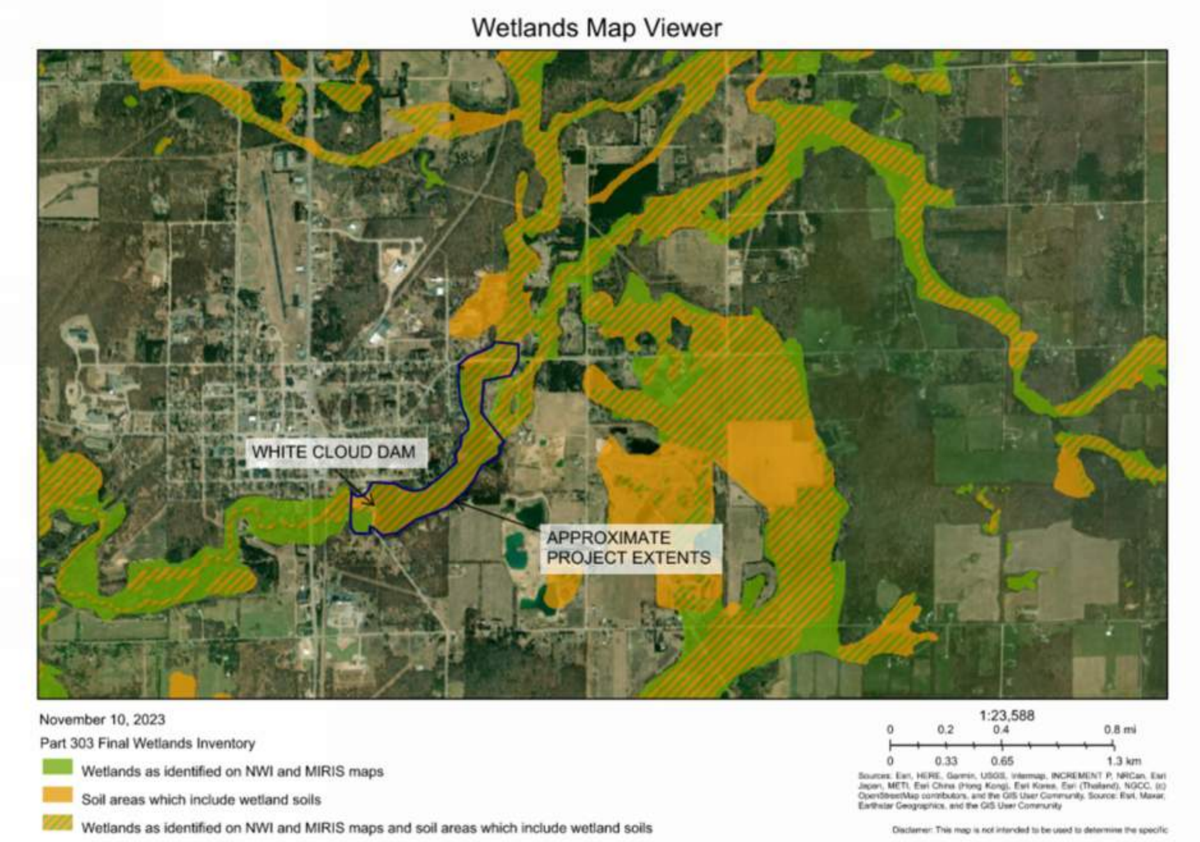


**Figure 2-9: Map of potential wetlands areas near the project site identified by the EGLE Wetlands Map Viewer.**

All wetlands within the project area would be regulated by EGLE pursuant to Part 303 of the Natural Resources and Environmental Protection Act (NREPA) as they are adjacent to and/or within 500 feet of the White River. Any impacts to regulated wetlands require a permit, which can be obtained from EGLE.

### *2.12.2 Threatened and Endangered Species*

Information on threatened and endangered (T/E) species was obtained from the Michigan Natural Features Inventory (MNFI) database. The MNFI database contains location information on species listed as special concern, threatened, or endangered by the State of Michigan. Although not afforded legal protections, special concern species are monitored by the State of Michigan and may be considered when assessing potential ecological impacts.

The MNFI database identified three species listed as threatened by the State of Michigan that are known to occur within 1.5 miles of the project area. These species include wood turtle (*Glyptemys insculpta*), Karner blue butterfly (*Lycaeides melissa samuelis*), and little brown bat (*Myotis lucifugus*). Karner blue butterfly is also listed as endangered by USFWS and therefore subject to federal protections. Three additional special concern species are also known to occur within 1.5 miles of the project area, including pickerel frog (*Lithobates palustris*) (Appendix D).

This desktop review reflects the known state of rare and protected animal and plant species populations at the site as of December 2023. Natural systems and plant and animal populations are dynamic. Conditions within the project area may change to the benefit or detriment of any or all the species listed in this report. All species listed as either state or federally threatened or endangered are protected by law and regulated by either the Michigan Department of Natural Resources (MDNR) and/or USFWS.

Regardless of the preferred alternative selected, any work at the dam would necessitate a field delineation of wetlands and formal assessment of potential impact to threatened or endangered species.

# 3. Assessment of Existing Conditions

Based on the existing conditions data collected and described in Section 2, GEI completed engineering analysis including hydrologic and hydraulic, geotechnical, and structural evaluations. The primary goals of these engineering analyses are to evaluate the existing discharge capacity of the dam, assess the slope stability of the existing earthen embankment, and assess the structural stability of the existing concrete primary spillway structure.

## 3.1 Hydrologic and Hydraulic Engineering

### *3.1.1 Introduction*

The Michigan Department of Environment, Great Lakes, and Energy (EGLE) is the regulatory agency governing the spillway capacity of dams in the State of Michigan. The EGLE dam safety criteria are contained in Part 315, Dam Safety of the Natural Resources, and Environmental Protection Act, 1994 PA 451 (NREPA). Dams are assigned a hazard classification and corresponding minimum flood flow requirements based on the potential downstream impacts caused by failure of the dam. There are three hazard potential categories as outlined in Table 3-1**.**

**Table 3-1: Hazardous Potential Category**

| Classification | Loss of Human Life | Economic, Environmental, Lifeline Losses |
|---|---|---|
| Low | None Expected | Low and Generally Limited to Owner |
| Significant | None Expected | Yes |
| High | Probable (One or More Expected) | Yes |

White Cloud Dam is classified as a high hazard dam and must pass the 200-year event.

### *3.1.2 Hydrologic and Hydraulic Modeling*

GEI developed a hydrologic model for the White Cloud dam and watershed using HEC-HMS software in order to assess flood flows coming into the impoundment. The model includes seven subwatersheds to represent the 95 mi$^2$ watershed. Watershed and channel routing parameters were developed from National Land Cover Database information, Web Soil Survey data, 2020 NOAA impervious area data, 2019 USGS LiDAR data, and GEI's 2023 and 2024 survey information. GEI used a near-overtopping event in August 2021 to validate the hydrology model performance by comparing the model results to observed information. Approximately 3.3 inches of rain fell between August 8 and August 10, with approximately 2.5 inches falling within 24 hours, and as a result the water level in the impoundment nearly reached the RCC spillway elevation. To simulate this event, the hydrology model was run with hourly rainfall data from a Weather Underground station within the watershed from July 31 to August 12. The maximum simulated reservoir elevation was within 0.4 foot of the RCC spillway elevation, which is comparable to observed conditions, and indicates that the model is valid for estimating inflow floods. Following model validation, multiple storm events including the design peak flow event (200-year flood) were simulated using NOAA Atlas 14 rainfall depths and a Type II SCS rainfall distribution. The modeled peak flood flows are provided in Section 3.1.1.

Following development of inflows, GEI simulated routing those flood flows into the impoundment and through the spillways with multiple stoplog configurations. EGLE provided GEI with a HEC-RAS version 6.2 model originally developed by Holland Engineering, formerly OMM from the Summer 2023. The model was originally developed to meet the 2022 EGLE dam safety inspection requirement for updated hydraulic capacity calculation for the Dam.

GEI reviewed the model and made the following changes in order to accurately model the existing conditions at the White Cloud Dam:

- Updated inflow hydrographs with the HEC-HMS modeled flood flows,
- Updated elevations of spillways and floodwall elevations based on 2024 survey data and datum conversion from the as-built plans from NGVD29 to NAVD88,
- Updated controlling dam crest section with the Dam to accurately reflect the floodwall and overtopping crest section of the left embankment,
- Converted the upstream 1D storage area to a 2D mesh to accurately pass flow through the dam spillways,
- Altered weir geometry to reflect the positioning of sluice gates within the spillway structure,
- Added new and modified existing terrain modifications to more accurately represent local topography, and
- Modified the simulation parameters to more accurately represent event conditions.

The model was run under several storm event scenarios. The first set of scenarios assumed that the impoundment was at normal summer pool elevation (845 feet) at the beginning of the event and the stoplogs were at their normal level and were not manipulated during the event. The second set of scenarios assumed that all stoplogs were removed before the water level in the impoundment began to rise, which effectively lowered the impoundment 1 foot below summer pool elevation before the storm began. The final set of scenarios tested various stoplog manipulations during a 10-year event to prevent overtopping the RCC spillway. All scenarios assumed that the slide gates were completely raised for the duration of the storm event. All scenarios assumed that sandbags were effectively installed at the boat launch located on the right embankment up to the flood wall elevation. All scenarios assumed the auxiliary spillway walls between the road and the pedestrian bridge were raised to elevation 851 feet.

### 3.1.3 Hydrologic Modeling Results

The flood flows simulated using HEC-HMS are provided in Table 3-2. The modeled 200-year storm peak flow was 3,1567 cfs, which is higher than the EGLE estimated peak flow of 2,600 cfs. The EGLE database estimates were developed from a regression equation, so the HEC-HMS simulated flows are expected to be based on a more detailed representation of watershed conditions and hydrologic processes.

**Table 3-2. Model Results compared to EGLE Peak Flows**

| Storm Event | 24-Hour Rainfall (inches) | EGLE Flow (CFS) | HEC-HMS Flow (CFS) | Difference (CFS) |
|---|---|---|---|---|
| 1-year | 2.15 | -- | 192 | -- |
| 2-year | 2.42 | -- | 205 | -- |
| 5-year | 2.98 | -- | 234 | -- |
| 10-year | 3.56 | 1200 | 267 | 933 |
| 25-year | 4.52 | -- | 446 | -- |
| 50-year | 5.41 | 1900 | 891 | 1009 |
| 100-year | 6.41 | 2300 | 1878 | 422 |
| 200-year | 7.55 | 2600 | 3157 | -557 |
| 500-year | 9.24 | 3200 | 5022 | -1822 |

### 3.1.4 Hydraulic Capacity Results

The results of the hydraulic analysis show that the floodwall, which is 850.3 feet at the lowest point, was overtopped during the 200-year storm event regardless of stoplog operation, even when all stoplogs were removed before the flood hydrograph arrived, which effectively lowered the pond 1 foot below the typical summer pool (845 feet) before the storm arrived (see Figure 3-1). The simulated 100-year storm event overtopped the RCC spillway and had minimal freeboard (0.5 foot) to the floodwall when all stoplogs were removed before the flood hydrograph arrived, which effectively lowered the pond to 1-ft below the summer pool before the storm arrived. At normal summer pool levels, with no manipulation of stoplogs during the event, the simulated 2-year event had approximately 0.3 foot of freeboard to the RCC spillway, and the 5-year and larger events overtopped the RCC spillway (Figure 3-2). The 10-year event was prevented from overtopping the RCC spillway (with 0.4 foot of freeboard) by removing a total of 4 feet of stoplogs from both sides of the auxiliary spillway and all three primary spillway gates at a rate of one 6-inch board per hour from each gate after the reservoir rose 0.5 foot above normal summer pool at the beginning of the event (Figure 3-3). Removing 6 feet of stoplogs from this scenario resulted in 0.9 foot freeboard to the RCC spillway.

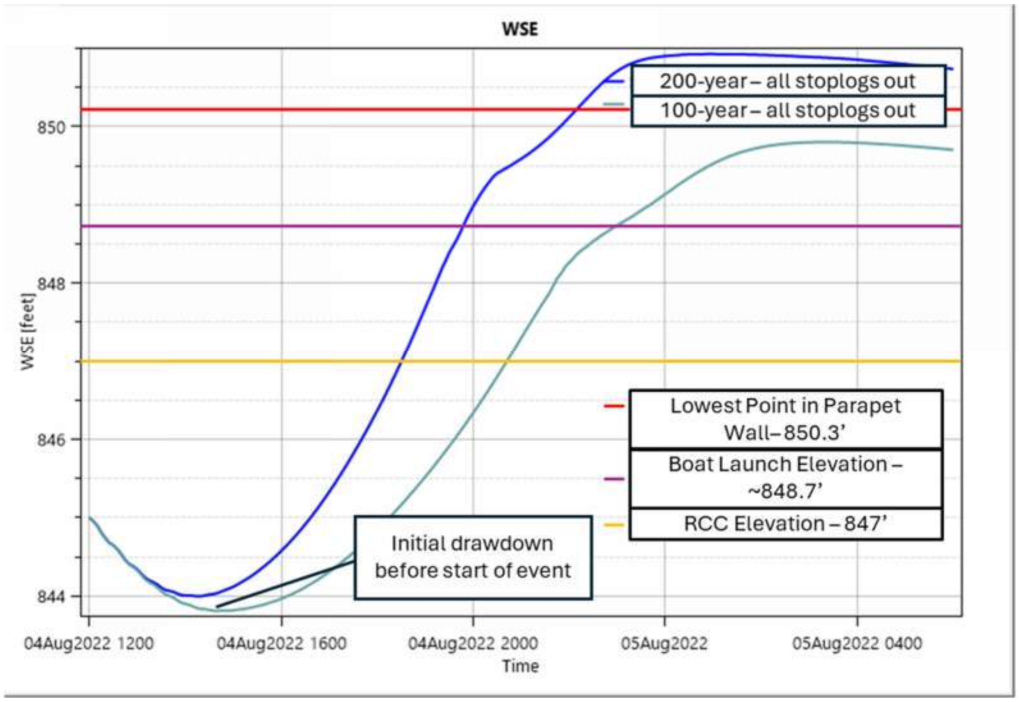


Figure 3-1: 100-year and 200-year Simulated Water Levels with Stoplogs Out

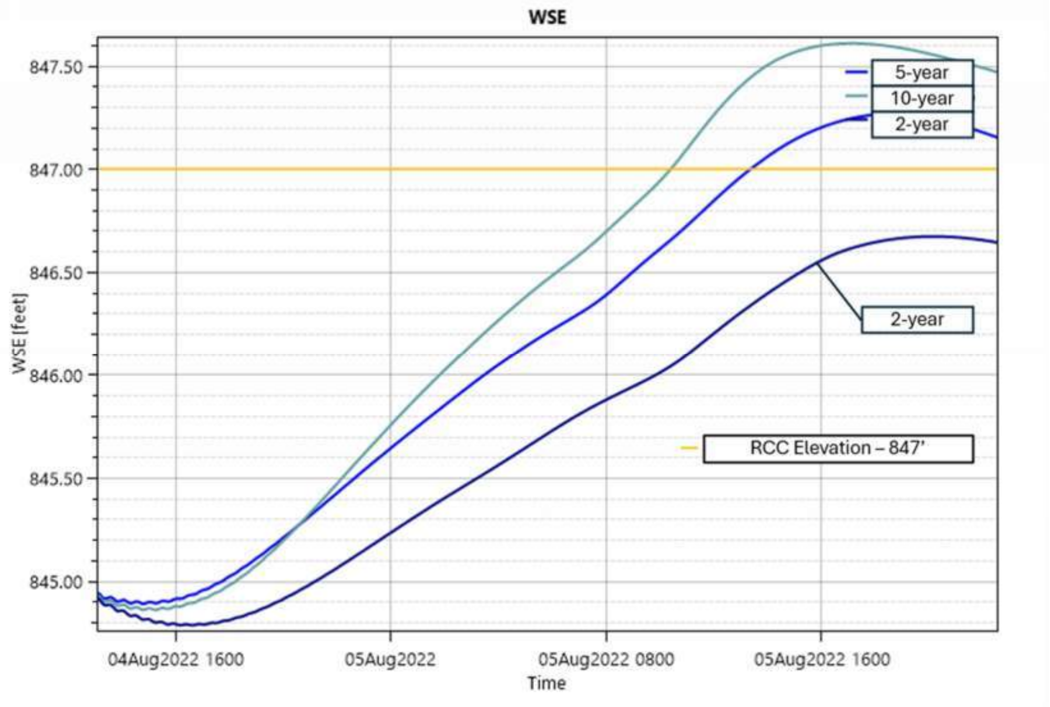


Figure 3-2: 2-Year, 5-Year, and 10-Year Simulated Water Levels with Stoplogs In

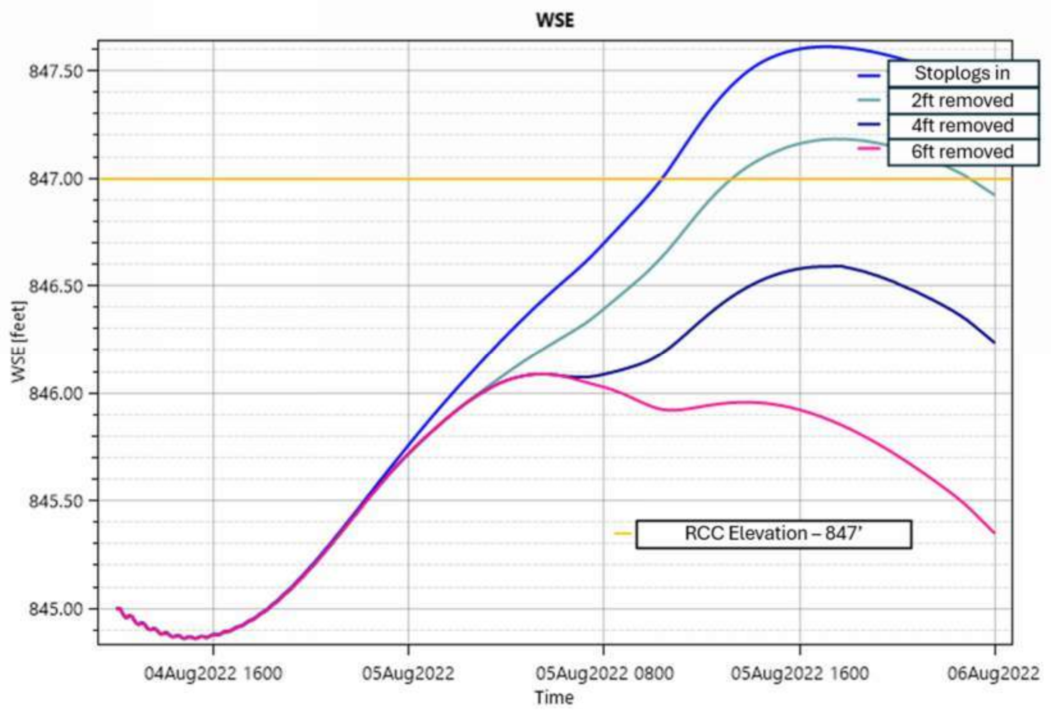


**Figure 3-3: Stoplog Scenarios for 10-Year Event**

Storm events larger than a 50-year event will require sandbags to be installed at the boat launch to prevent overtopping regardless of stoplog manipulation, since the boat launch is overtopped at approximately 848.7'. Stoplogs should be removed for 50-year storm events and smaller to prevent overtopping at the boat launch.

An important note is that all storm hydrographs developed in HEC-HMS assumed normal antecedent conditions leading up to the modeled event. In other words, the watershed soils were not completely saturated or completely dry. If conditions leading up to an actual real-world event had saturated soils from recent rainfall, the incoming hydrographs would be larger than those shown here, and additional stoplogs should be removed.

The slide gates at both the primary and auxiliary spillways were assumed to be kept fully open and above the water level for the entire duration of all flood events so as to not restrict flows. The White Cloud dam operations manual states that the slide gates can restrict the flow during a flood and allow the stoplogs to be accessed for removal or replacement. Based on the results of this hydraulic analysis, it is recommended to only lower the slide gates briefly if needed to remove stoplogs and then raise the gates above the water level again after stoplogs have been removed.

Based on these results, the White Cloud Dam does not appear to have adequate spillway capacity for the 200-year event. Even with all stoplogs removed prior to the start of the event, there is a risk of overtopping the floodwalls (Table 3-3). The impoundment should be managed in part by using weather forecast information. Forecasted 10-year rainfall events and smaller can be managed by removing at least 4 to 6 feet of stoplogs as soon as the impoundment rises approximately 0.5 foot as shown in Figure 3-3 to avoid overtopping the RCC spillway. Forecasted events larger than the 10-year should be managed by removing all stoplogs before the impoundment begins to rise to prevent or minimize the amount of floodwall overtopping. A hydraulic technical memo is included in Appendix E.

**Table 3-3: Hydraulic Capacity Results**

| | **Stoplogs Removed Before Event/Gates Open** | **Stoplogs Removed Before Event/Gates Open** |
|---|---|---|
| Flood Year | 100 | 200 |
| Peak Flow (cfs) | 1878 | 3157 |
| Maximum Water Surface Elevation (feet) | 849.8 | 850.9 |
| Overtopping RCC Spillway | Yes | Yes |
| Freeboard (feet) Measured from Top of Parapet Wall | 0.5 | Overtopped |

### *3.1.5 Wave Run Up Calculations*

Wave run up calculations were not complete since the dam overtops during the design flood.

## 3.2 Geotechnical Engineering

This section summarizes the geotechnical analyses for the earthen embankments. The right earthen embankment adjacent to the principal spillway structure was selected for evaluation as it is the steepest unarmored earthen section with visible seepage at the toe of the embankment. Key components of the geotechnical analysis include estimating material properties and completing embankment seepage and slope stability analysis and are discussed in depth in Appendix F.

### *3.2.1 Embankment Seepage and Stability Analyses*

Downstream embankment stability analyses were performed in accordance with the criteria provided in the USACE Engineering Manuals (EM 1110-2-1901, EM 1110-2-1902, and EM 1110-2-2300). The SEEP/W and SLOPE/W modules of the GeoStudio software package were used to model seepage and slope stability of existing conditions. Section geometry was based on the publicly available existing Lidar topographic information.

Following the previously completed preliminary stability analyses, a geotechnical investigation was completed. A total of three geotechnical soil borings were completed along the crest alignment of the dam on October 23, 2024. Subsurface conditions for the embankment fill and foundation soils were characterized based on the geotechnical investigation results and laboratory data. Phreatic surface water levels in the Dam were assumed based on normal pool headwater, water surface elevations obtained while drilling, and visual site observations of seepage expressed on the downstream face. No long-term phreatic surface data is available as monitoring wells are not present within the right embankment.

The phreatic surface through the embankment was estimated using SEEP/W and the stability analysis was performed using the program SLOPE/W. The shape and location of critical slip surfaces considered were required to either:

- Breach the embankment crest, and/or
- Intercept the phreatic surface leading to loss of the impoundment.

Shallow (sloughing-type) failure surfaces, which do not meet these criteria, are considered routine maintenance issues that could be immediately addressed though the programmatic upkeep of the facility and are therefore not considered critical to dam stability. A minimum slip surface depth of 5 feet was set for failure surface searches.

### *3.2.2 Minimum Required Factors of Safety and Loading Conditions*

The normal pool loading and flood pool loading conditions were evaluated at the representative cross section and compared to USACE Required Factor of Safety (FS) summarized below:

**Table 3-4: Summary of Minimum Required Factors of Safety**

| Loading Condition | USACE Minimum Required FS (EM 1110-2-1902) |
|---|---|
| Steady State Seepage, Normal Max Pool | 1.5 |
| Steady State Seepage, Surcharge Pool | 1.4 |

### *3.2.3 Material Properties and Seepage Calibration*

The subsurface conditions for the embankment and foundation soils were determined from the results of the geotechnical investigation. The following parameters were assumed in the seepage and slope stability analysis:

**Table 3-5: Material Properties**

| Soil | Unit Weight (pcf) | Friction Angle (deg) | Hydraulic Conductivity (ft/sec) | Hydraulic Conductivity Ratio | USCS Classification |
|---|---|---|---|---|---|
| Embankment Fill | 125 | 33 | $4 * 10^{-6}$ | 0.02 | SM |
| Foundation | 127 | 34 | $4 * 10^{-6}$ | 0.02 | SM |

The results of the geotechnical investigation indicated that the embankment consisted of similar material ranging from USCS classifications of SP – SM. Soil index properties and strength parameters for the embankment fill and foundation materials were developed from the geotechnical investigation results, published correlations between SPT blow counts and vertical effective stress (Gibbs and Holtz, 1957), and published correlations between SPT blow counts and relative density (NAVFAC DM 7.1, 1986). The soils were assumed to have no effective cohesion due to the USCS classifications and laboratory testing performed. Material parameter development and selection is presented in more detail in Appendix F.

One representative section along the dam was chosen and developed to perform the seepage calibration analyses. The SEEP/W model was calibrated to the exiting conditions based on the normal pool headwater elevation, water level elevation in the borings, and seepage breakout at the toe of the right earthen embankment. The calibration included adjusting the conductivity ratios of the soils and adjusting the hydraulic conductivities to most closely match the seepage observed on the downstream face at the site for a normal pool headwater elevation. The conductivity ratio was assumed to be 1.0 initially and then modified as needed to calibrate the seepage model to match the seepage model location. The calibrated seepage model is included in Appendix F. Following the calibration of the normal pool loading condition, the same parameters were used to model the flood pool loading condition with an increased headwater elevation. This assumes that the phreatic surface through the embankment soils would respond similarly to the normal pool loading condition.

### *3.2.4 Stability Analysis Results*

Based on the available historical information, topographic data, visual observations, and subsurface information a seepage and slope stability model was evaluated for both normal pool and flood leading conditions. These results were evaluated in comparison with USACE guidelines for minimum required factor of safety values. The results of the slope stability analysis for both the normal pool and flood pool loading conditions are summarized below:

**Table 3-6: Stability Analysis Results**

| Load Condition | Calculated FS | USACE Minimum Required FS |
|---|---|---|
| Normal Pool (845.5 feet) | 1.2 | 1.5 |
| Flood Pool (848.8 feet) | 1.1 | 1.4 |

Based on the condition observed on-site, the assumptions, and analysis completed, the slope appears to be stable in its current condition. However, industry standards recommend a FS of at least 1.5 for normal operating conditions and 1.4 for surcharge (flood) loading conditions, which the current dam embankment does not meet.

Based on the 2014 USGS seismic hazard map for Michigan and commonly accepted standards of practice as defined by the FERC Engineering Guidelines, it is not considered necessary to perform a site-specific seismic hazard analysis for the White Cloud Dam; therefore, the analysis did not account for seismic loading. Appendix F discusses this in more detail.

## 3.3 Structural Engineering

The existing principal spillway consists of the inlet structure (gates and stoplogs with concrete weir controlling section), box culvert, and chute. The following sections describe the basis of the structural analysis and a summary of the analysis results. The structural analysis criteria and evaluation are included in Appendix G. In addition, GEI preformed an analysis to establish a maximum allowable single-axle weight for the bridge over the dam spillway. These findings are provided in the Bridge Investigation and Temporary Repairs Memo dated November 7, 2024, and also attached in Appendix G.

### *3.3.1 Global Stability Analysis*

The inlet spillway is a trapezoidal structure which consists of three inlet gates, a principal gate, and two secondary gates with multiple inlet levels. It is 36 feet wide at the upstream edge and 10 feet wide were it transitions to the principal spillway chute. The global stability for this structure was considered in two parts to resist hydrostatic forces as follows and outlined in Figure 3-4:

- Overturning - Primary inlet structure (upstream of roadway).
- Sliding - Primary inlet plus the expanded inlet structure, chute, and retaining walls.

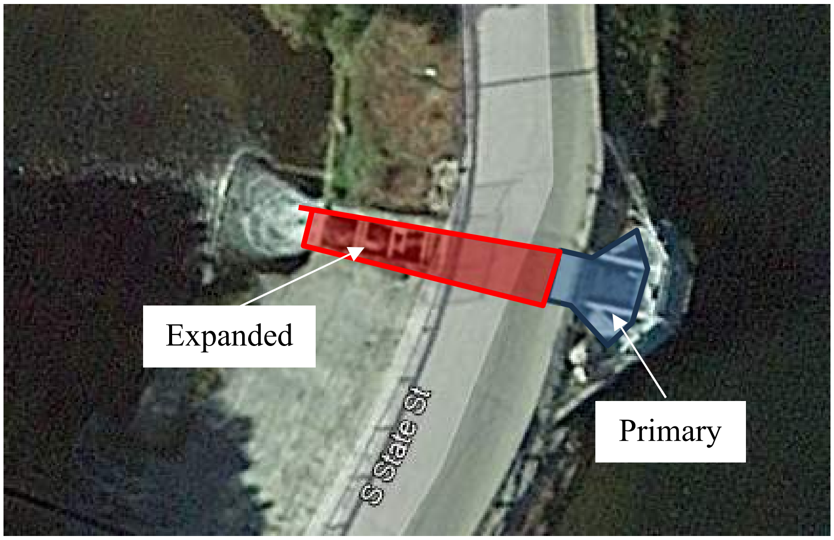


**Figure 3-4: Spillway Analysis Sections**

No existing drawings for the inlet structure and principal spillway were available during the time of Analysis, drawing package for the 1989 reconstruction of the Auxiliary Spillway by OMM titled “White Cloud Dam Reconstruction” were used along with miscellaneous available details and field measurements to scale most pertinent dimensions required to complete the stability analysis. Due to a lack of confirmed geometry, some assumptions have been made to the details of foundation configurations, the structural self-weight has been calculated with this geometry and normal weight concrete. Passive earth pressure resistance has been assumed based on geotechnical data obtained at the site. The passive soil is assumed to be in good condition. Passive earth pressure utilized as resisting forces have been reduced in accordance with the load factors outlined in USACE_1110_2_2100 ‘Stability Analysis of Concrete Structures’ due to the risk of voids/washout occurring during an overtopping event. The friction coefficient chosen for sliding factors of safety has been assumed relative to typical soils found on site and compared to regional typical soils.

The inlet structure was analyzed with details included in Appendix G. Uplift pressures are assumed to be equal to the headwater elevation with a linear distribution down to the tailwater elevation for the length of the inlet structure. This assumption is supported by the reported water level in soil boring locations through the embankment which indicate saturated soil approximately following this distribution. See Appendix F for Geotechnical boring results. Additionally, there is no documented drainage of the inlet. Table 3-7 below summarizes the analysis results along with industry standard factors of safety.

**Table 3-7: Global Stability Analysis Results**

| Structure | Load Condition | Calculated FS | USACE Minimum Required FS |
|---|---|---|---|
| Inlet | Overturning – Normal Pool | Resultant outside base | Resultant within kern |
| Inlet | Overturning – Normal Pool w/ ice | Resultant outside base | Resultant within kern |
| Inlet | Overturning – PMF Gate Open | Resultant outside base | Resultant within base |
| Inlet | Overturning – PMF Gate Closed | Resultant outside base | Resultant within base |
| Inlet & Chute | Sliding – Normal Pool | 14.3 | 3 |
| Inlet & Chute | Sliding – Normal Pool w/ ice | 6.4 | 3 |
| Inlet & Chute | Sliding – PMF Gate Open | 8.7 | 2.2 |
| Inlet & Chute | Sliding – PMF Gate Closed | 7.8 | 2.2 |

The results show the primary spillway inlet structure does not meet the minimum stability criteria based on current industry standard for overturning conditions. However, the structure was determined to be within the standards for resisting sliding with the embankment intact.

### 3.3.2 Concrete Strength Design

GEI evaluated the cores obtained during the structural site investigation program to inform the in place compressive strength of the concrete slab and walls for the spillway and chute structure. The compressive strength laboratory results are outline in Table 2-3 in Section 2.11.2 and provide in Appendix G. The methodology outlined in American Concrete Institute (ACI) 214.4R-10 “Guide for Obtaining Cores and Interpreting Compressive Strength Result” was used to inform the design strength based on the compressive break values. The analysis of the cores can be found in Appendix G. The results indicate that the concrete has an in-situ strength of approximately 5,000 psi with a 90% confidence that the value is equal to or less than the true value. This is considered a reasonable confidence level given the importance of the structure. The cores obtained from the left spillway walls appear to have been sampled in the section of the spillway that was rebuilt in the 1980s. Core 2B was not used in the compressive strength analysis as it was found to be an outlier in the data set obtained based on the compressive strength value of 2,770 psi compared to all other cores exceeding 6,000 psi. Since the wall cores were obtained in the 1980s rebuild location the compressive strength of the walls built prior to the dam failure were evaluated using 3,000 psi.

GEI evaluated the thickness concrete and presence of reinforcing steel in the concrete slab and spillway/chute walls utilizing GPR. In general, the concrete slab appears to have reinforcing steel place at approximately 12 inches on center in both directions. The reinforcing steel appears to be place approximately 9 inches from the top of slab and the slab is approximately 11 inches thick. The concrete walls did not appear to have steel reinforcement, and the wall are approximately 12 inches thick. The information obtained from the GPR is typical of a structure of this era.

### 3.3.3 Summary of Structural Analysis

The existing intake structure and chute was analyzed at key structural locations: the piers support the gates and stoplogs at the inlet, the wing walls in the inlet structure, the chute wall immediately downstream of the bridge, the chute wall at its shortest span, and the slab along the bottom of the inlet and chute.

The inlet piers are non-symmetrical prismatic column – beams fixed at the base to the slab foundation (exact geometry and fixity below the slab is unknown) and pinned at the top in the axis of flow via a steel strut. The piers are loaded via hydrostatic pressure from either the stoplogs or gate and are principally subject to bending and shear forces. No GPR or reinforcement data was available on the piers. The existing pier was analyzed utilizing two # 6 bars on the compression face with a resulting demand to capacity ratio of 5.23. The piers were found to NOT be in compliance with current design guidelines.

The intake structure is constructed of two wing walls funneling into the downstream chute, the wing walls are angled at 39 degrees outward along the axis of flow. The left wall experienced a failure in 1986 and was reconstructed. Then North wall is assumed to be constructed during the

1910 remodel, but exact age of the north wing wall could not be confirmed. GPR of the south wall indicates unreinforced concrete for the bottom 48 inches of the wall. The wing walls were analyzed assuming a dry condition, meaning no or low flow exists in the inlet and chute and the principal load on the wall is the active earth pressure, the water table was assumed to be linearly distributed from headwater to tailwater elevations. The wing wall was found to have a demand to capacity ratio of 4.07. The wing walls were found NOT to be in compliance with current design guidelines.

The intake chute then passes downstream beneath the bridge. The chute slab descends to the tailwater elevation with a flip bucket prior to entering the White River. Starting immediately downstream on the bridge the chute wall steps down with the embankment slope. No existing drawings were available on the chute wall. The geometry of the existing chute walls has been inferred from the reconstruction drawings titled White Cloud Dam Reconstruction dated October 1989 and from field observations and measurements. GPR on the chute walls indicate and unreinforced condition on the chute wall below 48 inches with mesh reinforcement above. The chute walls were analyzed at the maximum and minimum span heights, assuming a dry condition (low flows) in the inlet and chute and the principal load on the wall is the active earth pressure, the water table was assumed to be linearly distributed from headwater to tailwater elevations. The Chute wall maximum span was found to have a demand to capacity ratio of 5.89. The chute wall maximum span was found NOT to be in compliance with current design guidelines. The Chute wall minimum span was found to have a demand to capacity ratio of 0.89. The chute wall minimum span was found to be in compliance with current design guidelines.

Finally, the slab section which spans between the chute walls was analyzed in a dry condition for uplift forces. No existing drawings exist of the slab reinforcement or configuration. GPR performed on the slab indicates a slab thickness of 11 inches with rebar placed at 12 inches on center. The size of the reinforcement could not be confirmed, #4 bar was assumed for this analysis. Due to the load reversal experienced during uplift, the reinforcement, located near the bottom of the slab, does not contribute to slab capacity. The slab was found to have a demand to capacity ratio of 4.42. The slab was found to NOT be in compliance with current design guidelines.

### 3.3.4 Bridge Load Rate

GEI performed a preliminary bridge rating of the existing structure that spans over the dam’s primary spillway to inform temporary repairs and future recommendations. The temporary recommendations include limiting the speed limit for vehicles with an axle weight of 10 tons (20,000 lbs) or more and installing two 1-inch steel road plates stacked to span over the existing bridge. For reference calculations and temporary repair sketch refer to the Bridge Investigation and Temporary repair letter attached in Appendix G. Ultimately, GEI recommends performing an in-depth bridge inspection with any long-term load rating or repairs from the inspection findings, removing and replacing the bridge with no additional investigation, or fully shoring the bridge with inspection only to cover what is necessary to provide the shoring. GEI recommends that this should be done in the next 3 to 5 years and that speed limit and loading rating of the bridge be limited to the values above until this work is completed. While the temporary measures are in place, the City should inspect the plates and asphalt ramps quarterly to monitor for changes. The asphalt ramp may need occasional repairs/replacements depending on traffic wear.

## 3.4 Summary of Dam Observations

Based on the dam inspection performed by EGLE, the limited site observation performed by GEI and the above outlined analysis the following observations and deficiencies have been noted (Table 3-8) about the dam and have been categorized as follows.

- Dam Safety: Ensuring dams are constructed, operated, and maintained in a manner that protects people and property from the risk of failure.
- Public Safety: Protection of people and their property from harm.
- Operations: Activities involved in maintaining and protecting a dam and the area of affects.
- Maintenance: Regular work performed to keep dam safe and functioning.

**Table 3-8: Summary of Observations**

| Category | Observation/Deficiency |
|---|---|
| Dam Safety | Insufficient hydraulic capacity to pass the 200-year design flood. |
| | Dam modifications required to modify parapet wall to span gap of protection at current boat launch. |
| | Insufficient global factor of safety of right earthen embankment. |
| | Seepage at downstream toe of right earthen embankment. |
| | Tailwater erosion at downstream right earthen embankment. |
| | Concrete deficiencies located throughout the principal spillway. |
| | Primary spillway intake global stability – overturning, does not meet current industry standards for factors of safety or resultant location. |
| | Primary spillway intake and chute concrete structural elements do not meet current industry standards for factor of safety (demand to capacity ratios). |
| | Structural stability concern associated with severe deterioration of beams supporting the existing bridge over the primary spillway. |
| Public Safety | Install signage and floating barriers (booms) upstream of the spillways to warn and redirect swimmers and boats away from the spillway hazards. |
| | Deteriorating fencing and guardrail embedment along roadway shoulder and adjacent to primary spillway chute. |
| | Install fencing at auxiliary spillway crossing. |
| Operation | Remove sandbagging as part of the EAP. |
| | Update O&M Plan with results of H&H Analysis regarding stoplog operation to prevent overtopping. |
| | Trash rack in auxiliary spillway not in use. |
| | Install staff gauge to monitor and record impoundment levels. |

| Category | Observation/Deficiency |
|---|---|
| Maintenance | Rotation of flood wall at left embankment with possible upstream toe erosion. |
| | Fill and armor eroded area at auxiliary spillway left downstream wall. |
| | Remove vegetation from embankments. |
| | Cracking in pavement along crest of dam. |
| | Deterioration of RCC overtopping section on downstream left embankment. |
| | Minor concrete deficiencies at the auxiliary spillway walls. |
| | Deteriorating steel bracing across primary spillway chute to be removed. |
| | Minor concrete repair to traffic barrier over auxiliary spillway. |
| | Inadequate riprap along waterline of upstream slopes. |

# 4. Description of Alternatives

Based on the summary of observations and deficiencies listed above the following alternatives were evaluated:

- Dam Rehabilitation under Current Regulations
- Dam Rehabilitation under Proposed Regulations
- Dam Removal

## 4.1 Dam Rehabilitation

The dam rehabilitation alternatives evaluate maintaining the White Cloud Dam and implementing dam safety repairs based on GEI's preliminary evaluation of the structures. The dam safety observations and deficiencies listed in Table 3-8 above are the basis for the recommended rehabilitation options. Proposed changes to current regulations were evaluated and changes effecting the White Cloud Dam are discussed below.

### *4.1.1 Dam Safety Repairs*

Figure 4-1 outlines the location of the above listed dam safety issues and Table 4-1 below outlines proposed options for the dam safety observations/deficiencies. Each proposed option evaluates the pros and cons, estimated constructed cost plus 30% contingency, and priority of repairs as it relates to ensuring dams are constructed, operated, and maintained in a manner that protects people and property from the risk of failure.

Dam safety repairs were the focus for the dam rehabilitation alternatives because these items have the greatest potential to impact the structure’s overall ability to safely maintain an impoundment and pass flood flows. The other observations/deficiencies categorized as public safety, operation, and maintenance should still be implemented, however dam safety deficiencies are the highest priority to address in a timely manner.

Table 4-1 provides an ala carte menu of alternatives, where applicable, to address each dam safety deficiency identified. Some alternatives address multiple deficiencies. It is expected that a range in costs for initial dam rehabilitation is likely and is dependent on the selection of alternatives to address deficiencies. This list was provided to assist the City with identifying smaller projects that might be advanced to address dam safety deficiencies as funding becomes available.

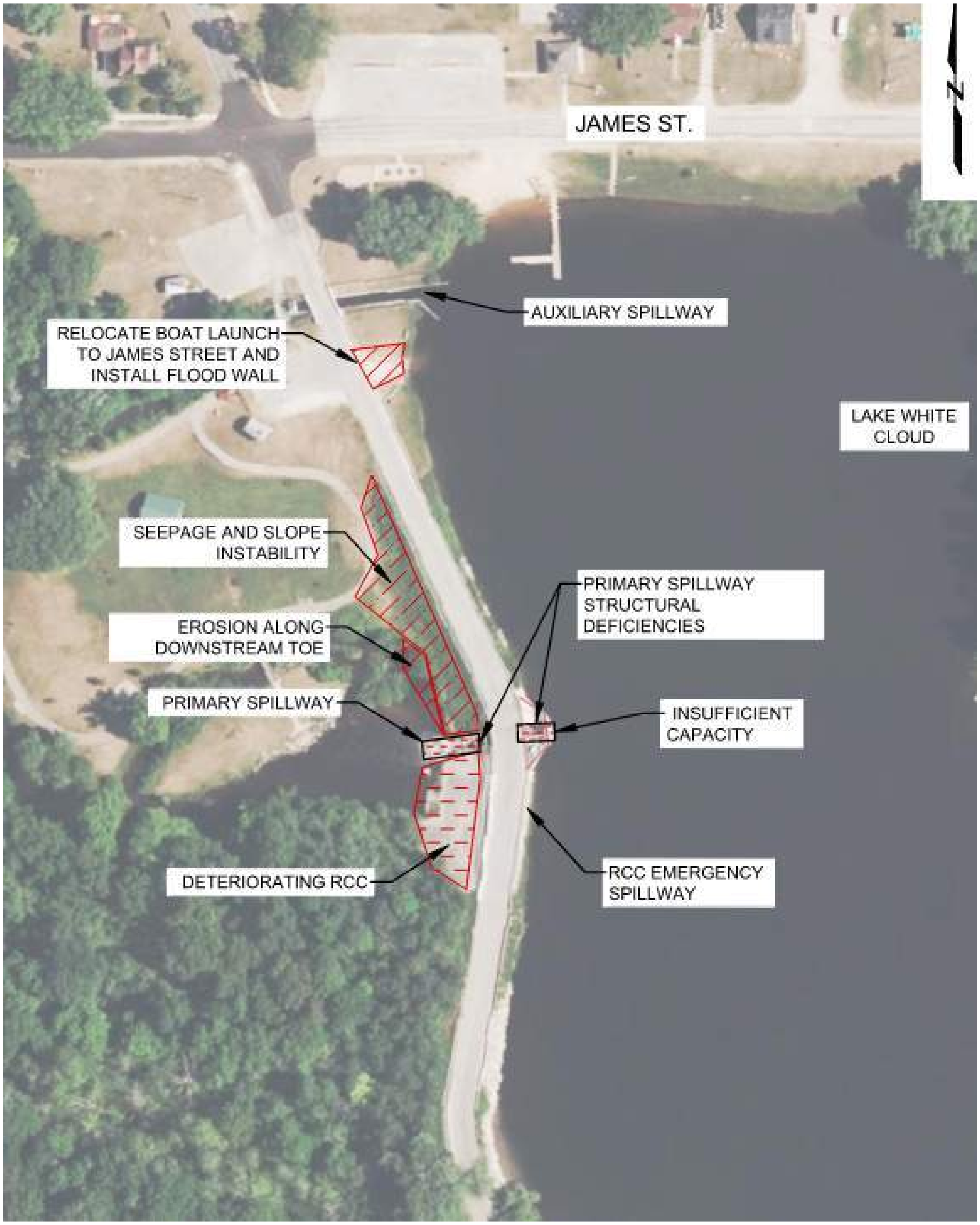


Figure 4-1: Dam Safety Deficiencies

**Table 4-1. Dam Safety Deficiencies and Rehabilitation Options**

| Dam Safety Issue | Repair Description | Pros | Cons | Cost[1] | Priority |
|---|---|---|---|---|---|
| Insufficient hydraulic capacity | Alt 1a: Increase the size of the Auxiliary Spillway to increase discharge capacity and provide a minimum of 1 foot of freeboard as measured from the top of the parapet wall. | Would allow the dam to safely pass the 200-year design flood event without overtopping.<br>During construction could maintain flow through primary spillway.<br>Could potential reuse north retaining wall. | Would require full replacement of auxiliary spillway structure, which isn't that old and is in acceptable condition based on preliminary analysis and visual observations. | $1,170,000 | HIGH – given the previous overtopping or near overtopping events in the last 10 years as well as history of dam failures, this should be a high priority to address. |
| | Alt 1b: Increase the size of the Primary Spillway to increase discharge capacity and provide a minimum of 1 foot of freeboard as measured from the top of the parapet wall. | Would allow the dam to safely pass the 200-year design flood event without overtopping.<br>Would replace a structure that already has structural concerns and deteriorating concrete.<br>Replaces the oldest structure on the dam.<br>Ability to design a new structure with maintenance and operation in mind to assist City DPW for easier operation.<br>Likely would also involve replacement of existing bridge so this option would have multiple benefits | Would require temporary drawdown of the impoundment and bypass flow likely for the duration of construction.<br>Likely would require replacement of spillway inlet, concrete retaining walls, base slab, culvert, and discharge spillway. | $2,730,000 | |
| Right embankment flood wall discontinuity. | Alt 2: Move the existing boat launch to the north side of the impoundment and construct a new floodwall in the current gap. | Eliminates the need for sandbags during flood events. | Requires moving the boat launch. | $260,000 | MEDIUM – While the current operating procedures call for sand bagging this area, it would be best if the boat launch was moved and a continues floodwall is established. |
| Insufficient global factor of safety of right earthen embankment. | Alt 3: Install buttress with graded filter at toe of downstream slope. | Increases global stability factor of safety.<br>Lowers phreatic surface through embankment. | May require excavation into embankment disturbing roadway.<br>May required relocation of path at toe of embankment | $975,000 | HIGH – Given the active an ongoing seepage observed on the embankment and the geotechnical analysis that showed the embankment does not meet industry standard factors of safety under normal conditions, this should be a high priority to address. |
| Seepage at downstream toe of right earthen embankment. | Alt 4a: Install steel sheet pile seepage cutoff wall to lower phreatic surface and limit seepage. | Lengths seepage path through embankment.<br>Depending on location could function and new flood wall. | May not address insufficient global stability factor of safety on downstream slope. | $1,300,000 | |
| | Alt 4b: See Alt 3. | See Alt 3. | See Alt 3. | See Alt 3. | |
| Tailwater erosion at downstream right earthen embankment. | Alt 5: Install riprap erosion protection at right earthen embankment near outlet. | Addresses EGLE Dam Safety recommendation and protects toe of slope from future erosion. | None identified. | $78,000 | LOW – This does not appear to be an imminent threat to the dam; however, this should be regularly observed and if condition worsens armoring should be installed. |

| Dam Safety Issue | Repair Description | Pros | Cons | Cost[1] | Priority |
|---|---|---|---|---|---|
| Concrete deficiencies located throughout the principal spillway. | Alt 6: Repair surficial concrete deficiencies. | Address surface deterioration of concrete. | May not be addressing root cause of concrete deterioration leading to deterioration of repairs and additional deterioration of existing concrete. | $650,000 | MEDIUM – These issues are concerning, however, could all be addressed with replacement of the primary spillway as described in Alt 1b. |
| Primary spillway intake global stability – overturning | Alt 7a: Preform a 3D FEM based stability study. | Less conservative analysis approach. | May still result in unfavorable results. | $13,000 | |
| | Alt 7b: Preform additional field investigations including phreatic surface directly adjacent to the structure, pot-holing to verify subsurface structural dimensions. | Less conservative analysis approach. | May still result in unfavorable results. | $26,000 | |
| | Alt 7c: Modify the structure to add mass to meet current stability. | Improved global stability. | Structural concrete issues remain. | $195,000 | |
| | Alt 7d: Replace existing structure.<br>See Alt 1b. | New intake structure that meets industry standards. Addresses concrete strength deficiency.<br>See Alt 1b. | Significant cost.<br>See Alt 1b. | See Alt 1b. | |
| Primary spillway intake and chute concrete structural | Alt 8a: Selective demo to evaluate concrete thicknesses and presence of reinforcing. | Possibility of favorable strength conditions. | May results in no additional rebar, may uncover additional issues. | $19,500 | |
| | Alt 8b: Remove and replace intake and chute.<br>See Alt 1b. | New structure that meets standards.<br>See Alt 1b. | Significant cost.<br>See Alt 1b. | See Alt 1b. | |
| Structural stability concern associated with severe deterioration of beams supporting the existing bridge over the primary spillway. | Alt 9: Replace bridge over primary spillway. | Fully eliminates deteriorating deck.<br>Reopens roadway. | Utilizes existing spillway walls as abutments.<br>Cost of construction could be significant.<br>Does not address spillway capacity, inlet overturing or structural concrete deficiencies. | $520,000 | |
| Deteriorating Condition of RCC | Alt 10: Overlay existing RCC with protective coating. | Addressed deteriorating top coat of RCC. | May not address longer term potential deterioration of material. | $273,000 | MEDIUM – If the RCC Spillway continues to deteriorate, this could impact the stability of the structure during a flood and could lead to dam failure. To prevent near-term full replacement, it is advised that this repair be completed. |

1. Costs presented in this table are estimated construction costs based on 2024 dollars with a 30% contingency applied to address uncertainties due to the lack of design plans. These costs also do not account for engineering design or permitting costs. More detailed cost breakouts for low and high end estimates to address all dam safety issues are presented in Appendix H.

### *4.1.2 Environmental Impacts*

The dam rehabilitation items will have a minimal impact to the environment, with any impact being constrained to the existing dam and dam embankments. A wetland delineation in the immediate vicinity of the dam on both the upstream and downstream edges of the dam and earthen embankments will be required to quantify wetland impacts. Additionally, a formal evaluation of threatened and endangered species and their habitat within the immediate vicinity of the dam will need to be conducted. Wetland and T/E species impacts as expected to be minimal, given the anticipated footprint of the dam rehabilitation items.

Any work requiring a drawdown of the impoundment will need to consider potential impact to state or federal listed mussel species and timing of the drawdown to limit impacts to aquatic organisms.

### *4.1.3 Proposed Regulation Changes*

In 2021, the EGLE Dam Safety Task Force released a document outlining recommended more stringent regulatory requirements to enhance dam safety in Michigan, which align with national standards. These proposals suggest amendments to Part 315, Dam Safety (Part 315) of the Natural Resources and Environmental Protection Act, 1994 PA 451, as amended. At the time of this report, it is uncertain when and if these recommendations will be included in the Dam Safety Act. However, given the life span of a dam, it is in the interest of the City to evaluate potential long-term added costs if legislation approves more stringent measures. Table 4-2 highlights the major potential regulatory changes that would most significantly impact long-term maintenance of the White Cloud Dam and City obligations. These recommended changes are based on the Dam's classification as a 'High Hazard' dam by the state of Michigan.

**Table 4-2: Summary of Potential Regulatory Changes for High Hazard Dams**

| Regulatory Change | Current | Proposed |
|---|---|---|
| Engineering Inspections | 3 years | 1 year (visual), 10 years (in-depth evaluation) |
| Spillway Capacity | 200-year (1/2 PMF if over 40 feet high) or flood of record | PMF or IDF |
| Licensing Requirements | None | 15-year Registration |
| Financial Assurance | None | Required |
| Insurance | None | Required |
| Emergency Action Plan | Update Annually – No Exercise Requirements | Update Annually – 5-year Exercise Requirement |

#### 4.1.3.1 Dam Inspection Frequency

If dam regulations change, the City may be required to contract and fund yearly high-level visual dam inspections much like what was done in 2023, if not provided by the State as currently done.

In addition to annual inspections, the City will also be required to perform periodic (no more than every 10 years) independent comprehensive reviews of the original design, construction, maintenance, repair, and probable failure modes conducted by a qualified and licensed team of engineers. This comprehensive assessment will likely include exploratory investigations and detailed engineering analyses.

#### 4.1.3.2 Spillway Capacity

Updated regulations will necessitate spillway capacity considerations for either the Probable Maximum Flow (PMF) or Inflow Design Flood (IDF) events. Both PMF and IDF events are used to assess the maximum possible flow rates in water systems. However, they differ in their scope and application. Determining the maximum IDF utilizes a risk-based approach for sizing the spillway, versus the prescriptive approach of the PMF. Aspects comparing the two methods are found in Table 4-3.

**Table 4-3: Comparison of Probable Maximum Flow (PMF) and Inflow Design Flood (IDF)**

| Aspect | Probable Maximum Flow (PMF) | Inflow Design Flood (IDF) |
|---|---|---|
| **Definition** | Theoretical maximum flow rate under extreme meteorological conditions. | A risk-based approach to selecting a design flood based on consequence of failure during discrete flood conditions. |
| **Purpose** | Design and assess the safety of large hydraulic structures, particularly dams. | Balance the risks of hydrologic failure of a dam with the potential downstream consequences. |
| **Calculation** | Based on extreme meteorological conditions, considering factors like precipitation rates and topography. | Based on hydraulic modeling of incremental flood events and consequences of failure. |
| **Frequency** | Extremely rare with a very low probability of occurrence (e.g., "1 in 10,000-year" event). | More frequent, typically with return periods ranging from 50 to 10,000 years. |

Given the current spillway capacity at the White Cloud Dam is insufficient for the current design event (200-year event), accommodating either a PMF storm event or IDF storm event could necessitate doubling the spillway capacity and require significant dam modifications or replacement. An updated Hydrologic and Hydraulic (H&H) Model of the Dam, dam breach inundation analysis, and site-specific PMF study will be necessary to establish site-specific PMF and IDF values. Regardless of which flow calculation method yields the smaller flow rate, the cost to accommodate the updated flow rate will be substantial.

#### 4.1.3.3 Licensing Requirements

Under current regulations, a dam owner only seeks a permit through the State of Michigan at the time of construction or modification. The proposed regulations necessitate the City to apply for a license renewal every 15 years. During the renewal process the City will report on maintenance, operation, and engineering investigations, including annual inspection reports and independent comprehensive reviews. Failure to secure a license renewal could require the removal of the Dam at the City's expense.

The recommended licensing requirements dictate that the Dam owner must maintain adequate insurance to cover all liabilities resulting from a dam failure. The City currently holds an insurance policy with a limit of 3M per occurrence. If a claim were filed against the City of White Cloud for bodily injury or property damage caused by the Dam, the $3M per occurrence limit would apply. However, coverage for a claim involving the Dam could vary depending upon the details of the loss. Based on damages accrued from other dam failures, $3M likely would not sufficiently cover all liabilities from a dam failure and the City's insurance policy would need to be significantly increased.

As part of the licensing renewal, the City is also required to provide evidence of fiscal responsibility or security to ensure the continued safe operation and maintenance of the Dam.

### *4.1.4 Other Benefits and Drawbacks*

In addition to action items needed to maintain the Dam discussed above, Table 4-4 outlines other benefits and drawbacks for this alternative.

**Table 4-4: Benefits and Drawbacks of Maintaining Dam**

| Maintain Dam Alternative | |
|---|---|
| **Benefit** | **Drawback** |
| - Current recreational use maintained.<br>- Meets current community desire. | - Water quality issues and ecosystem disruption.<br>- Disrupts fish passage.<br>- Continued expense for the life of the Dam.<br>- Maintenance costs and aging infrastructure.<br>- Continued sediment buildup.<br>- Initial repairs for maintaining the dam could be more than removal. |

### *4.1.5 Cost*

GEI has developed construction cost estimates for the items listed above. The estimated costs were developed in accordance with AACE 69R-12 - Class 4 which allows for an accuracy range of plus 20% to 50% on the high end, and minus 15% to 30% on the low end, after the application of contingency. This represents about an 80% confidence level that the actual cost will fall within the bounds of the low and high ranges (AACE 69R-12). Our estimated costs include an assumed 30% contingency to account for unknown risks.

Line items for the cost estimate were developed from the rehabilitation items discussed above. In addition to the rehabilitation items listed above we anticipate the dam rehab will likely require a fish passage structure based on MDNR's 2019 report for the White River and understanding of current and future EGLE regulatory requirements. Quantities used in the cost estimate were estimated from photos, available project drawings, and engineering judgement. Unit prices for each line item were developed using a combination of RS Means construction cost estimating software, contractor bid prices from similar construction projects, and engineering judgement. It is important to note that the actual bids and overall project expenses may vary, influenced by factors such as the contractor's perceived risks, site accessibility, seasonal conditions, market dynamics, and other related considerations.

The total estimated cost range to address the dam safety issues alone, plus contingency is **$8,550,000 to $10,770,000**. This cost includes permitting and engineering/construction observation costs in addition to the cost to construct. Detailed cost breakdowns are included within Appendix H.

#### 4.1.5.1 Ongoing Cost

After initial repairs are completed, ongoing financial commitments will be necessary for the dam. If not initially addressed, in the coming years the current list of repairs may continue to grow, and the cost associated with the repairs could increase exponentially.

Additional ongoing costs involve the operation and maintenance of the dam. City personnel will need to continue regularly assessing the dam's condition, conduct routine mowing, and ensure embankment slopes remain free from woody vegetation. They will also be responsible for keeping the spillway clear of debris and conducting regular checks to verify the functionality of all components.

If the City chooses to maintain the dam, additional long-term structural retrofits similar to the current recommended repairs will be necessary. These continued repairs are essential to prevent failure, given the typical 50- to 100-year lifespan of dams.

#### 4.1.5.2 50-year Life Cycle Cost

Given the lifespan of a dam and the requirement for ongoing repairs, it is likely that maintenance similar to what is recommended above will be needed approximately every 50 years. Additionally, over the next 50 years, the Dam will necessitate annual maintenance, operations, periodic inspections, and insurance, incurring additional costs within the evaluated timeframe. Table 4-5 highlights and compares estimated long-term costs of the Dam, outlining initial repairs, 50-year life cycle cost represented in 2024 dollars, and an estimation of the 50-year life cycle cost in future spending based on a 5% annual inflation rate. Once this 50-year life cycle is complete, the Dam will continue to require maintenance and repair as long as it stands. After the completion of this 50-year life cycle, the Dam will necessitate ongoing maintenance and repairs for the duration of its existence.

**Table 4-5: Cost Comparison for Maintaining Dam**

| Cost Comparison | | | | |
|---|---|---|---|---|
| | Low End Estimate | | High End Estimate | |
| | Current Dam Safety Regulations | Proposed Dam Safety Regulations | Current Dam Safety Regulations | Proposed Dam Safety Regulations |
| Initial Repairs | $8.5 Million | $8.5 Million | $10.7 Million | $10.7 Million |
| Life Cycle Cost of Dam through 50 years in 2024 dollars (including initial repairs) | $11.2 Million | $16.4 Million | $13.4 Million | $18.6 Million |
| Life Cycle Cost of Dam through 50 years in future spending (based on 5% inflation rate) (including initial repairs) | $33.8 Million | $47.9 Million | $41.6 Million | $57.8 Million |

### *4.1.6 Funding*

Limited funding opportunities are available for qualified recipients seeking dam rehabilitation. These funding sources would consider the White Cloud dam's overall risk, the extent of necessary repairs of the proposed projects, and the resulting risk reduction from the proposed project. The Dam is classified as high hazard and the EGLE inspection report classifies the dam as an overall poor rating, which may improve the City’s eligibility for grant funding. Any federal funding for the project would also require cultural resources review and State Historic Preservation Office (SHPO) Section 106 permit.

## 4.2 Dam Removal

This alternative proposes the removal of the White Cloud Dam, including the full removal of the spillway and associated structures to facilitate elimination of risks and liabilities associated with the dam and the restoration of the White River to a more natural state.

The following sections summarize the analyses of the factors impacting cost of dam removal and their implications for the White Cloud Dam, the Lake White Cloud impoundment, and the surrounding area, utilizing data from site investigations and engineering evaluations performed by GEI Consultants.

### 4.2.1 Sediment Management

The approach to sediment management and removal is determined by the specific volume and characteristics of sediment at the site. As discussed in Section 2.7, there is a large amount of sediment likely contained within the impoundment prevented from moving downstream by the White Cloud Dam. Sediment data collected as part of this project preliminarily shows little to no contamination concerns associated with the impounded sediment, which would make this material suitable for on-site use and lower dam removal costs.

Given the presumed amount of sediment within the impoundment and important fisheries present in the White River downstream of the White Cloud Dam, sediment management throughout the duration of dam removal should be considered by use of engineered controls (such as turbidity curtains), incremental dewatering/demolition, and construction methods (such as sediment dredging). The use of all three approaches will result in the greatest capture of sediment and prevent the material from moving downstream.

### 4.2.2 Removal of Dam and Management of Water

Managing water flow during dam removal is crucial for safety and sediment control. This process, known as dewatering, involves draining the impoundment behind the Dam. Various methods exist for dewatering and controlling water flow, including installing temporary cofferdams to divert flow around the Dam. Once flow is diverted, the Dam can be demolished in a controlled manner. Other methods, such as bypass pumping or siphon systems, or incremental demolition within active flow for dewatering can also be considered.

Each dam removal project presents unique challenges based on site characteristics and existing infrastructure. For the White Cloud Dam, utilizing the existing spillway gates to dewater and demolish the remaining concrete structure incrementally may be a feasible method of dewatering the impoundment. However, thorough hydraulic and structural assessments are necessary during the design phase to ensure appropriate methods are employed.

### 4.2.3 Impoundment Property Ownership

Based on a desktop review of Newaygo County’s GIS data, property ownership around the impoundment consists of residential, commercial, and public property, with much of the property under private ownership used for residential purposes (see Figure 4-2). North American Refractories Company owns the single lot designated ‘Commercial.’ This lot adjacent to the impoundment is currently vacant land but it is assumed it is part of the commercial operations directly to the south. This mixed ownership surrounding the impoundment could have important implications on construction costs and requirements for dam removal and channel restoration as riparian ownership of the bottomlands could come into question.

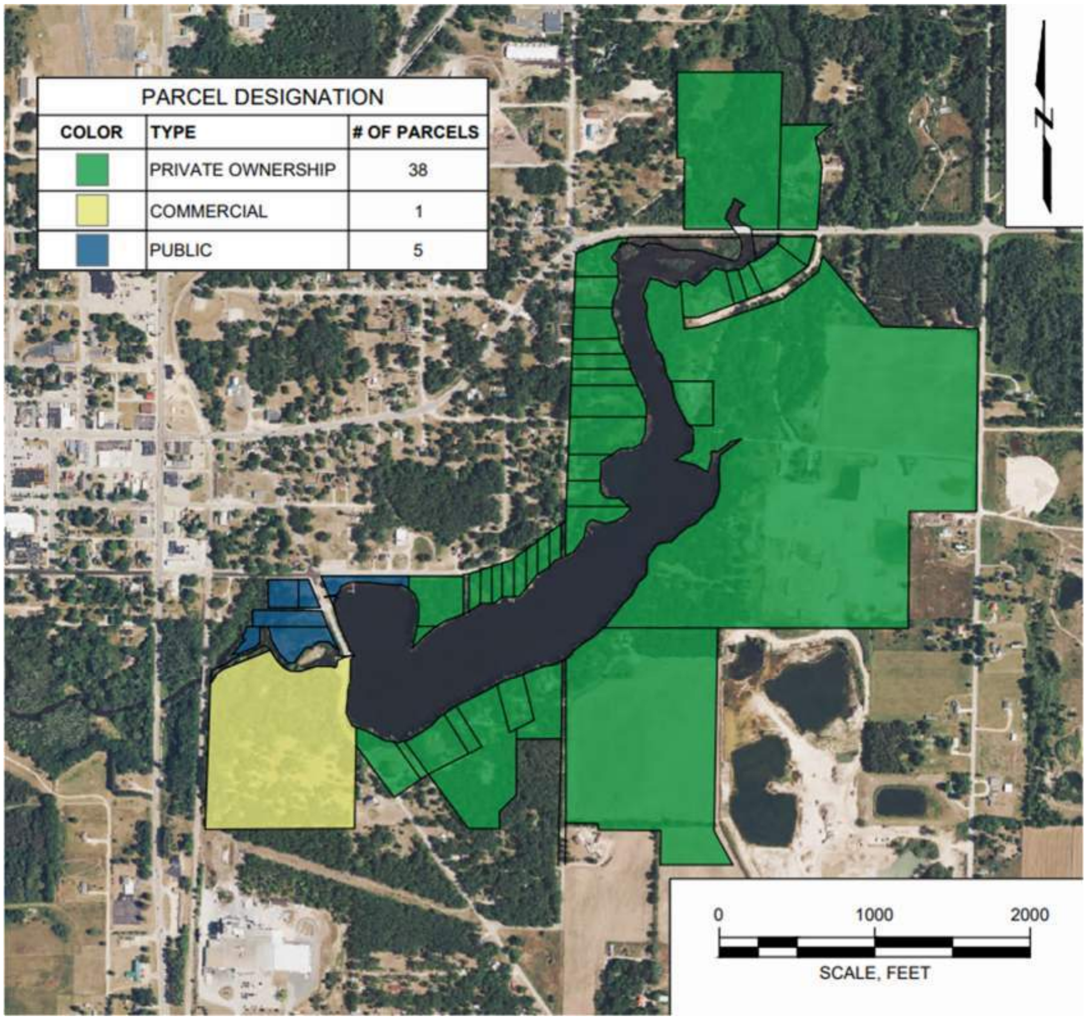


Figure 4-2: Property Ownership Adjacent to Lake White Cloud Impoundment

Dewatering the impoundment would expose nearly 40 acres of submerged land. Most or perhaps all of this land would be floodplain, some would likely form wetlands and some fringe area may form as upland, developable land.

Currently, the way the deed descriptions are shown on the Newaygo County parcel viewer, many legal descriptions within the mapping tool refer to the “EDGE OF WATER” or “LAKE WHITE CLOUD” as a property boundary line. Based on legal descriptions of the adjacent lot and the city’s ownership of the Dam, the City of White Cloud will presumably own the exposed bottomlands. However, if the City were to pursue dam removal, further research and legal consultation will be necessary to define potential future land ownership. A table detailing property ownership and publicly available legal descriptions and an associated parcel map can be found in Appendix I.

### 4.2.4 Environmental Impacts

Dam removal projects often involve assessing and addressing various environmental impacts, including those on wetlands and endangered species. In this context, the wetlands noted in Section 2.9 highlights potential impacts to wetlands downstream of E Pine Hill Avenue, where reduced water levels from dam removal could lead to some wetlands drying out. However, the removal process may also create opportunities for new wetlands to form on exposed impounded sediments within newly created floodplain areas.

The presence of the little brown bat in the area is noteworthy, as it may use trees near the dam as daytime roosting sites. Therefore, if tree removal is part of the dam removal activities, it is important to identify and preserve potential bat roost trees within the project area. Similar to the little brown bat, the northern long-eared bat may also use trees near the site for roosting. Therefore, efforts should be made to minimize disturbance to potential roosting trees during the project activities.

Species like the wood turtle, Blanding’s turtle, and pickerel frog could inhabit wetlands associated with Lake White Cloud and upstream sections of the White River. While changes in hydrology could affect emergent wetlands immediately upstream of Lake White Cloud, some riparian habitat along the riverbanks is expected to remain. Moreover, the creation of new wetlands in the former Lake White Cloud area could provide additional habitat for these species to re-establish.

Wetland delineation of the project area adjacent to the impoundment will be needed along with a formal assessment of T/E species and potential habitat.

Additional environmental considerations will include timing of construction to accommodate fish spawning periods in the White River, and an assessment to determine the presence of sensitive mussel species within the project area.

### 4.2.5 Channel and Floodplain Conceptual Design

After the Dam is removed, the river channel is restored to a more natural state, resembling the width, depth, and meandering of the pre-dam river channel. This restores the natural hydraulics of the river and reintroduces sediment transport to the river reach. Additionally, establishing a sufficient floodplain is crucial to provide relief during larger flood events and promote the stability of the river channel.

Figure 4-3 illustrates a possible stream restoration overview for the Lake White Cloud Impoundment. In this proposed stream channel, the upstream channel would connect to the existing channel just downstream of E. Pine Hill Ave at 844.14 feet, the existing grade of the channel measured during depth of refusal explorations. The tie in point here would limit impacts to the bridge. Since there is existing infrastructure immediately upstream of this tie in point and there is still approximately 5 feet of impounded sediment, grade control in the form of an engineered rock riffle will be needed to stabilize the channel and prevent head cutting upstream. The downstream connection point would be approximately 200 feet downstream of the Dam, at an existing grade elevation of 829 feet. The planned stream restoration would include approximately 5,300 linear feet of new stream channel with an average slope of 0.28 percent.

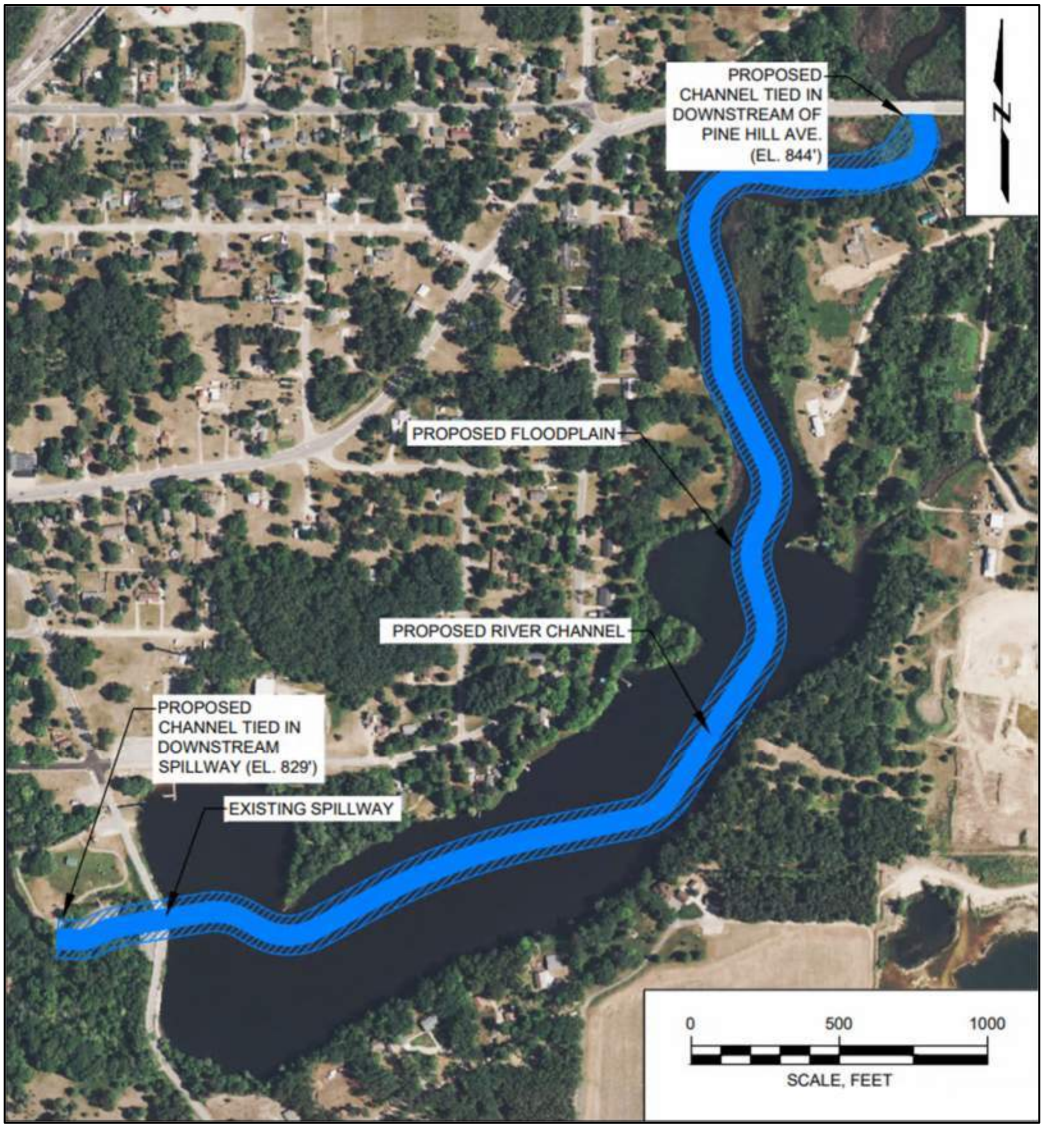


**Figure 4-3: Possible Stream Restoration Overview**

The conceptual design of alternative 3 is based on industry accepted design criteria for rivers found in Michigan. Specific attributes estimated for this reach include bankfull width and depth, or the width and depth of the channel just before the water enters the floodplain, the width of the floodplain bench, and the sinuosity of the river. The estimated river geometry is based on Stantec Regional Hydraulic Geometry and Discharge equations for the State of Michigan. Figure 4-4 illustrates these estimated cross-section river attributes within the proposed channel.

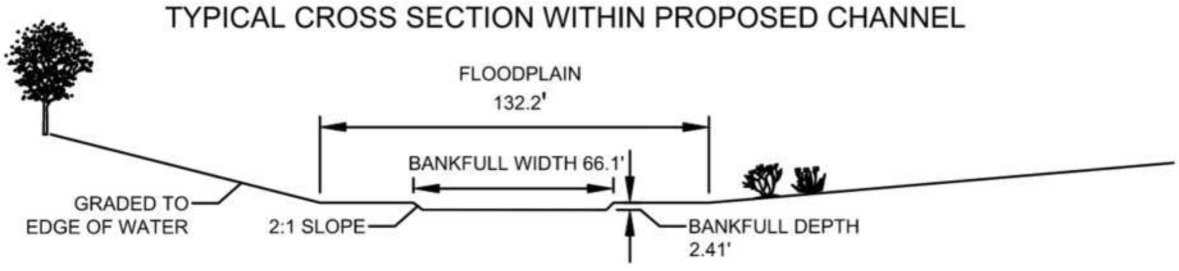


**Figure 4-4: Typical Cross Section Within Proposed Channel**

If the City moves forward with dam removal, the site-specific design will depend on river geometries gathered at an appropriate reference reach of the White River and verified through hydrologic and hydraulic analyses.

After stream channel restoration and floodplain establishment, the next step involves either active or passive river restoration. Active restoration includes installing bank stabilization measures, habitat structures, and a comprehensive seeding and planting plan to accelerate ecological recovery. Passive restoration allows natural processes to occur with limited human intervention. If passive restoration is considered, there are some areas of the project where engineered grade control or bank stabilization would be required to prevent harmful impacts from uncontrolled erosion or head cutting. These areas include establishing grade control at the upstream tie-in point and at the location of the existing dam where the soils have been significantly impacts by dam construction. Other areas to considered engineered stabilization would be along channel bends close to private property where bank erosion could harmfully impact adjacent property owners. While passive restoration can be more cost-effective, it may take longer to see desired results. Active restoration aims for quicker outcomes but may come with higher costs and environmental impacts. The choice between the two methods depends on project goals, site conditions, and stakeholder preferences.

An additional consideration for a dam removal scenario is the South State Street roadway. Currently the road sits on top of the dam. If the dam were to be removed a large culvert or spanning bridge would be required to maintain the road crossing. This structure would likely need to span at least the bankfull channel width to meet minimum permitting requirements. Given a 66-foot-width bankfull channel, a spanning bridge is most likely to be best suited to meet the needs of this project.

### *4.2.6 Other Benefits and Drawbacks*

In addition to the dam removal action items discussed above, Table 4-6 outlines other benefits and drawbacks of the dam removal alternative.

**Table 4-6: Other Benefits and Drawbacks of Dam Removal**

| **Dam Removal Alternative** | |
|---|---|
| **Benefit** | **Drawback** |
| - Improved condition of river ecosystem and surrounding natural resources.<br>- Possible parkland development opportunities for the City.<br>- Removing all future expenses associated with the Dam.<br>- Mitigating risk from the dam structure or a dam failure.<br>- Greater potential for outside funding opportunities to complete work. | - Absent outside funding, immediate upfront costs to rehabilitate portions of the dam may cost less than removal of the dam.<br>- Change in recreational use of impoundment.<br>- Not what the community currently desires. |

### *4.2.7 Cost and Funding*

The cost estimate for removing the Dam based on the conceptual design is $9.3M – $12.7M, passive and active restoration, respectively, which includes 30% contingency. A detailed cost breakdown is included within Appendix H. In this alternative, the Dam would be removed, and a natural river channel would replace it, eliminating the need for any future maintenance or repair.

**Table 4-7: Cost Comparison for Removing the Dam**

| **Cost Comparison** | | |
|---|---|---|
| | **Passive Restoration Approach** | **Active Restoration Approach** |
| Initial Repairs | $9.3M | $12.7M |
| Life Cycle Cost of Dam through 50 years in 2024 dollars (including removal costs) | $9.3M | $12.7M |
| Life Cycle Cost of Dam through 50 years in future spending (Based on 5% inflation rate) (including removal costs) | $9.3M | $12.7M |

Many of the available funding sources are based on a competitive pool of applicants where dam removal projects or other aquatic restoration projects are evaluated based on the amount of upstream habitat that is opened because of the removal. In the case of White Cloud Dam, the removal of the structure would open approximately 48 miles of the White River. Because the removal would connect such a large portion of the river, it would be highly competitive to receive funding from multiple grant opportunities. Additionally, with the increased focus on dam safety risk reduction, there are currently State and Federal funding sources for dam removal projects. Being a high hazard dam, dam removal at this site would score well with these grant programs. Generally, there is a larger source of grant funding opportunities and stakeholder groups that could potentially fund or assist with funding for dam removal than for dam rehabilitation or replacement. Any federal funding for the project would also require cultural resources review and State Historic Preservation Office (SHPO) Section 106 permit.

Appendix J includes a spreadsheet of known potential funding sources that could aid the City in funding the removal activities.

# 5. Summary

The White Cloud Dam is an aging piece of infrastructure. Multiple deficiencies and additional data needs have been presented in this report. It is important that the City and its community assess both engineering and non-engineering factors when selecting the most suitable alternative for the future of the Dam. Some of these factors include:

- Initial cost of repairs and removal,
- Life cycle cost of maintenance and upkeep,
- Potential funding opportunities for each alternative,
- Risk liability of the dam,
- Community and local organization interest in maintaining or removing the dam, and
- Future use of the impoundment or floodplain after dam rehabilitation or river restoration.

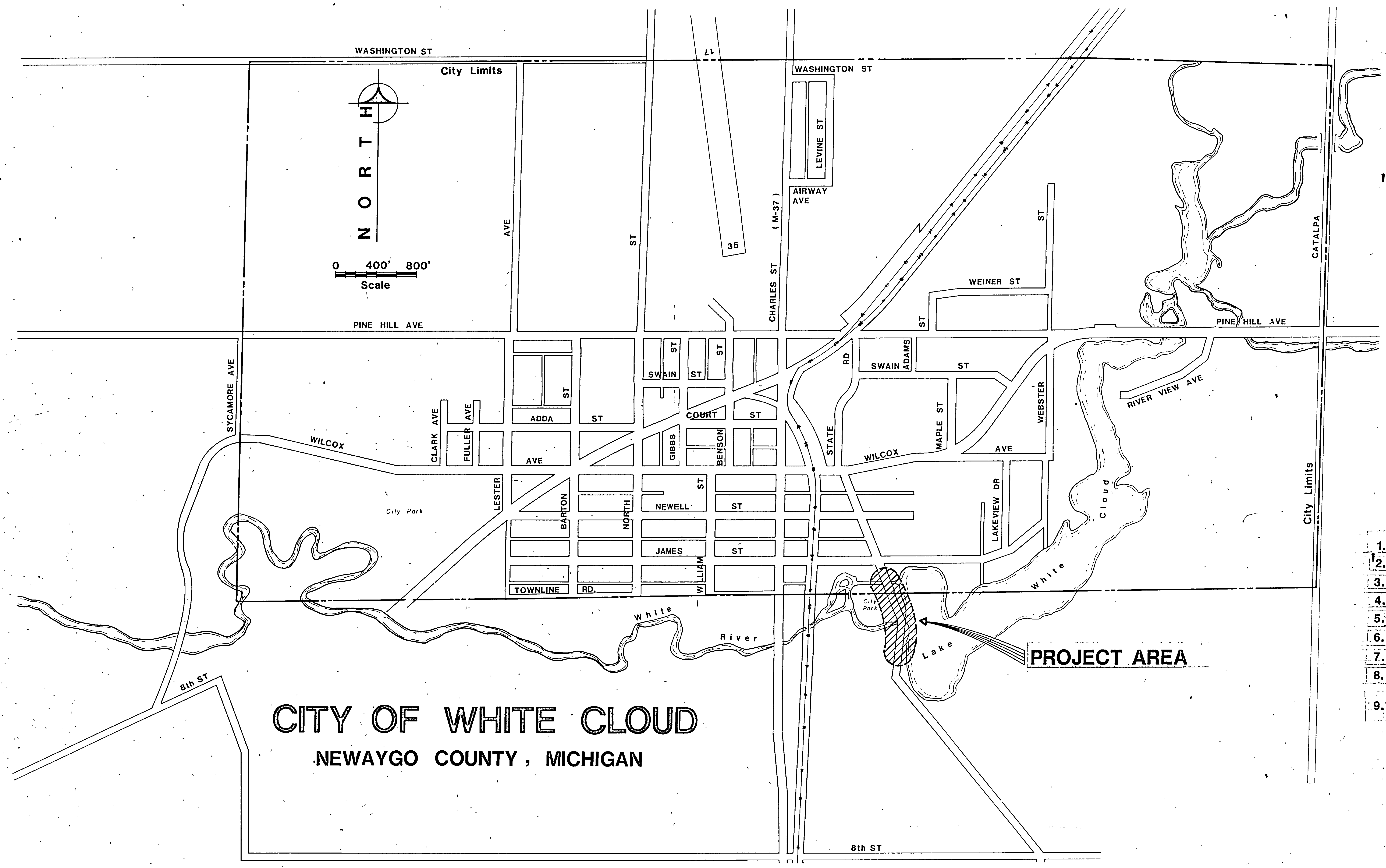
Assessing these factors, along with others identified in this report and by the City will aid in determining the most appropriate alternative for the City of White Cloud. Table 5-1 summarizes the cost comparison of the three alternatives, as cost typically plays a significant role in the decision-making process.

## Table 5-1: Dam Alternative Cost Comparison

| Cost Comparisons | | | |
|---|---|---|---|
| **Dam Alternative** | **Initial Cost** | **Life Cycle Cost through 50 years in 2024 dollars (including initial costs)** | **Life Cycle Cost through 50 years in future spending (Based on 5% inflation rate) (including initial costs)** |
| Alternative 1 – Dam Rehabilitation | $8.5M – $10.7M | $11.2M – 13.4M | $33.8M - $41.6M |
| Alternative 2 – Dam Rehabilitation with Future Regulations | $8.5M – $10.7M | $16.4M - $18.6M | $47.9M – $57.8M |
| Alternative 3 – Dam Removal | $9.3M - $12.7M | $9.3M - $12.7M | $9.3M - $12.7M |

# Appendix A – 1989 OMM Drawings

# WHITE CLOUD DAM RECONSTRUCTION

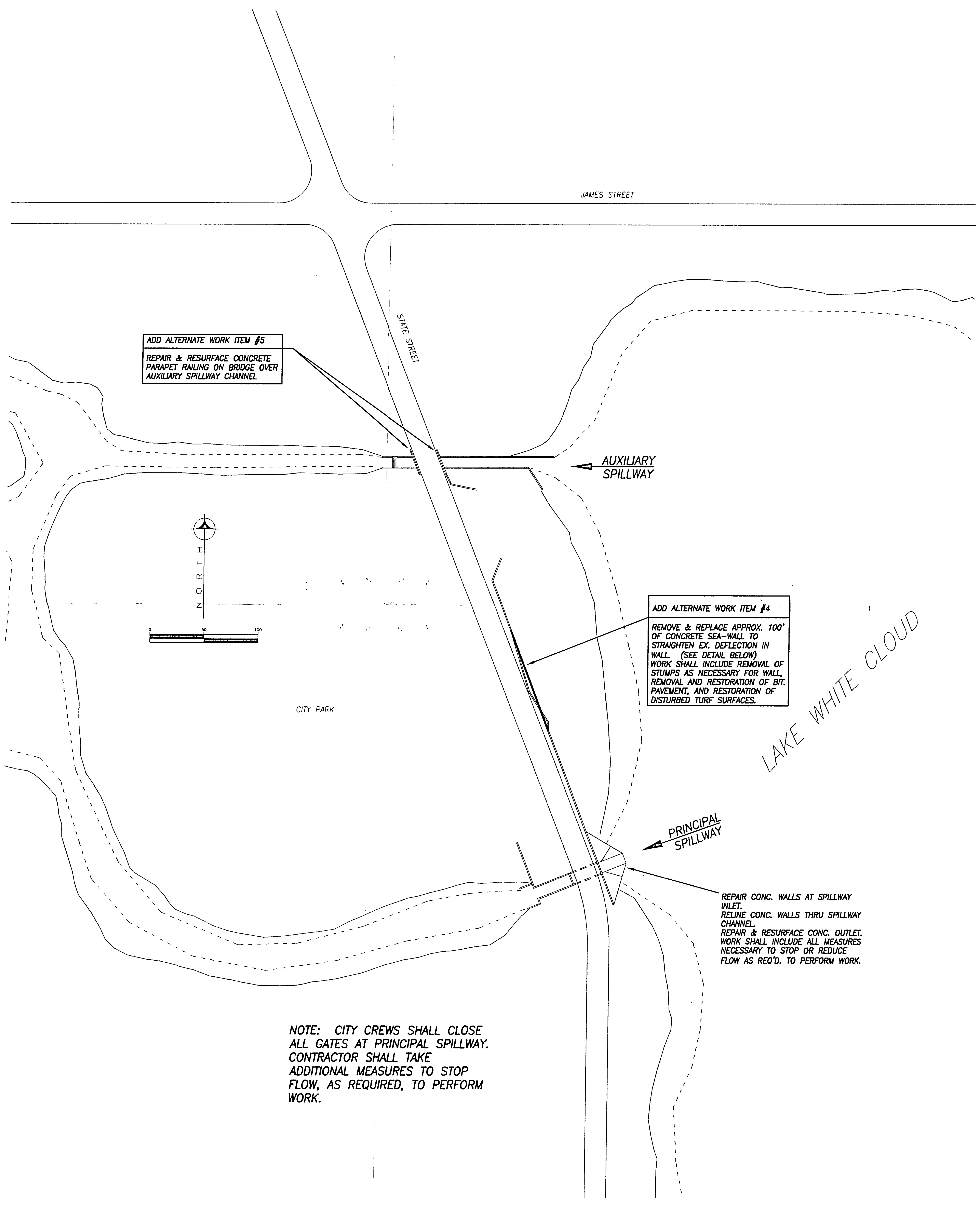


## CITY OF WHITE CLOUD

### NEWAYGO COUNTY, MICHIGAN

**SHEET INDEX**

*NOTE: Revised December, 1989 to reflect DNR review comments.*



ADD ALTERNATE WORK ITEM #5

REPAIR & RESURFACE CONCRETE PARAPET RAILING ON BRIDGE OVER AUXILIARY SPILLWAY CHANNEL

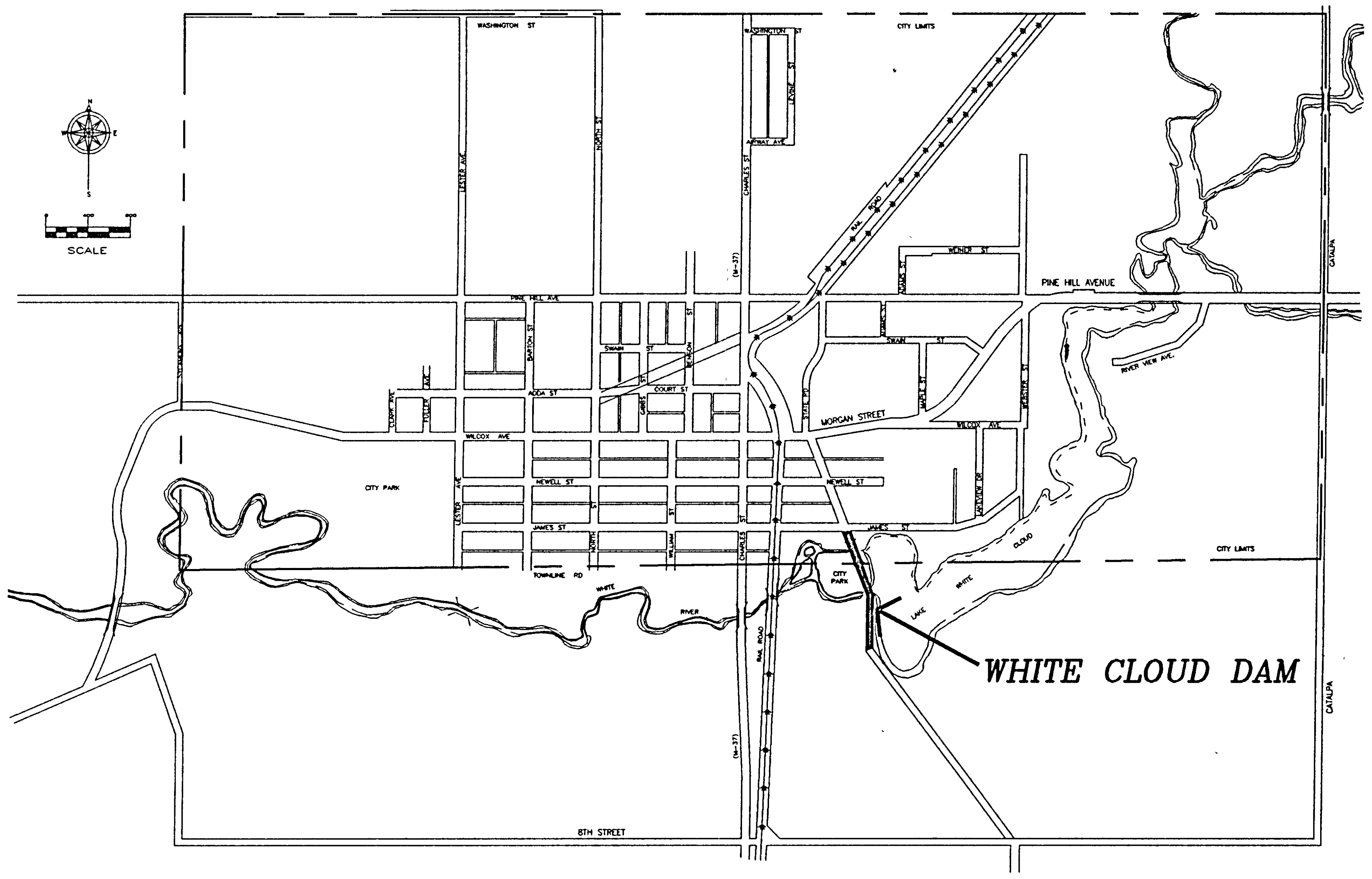
ADD ALTERNATE WORK ITEM #4

REMOVE & REPLACE APPROX. 100' OF CONCRETE SEA-WALL TO STRAIGHTEN EX. DEFLECTION IN WALL. (SEE DETAIL BELOW) WORK SHALL INCLUDE REMOVAL OF STUMPS AS NECESSARY FOR WALL, REMOVAL AND RESTORATION OF BIT. PAVEMENT, AND RESTORATION OF DISTURBED TURF SURFACES.

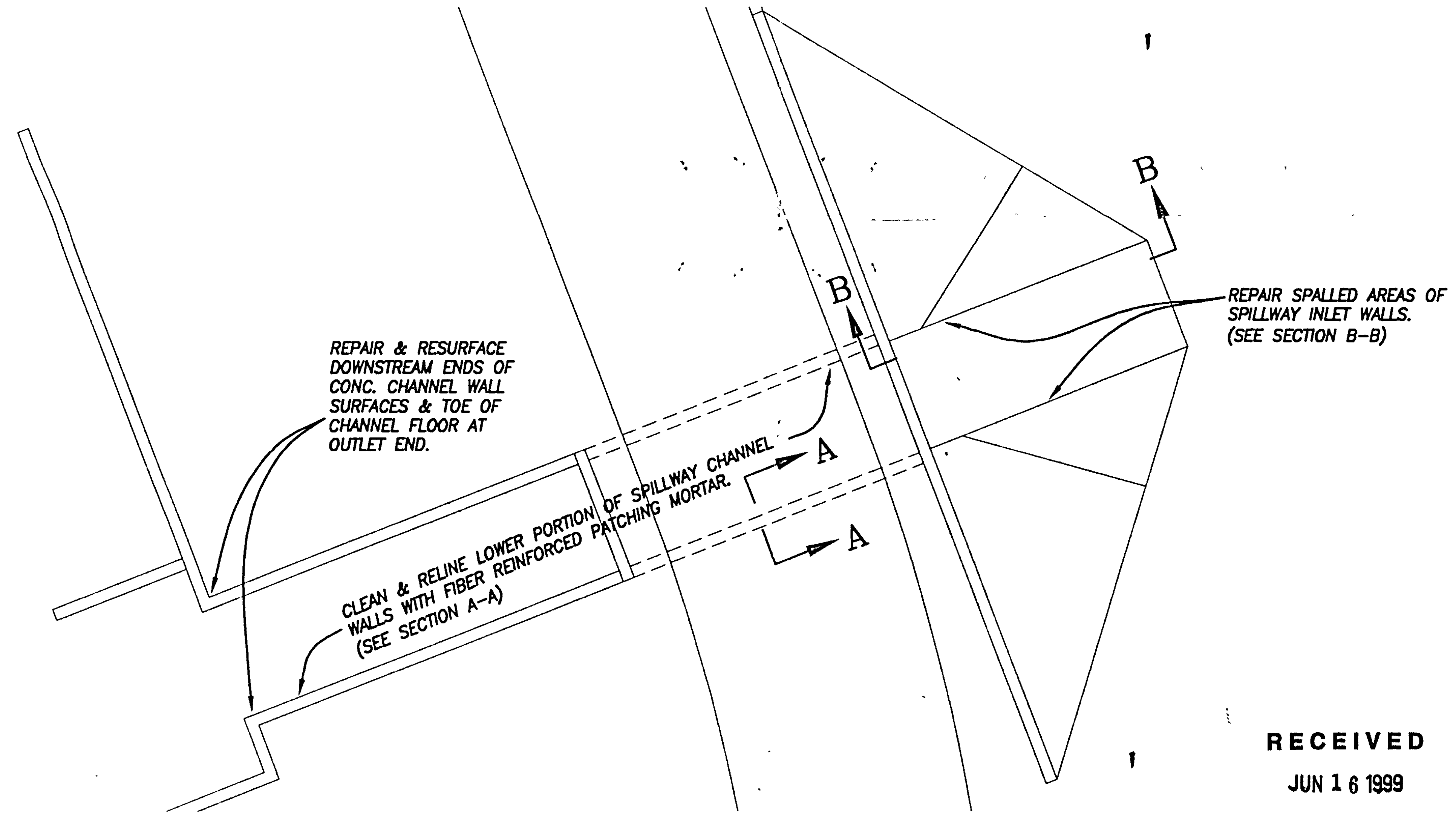
REPAIR CONC. WALLS AT SPILLWAY INLET.
RELINE CONC. WALLS THRU SPILLWAY CHANNEL.
REPAIR & RESURFACE CONC. OUTLET.
WORK SHALL INCLUDE ALL MEASURES NECESSARY TO STOP OR REDUCE FLOW AS REQ'D. TO PERFORM WORK.

NOTE: CITY CREWS SHALL CLOSE ALL GATES AT PRINCIPAL SPILLWAY. CONTRACTOR SHALL TAKE ADDITIONAL MEASURES TO STOP FLOW, AS REQUIRED, TO PERFORM WORK.

SITE PLAN



LOCATION MAP



REPAIR & RESURFACE DOWNSTREAM ENDS OF CONC. CHANNEL WALL SURFACES & TOE OF CHANNEL FLOOR AT OUTLET END.

CLEAN & RELINE LOWER PORTION OF SPILLWAY CHANNEL WALLS WITH FIBER REINFORCED PATCHING MORTAR. (SEE SECTION A-A)

REPAIR SPALLED AREAS OF SPILLWAY INLET WALLS. (SEE SECTION B-B)

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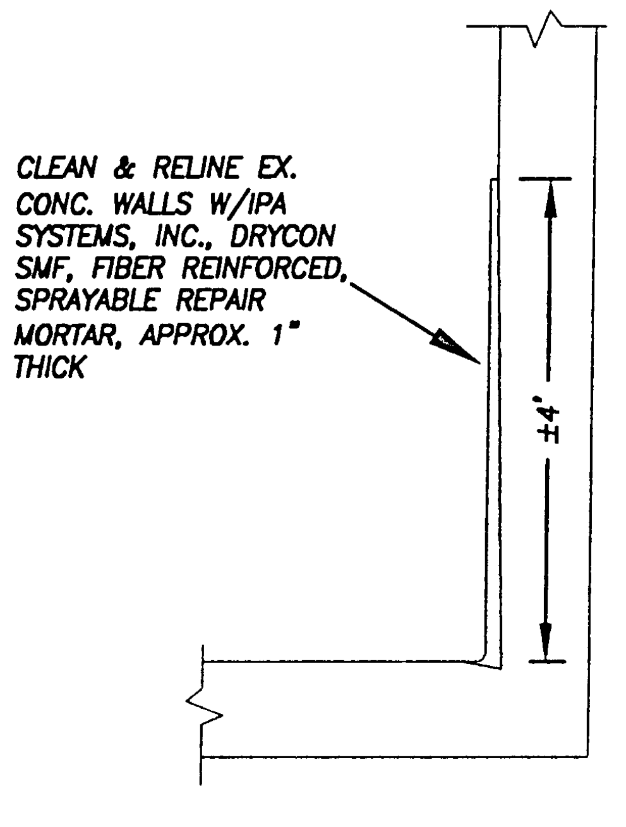
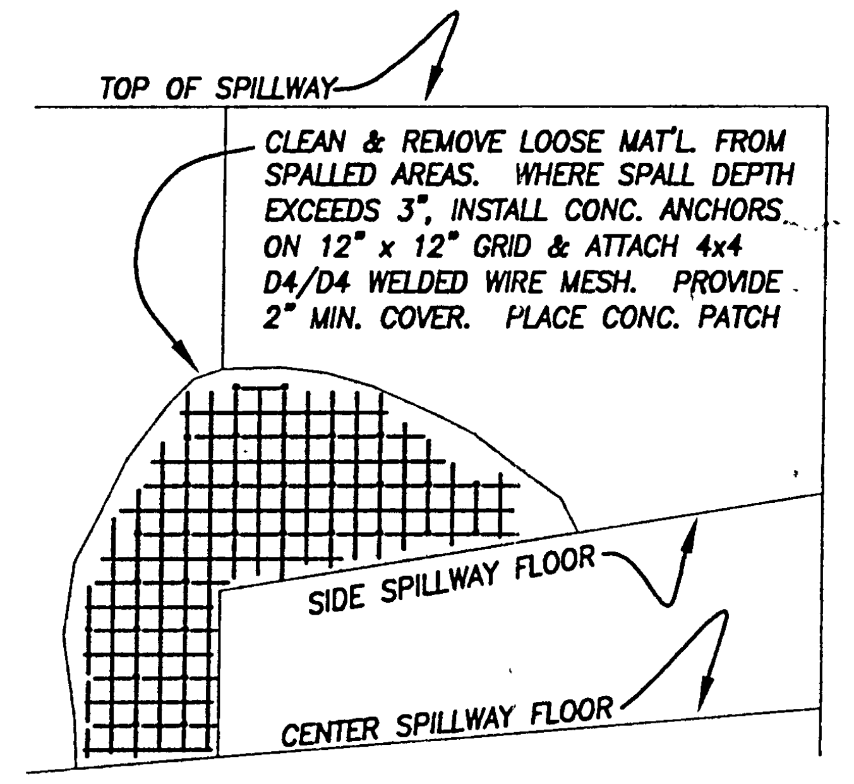
JUN 16 1999
ENVIRONMENTAL QUALITY
LAND & WATER MANAGEMENT

PRINCIPAL SPILLWAY DETAIL
Scale 1" = 10'



CLEAN & RELINE EX. CONC. WALLS W/IPA SYSTEMS, INC., DRYCON SMF, FIBER REINFORCED, SPRAYABLE REPAIR MORTAR, APPROX. 1" THICK

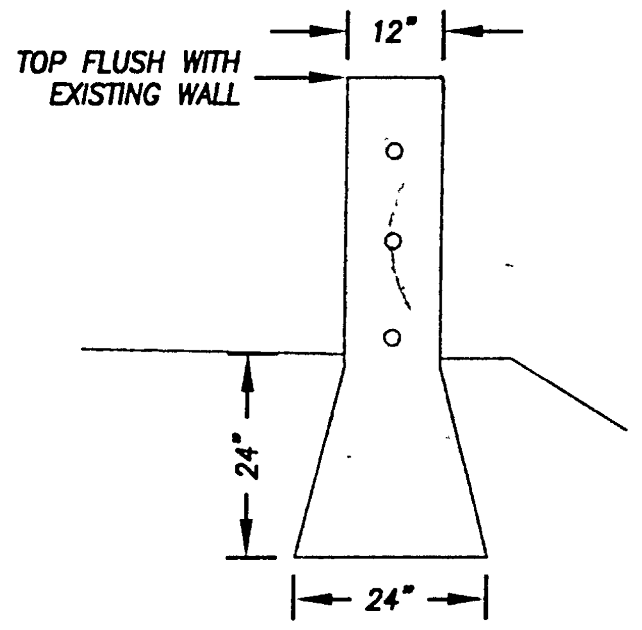
SECTION A-A
No Scale



CLEAN & REMOVE LOOSE MAT'L. FROM SPALLED AREAS. WHERE SPALL DEPTH EXCEEDS 3", INSTALL CONC. ANCHORS ON 12" x 12" GRID & ATTACH 4x4 D4/D4 WELDED WIRE MESH. PROVIDE 2" MIN. COVER. PLACE CONC. PATCH

SECTION B-B
No Scale

INSTALL BITUMINIZED FIBER JOINT FILLER AT EACH END & ANCHOR TO EX. WALL W/(3) ½" STAINLESS STEEL CONC. ANCHORS



SEA-WALL DETAIL
No Scale

omm engineering, inc.
civil engineers/surveyors
1680 east paris, s.e.
grand rapids, mi. 49546
616-957-4350

| | |
|---|---|
| sewer permit number | - |
| water permit number | - |
| approved by | |
| name | date |
| revised in accordance w/construction records | by - date - |

| revisions | by | date |
|---|---|---|
| | - | - |
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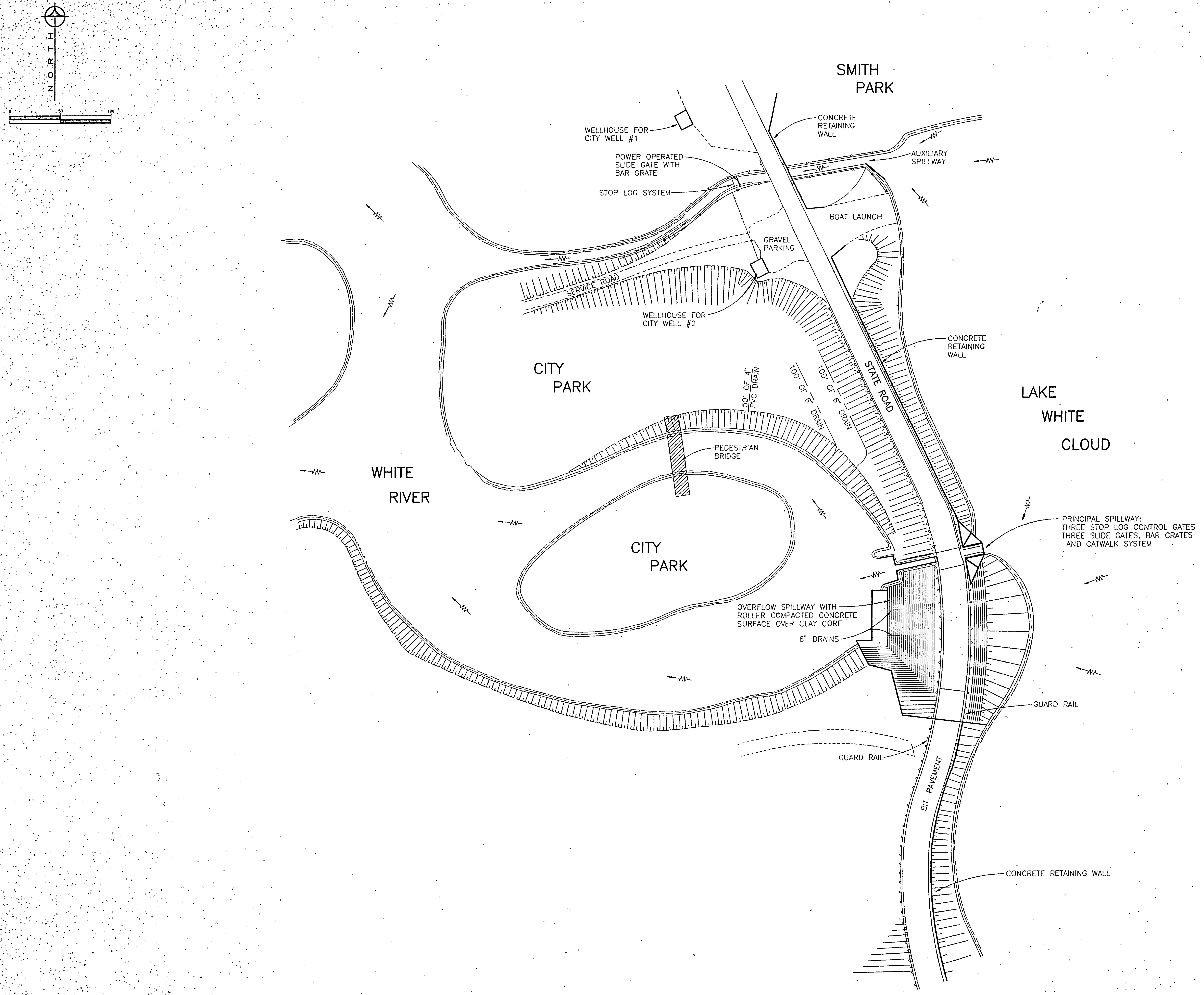
# WHITE CLOUD DAM CONCRETE REHABILITATION

CITY OF WHITE CLOUD
Newaygo County, Michigan

| | |
|---|---|
| drawn by DGM | date 4/99 |
| checked by LAM | date 4/99 |
| project number 3213 | |
| sheet number 1 of 1 | |

DRAWINGS\3213\3215base.dwg 5/03/99

WHITE CLOUD DAM



omm engineering, inc.
civil engineers/surveyors
1550 east beltline, s.e.
grand rapids, mi. 49506
616-957-4350

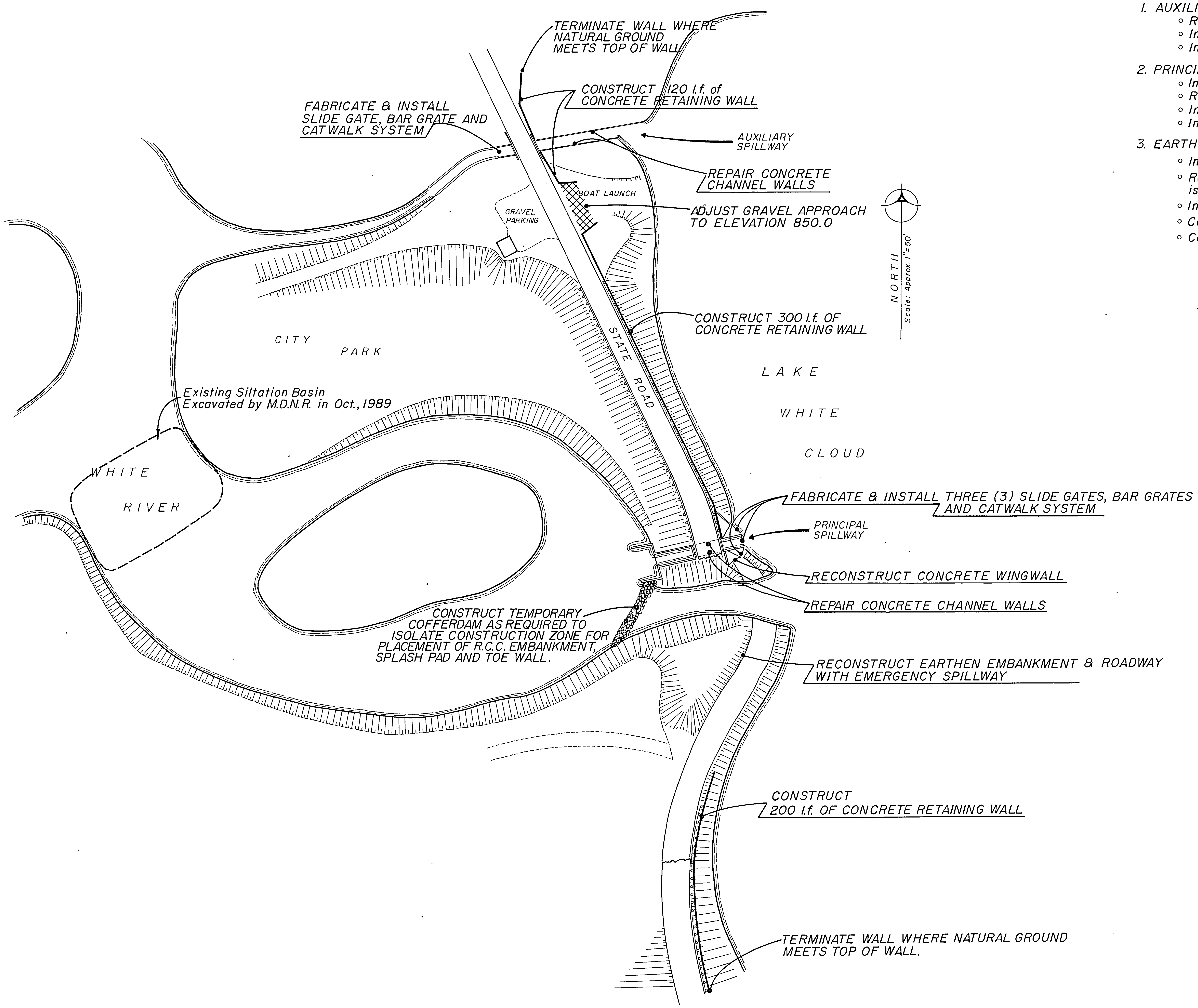
sewer permit number —
water permit number —
approved by
name
date
revised in accordance w/construction records by — date —

revisions by date

CITY OF WHITE CLOUD
NEWAYGO COUNTY, MICHIGAN
CITY PARK
AT
LAKE WHITE CLOUD

drawn by RAG date 9/96
checked by LAM date 9/96
project number 01301
sheet number 1 of 1

1301PARK 10/03/96



## CONSTRUCTION PRIORITY SCHEDULE

1. AUXILIARY SPILLWAY AREA
   - Repair Concrete Channel Walls
   - Install Slide Gate & Bar Grate
   - Install Catwalk
2. PRINCIPAL SPILLWAY AREA
   - Install Temporary Sheeting
   - Rebuild South Wingwall
   - Install Slide Gates & Bar Grates
   - Install Catwalk System
3. EARTHEN EMBANKMENT AREA
   - Install Temporary Culverts
   - Regulate Flow Through Culverts Until Flow is Diverted to Auxiliary Spillway
   - Install Cofferdam
   - Construct Emergency Spillway
   - Construct Retaining Walls

olson, meyers & may, inc.
civil engineers
1550 e. beltline se
grand rapids, mi. 49506
616-957-4350

| revisions | by | date |
|---|---|---|
| Revised December, 1989 to reflect D.N.R. review comments | EJ | 12-89 |

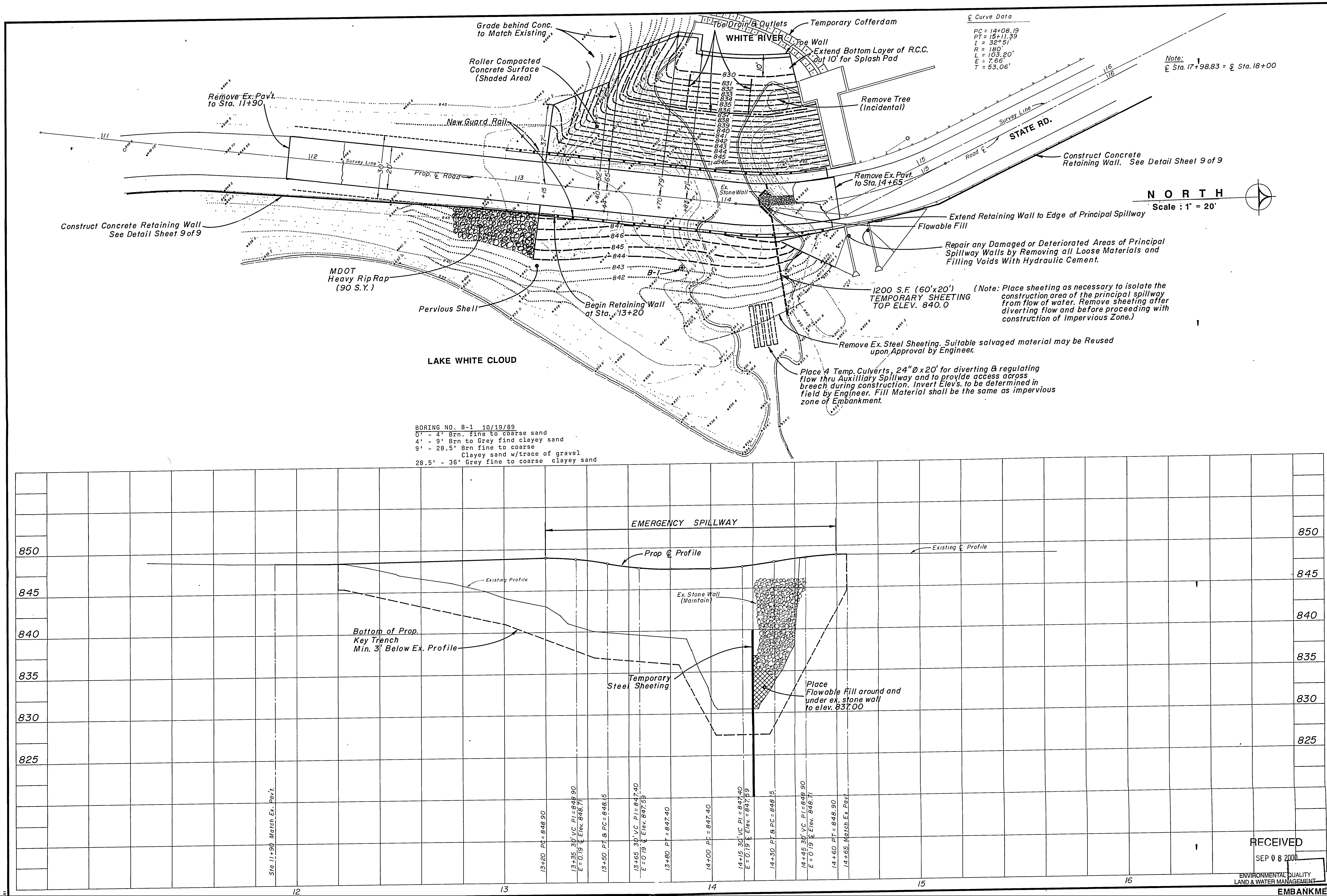
sewer permit number
water permit number
approved by
name
date
revised in accordance w/construction records by date

**CITY OF WHITE CLOUD**
**Newaygo County, Michigan**

**WHITE CLOUD DAM RECONSTRUCTION**

drawn by E.J. date 8-87
checked by D.G.M. date 11-89
project number 01355
sheet number 2 of 9

SITE PLAN



Grade behind Conc. to Match Existing

Roller Compacted Concrete Surface (Shaded Area)

Remove Ex. Pav't. to Sta. 11+90

New Guard Rail

Toe Drain & Outlets

Temporary Cofferdam

Toe Wall

Extend Bottom Layer of R.C.C. out 10' for Splash Pad

Remove Tree (Incidental)

Construct Concrete Retaining Wall. See Detail Sheet 9 of 9

Remove Ex. Pavt. to Sta. 14+65

Extend Retaining Wall to Edge of Principal Spillway

Flowable Fill

Repair any Damaged or Deteriorated Areas of Principal Spillway Walls by Removing all Loose Materials and Filling Voids With Hydraulic Cement.

Construct Concrete Retaining Wall See Detail Sheet 9 of 9

MDOT Heavy Rip Rap (90 S.Y.)

Pervious Shell

Begin Retaining Wall at Sta. 13+20

1200 S.F. (60'x20') TEMPORARY SHEETING TOP ELEV. 840.0

(Note: Place sheeting as necessary to isolate the construction area of the principal spillway from flow of water. Remove sheeting after diverting flow and before proceeding with construction of Impervious Zone.)

Remove Ex. Steel Sheeting. Suitable salvaged material may be Reused upon Approval by Engineer.

Place 4 Temp. Culverts, 24"Ø x 20' for diverting & regulating flow thru Auxilliary Spillway and to provide access across breech during construction. Invert Elev's. to be determined in field by Engineer. Fill Material shall be the same as impervious zone of Embankment.

℄ Curve Data

PC = 14+08.19
PT = 15+11.39
I = 32°51'
R = 180'
L = 103.20'
E = 7.66'
T = 53.06'

Note:
℄ Sta. 17+98.83 = ℄ Sta. 18+00

BORING NO. B-1 10/19/89
0' - 4' Brn. fine to coarse sand
4' - 9' Brn to Grey find clayey sand
9' - 28.5' Brn fine to coarse
Clayey sand w/trace of gravel
28.5' - 36' Grey fine to coarse clayey sand

Bottom of Prop. Key Trench Min. 3' Below Ex. Profile

Temporary Steel Sheeting

Place Flowable Fill around and under ex. stone wall to elev. 837.00

olson, meyers & may, inc.
civil engineers
1550 e. beltline se
grand rapids, mi. 49506
616-957-4350

sewer permit number
water permit number
approved by
name
date
revised in accordance w/construction records
by ___ date ___

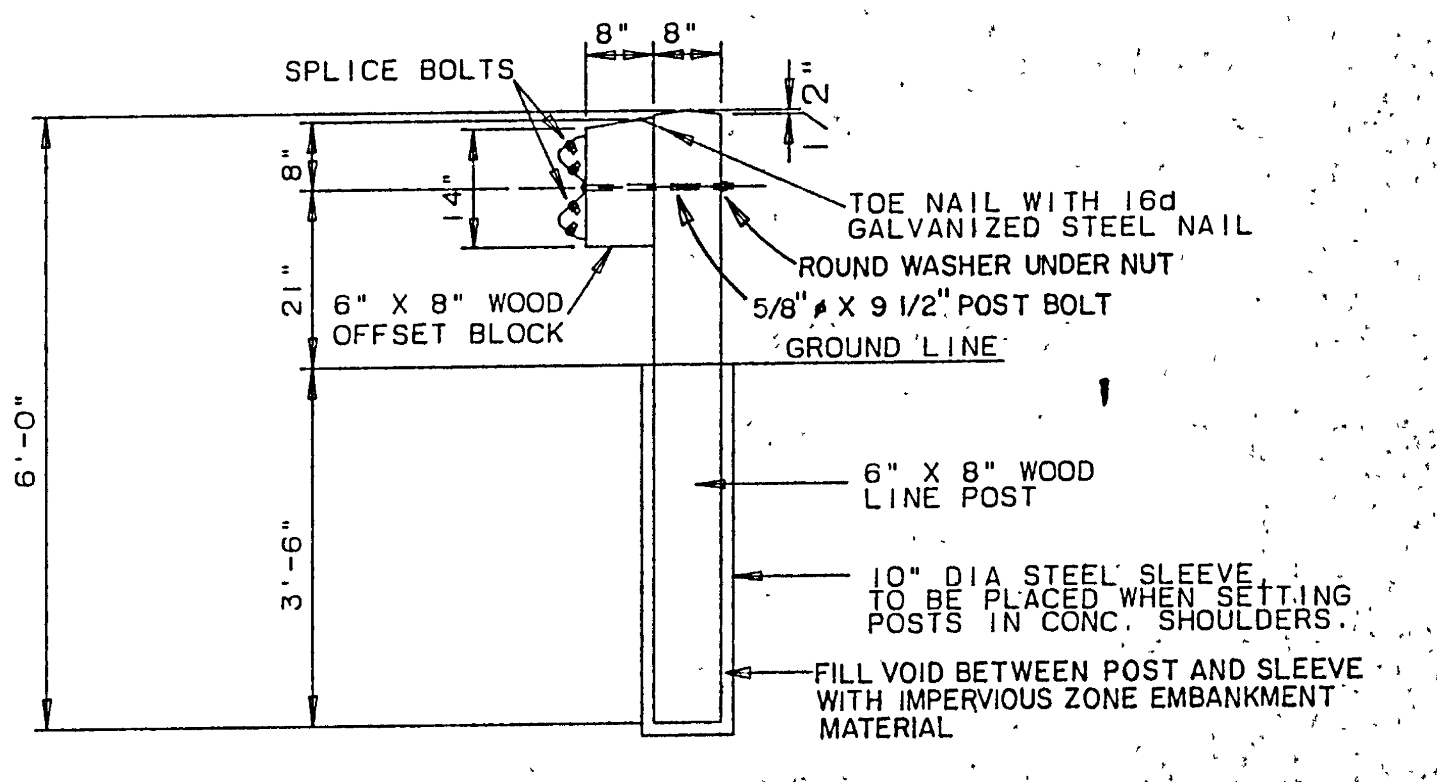
| revisions | by | date |
|---|---|---|
| Revised December, 1989 to reflect DNR review comments. | E J | 12-89 |

CITY OF WHITE CLOUD
Newaygo County, Michigan

WHITE CLOUD DAM
RECONSTRUCTION

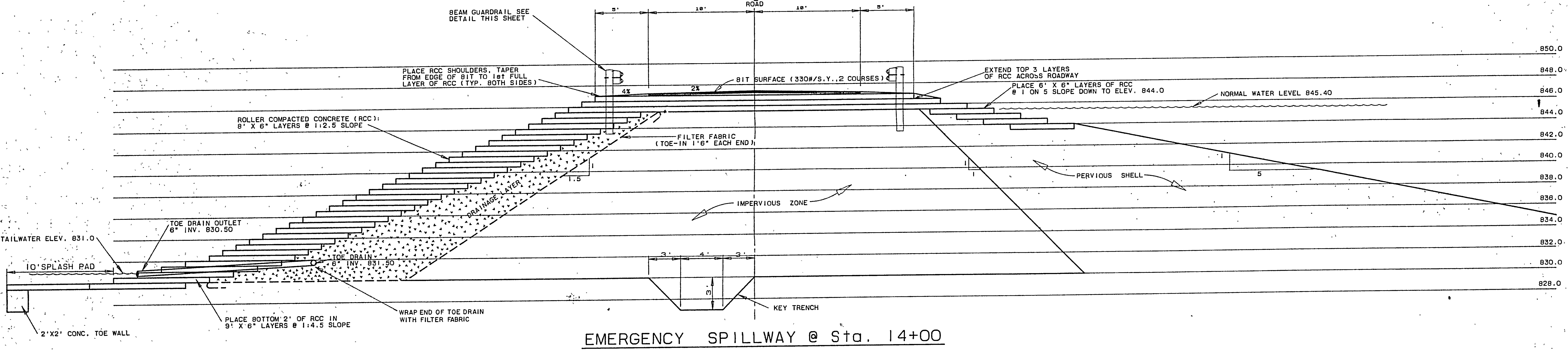
drawn by E.J. date 10-89
checked by D.G.M. date 11-89
project number 01355
sheet number 3 of 9

EMBANKMENT RECONSTRUCTION



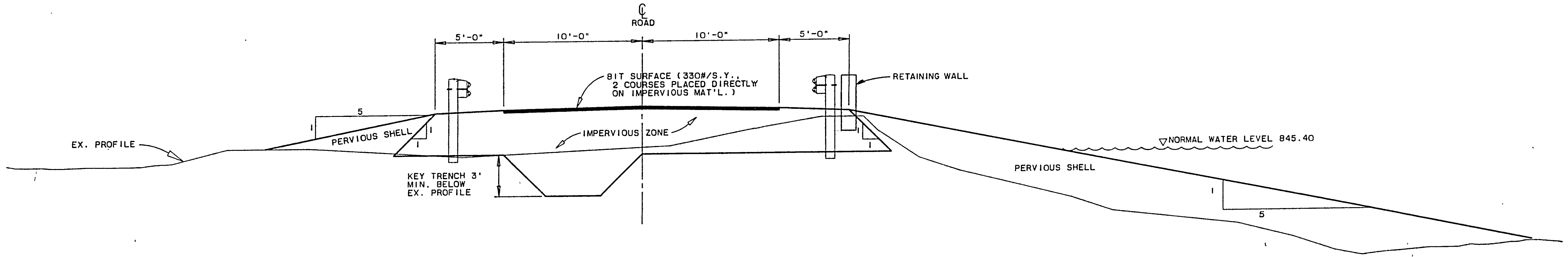
## TYPICAL MDOT - TYPE B BEAM GUARDRAIL 8 POST

TYPICAL POST SPACING = 6'-3"
NO SCALE



## EMERGENCY SPILLWAY @ Sta. 14+00

TYPICAL SECTION THRU EMBANKMENT
SCALE: 1" = 5'



## TYPICAL SECTION THRU EMBANKMENT @ Sta. 13+00

OUTSIDE OF EMERGENCY SPILLWAY
SCALE 1" = 5'

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civil engineers/surveyors
1550 e. beltline se
grand rapids, mi. 49506
616-957-4350

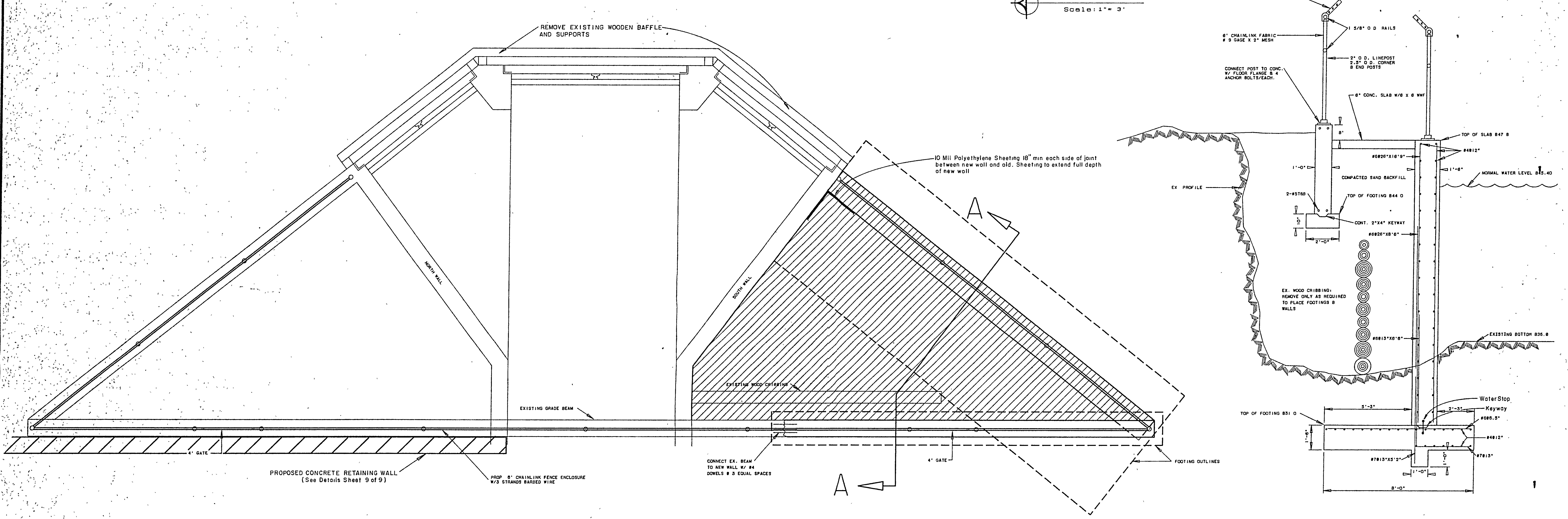
| | | | |
|---|---|---|---|
| Revised December, 1989 to reflect DNR review comments | E.J. | 12-89 | sewer permit number |
| | | | water permit number |
| | | | approved by |
| | | | name ______ date |
| revisions | by | date | revised in accordance by ______ w/construction records date ______ |

CITY OF WHITE CLOUD
Newaygo County, Michigan

# WHITE CLOUD DAM RECONSTRUCTION

drawn by date
M C S 10/31/89
checked by date
D G M 10/31/89
project number
1355
sheet number
4 of 9

EMBANKMENT CROSS-SECTIONS



# SECTION A-A

Notes: ① Existing fence and handrails to be removed. Remnants imbedded in existing concrete shall be cut off at grade.

② Existing concrete surfaces shall be brushed and all loose material removed. Spalled or broken areas shall be repaired with an epoxy based grout to provide a surface adequate for mounting all accessories ( Gates, Catwalks, Fences, Etc.)

▨ NEW CONCRETE CONSTRUCTION

olson meyers & may, inc.
civil engineers/surveyors
1550 e. beltline se
grand rapids, mi. 49506
616-957-4350

| | | |
|---|---|---|
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| revisions | by | date |

sewer permit number
water permit number
approved by
name date
revised in accordance by
w/construction records date

CITY OF WHITE CLOUD
Newaygo County, Michigan

WHITE CLOUD DAM
RECONSTRUCTION

drawn by date
M C S 11/06/89
checked by date
J L R 11/06/89
project number
1355
sheet number
5 of 9

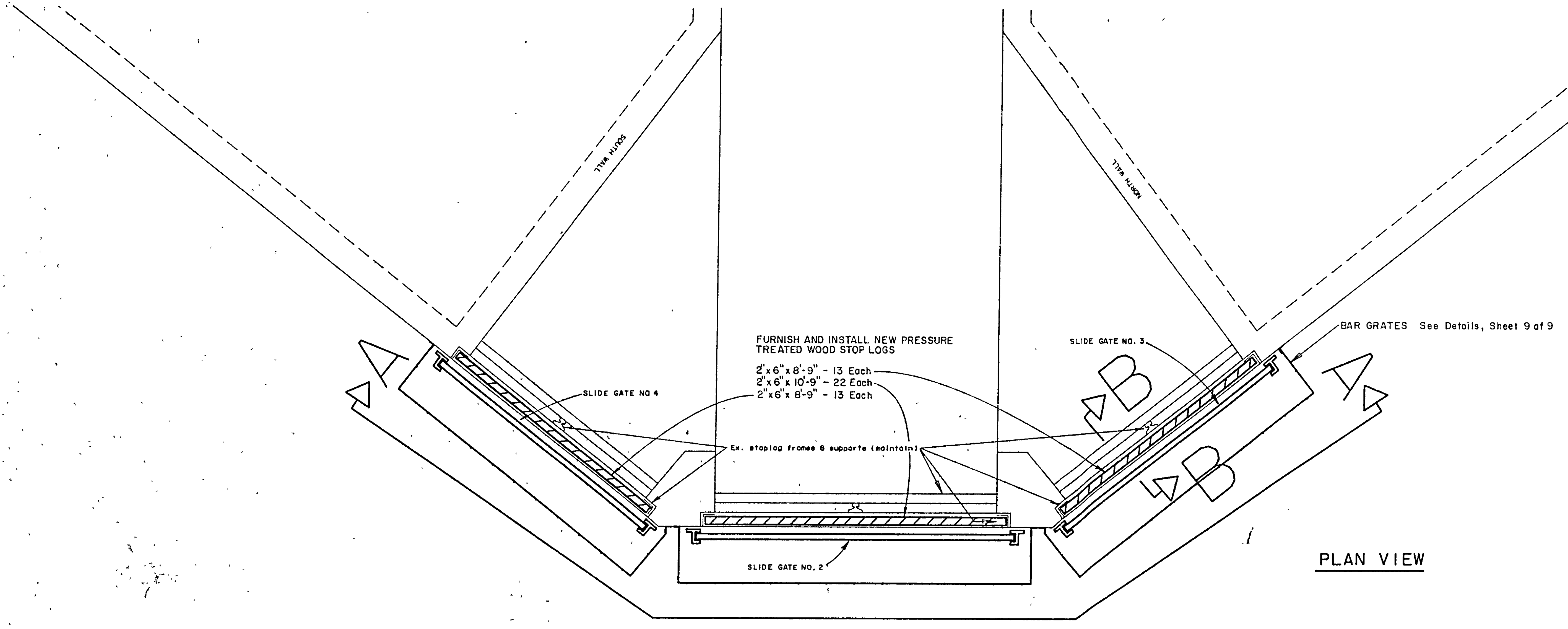
FENCING AND WINGWALL REPAIR: PRINCIPAL SPILLWAY

Notes:

① Slide Gate, Frame 8 Hoist shown for the purpose of illustrating location, required opening dimensions and water levels. Actual details of member sizes, fabrication and installation to be supplied by manufacturer

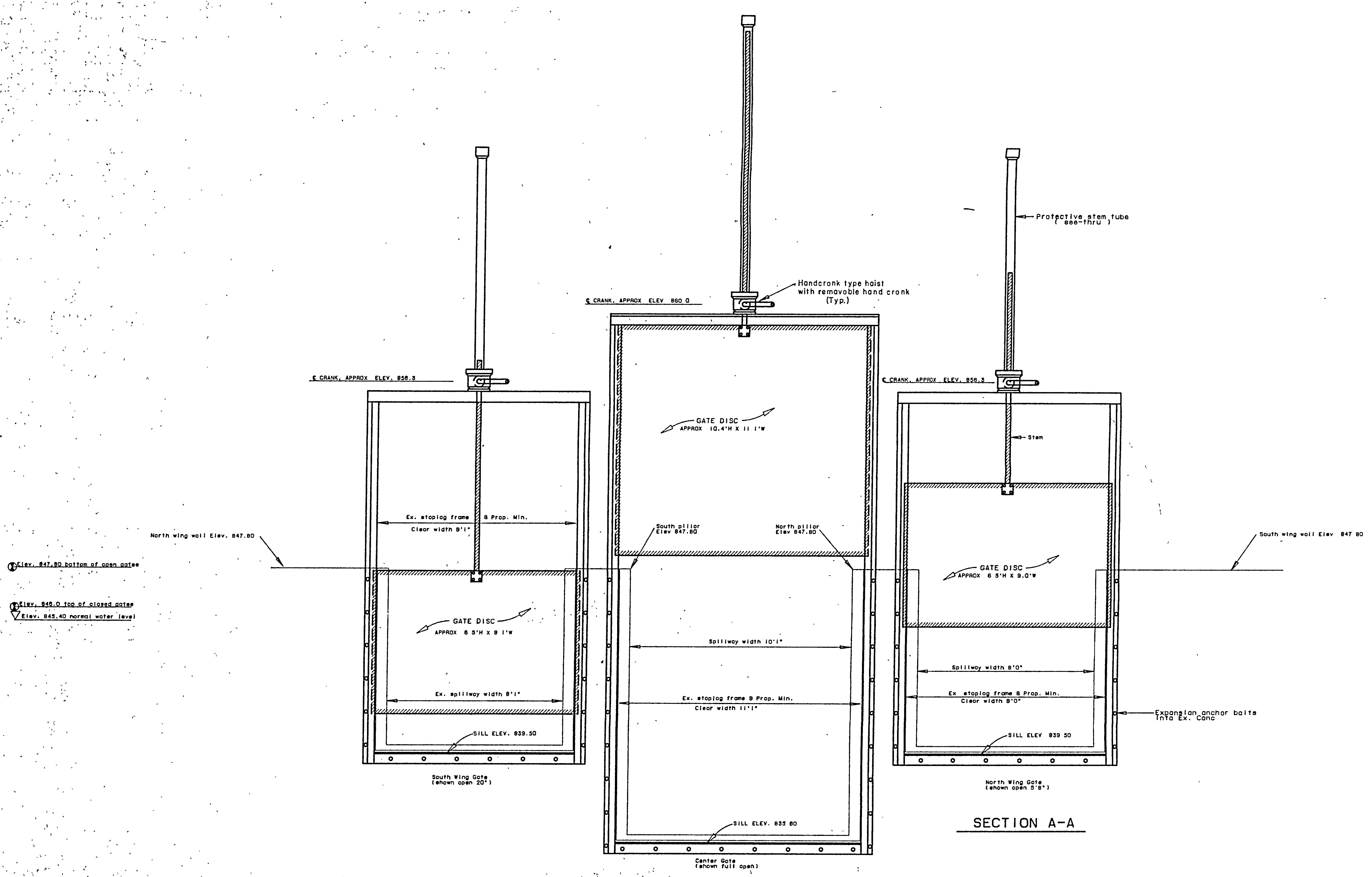
② CONTRACTOR shall make field measurements as may be required by slide gate manufacturer to establish exact dimensions necessary for gate fabrication

③ Lumber for Stop Logs shall be pressure treated to meet A W.P B standard LP-22 for ground contact ( 0.40 #/cf Retention)

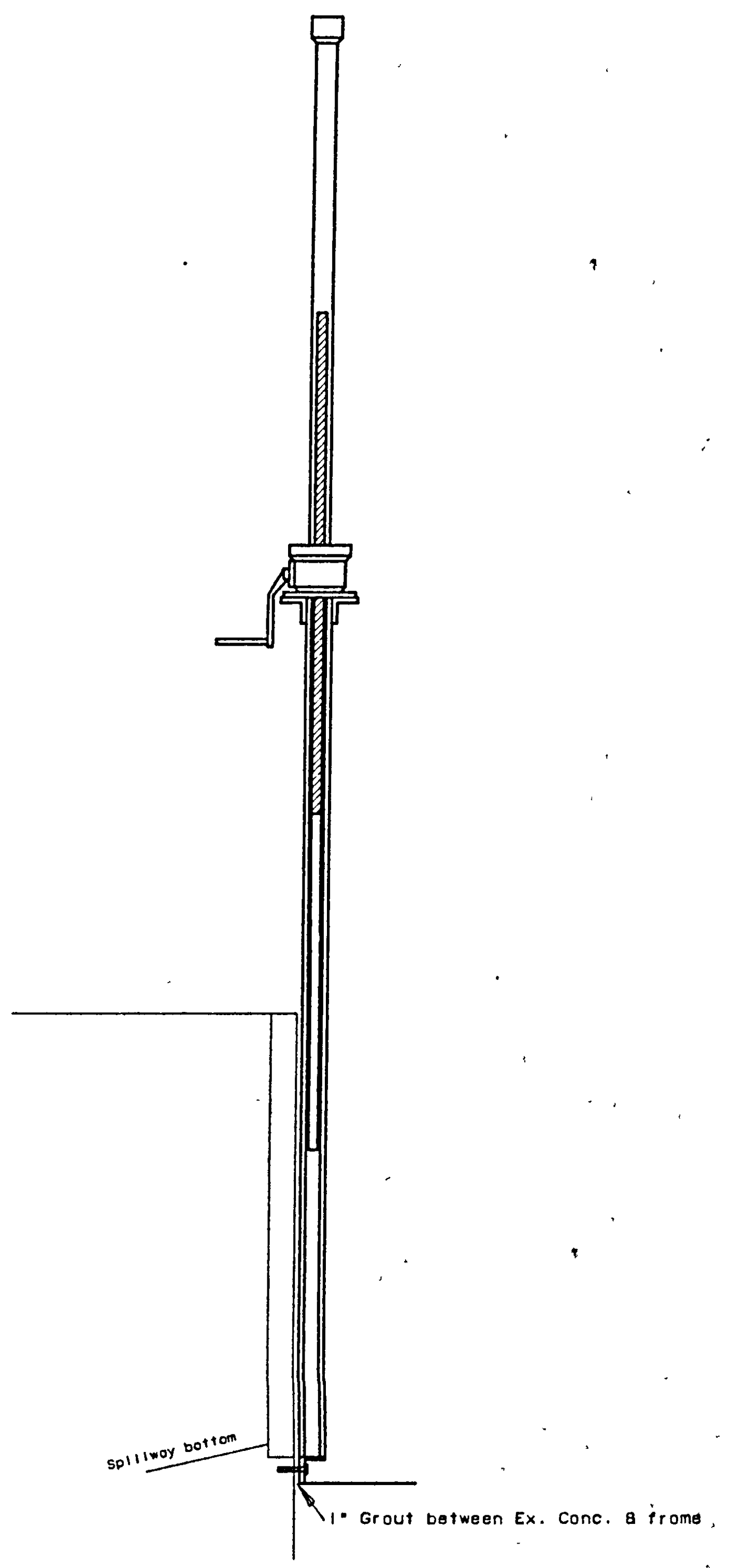


PLAN VIEW

NORTH
Scale:1"= 3'



SECTION A-A



SECTION B-B

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olson meyers & may, inc.
civil engineers/surveyors
1550 e. beltline se
grand rapids, mi. 49506
616-957-4350

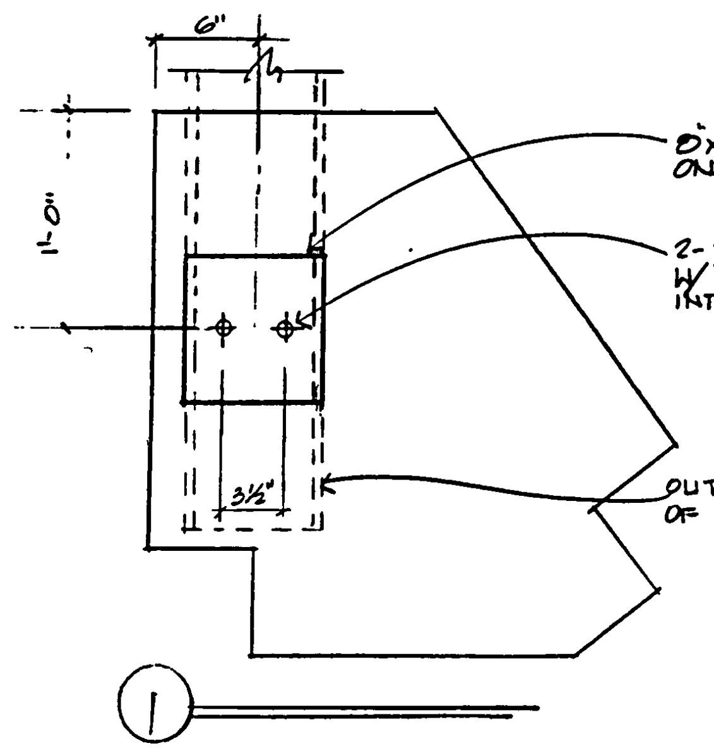
| | | | |
|---|---|---|---|
| Revised December 1989 to reflect MDNR review comments | E.J | 12-89 | sewer permit number |
| Add Stop Logs | E.J | 1-90 | water permit number |
| | | | approved by |
| | | | name date |
| revisions | by | date | revised in accordance by w/construction records date |

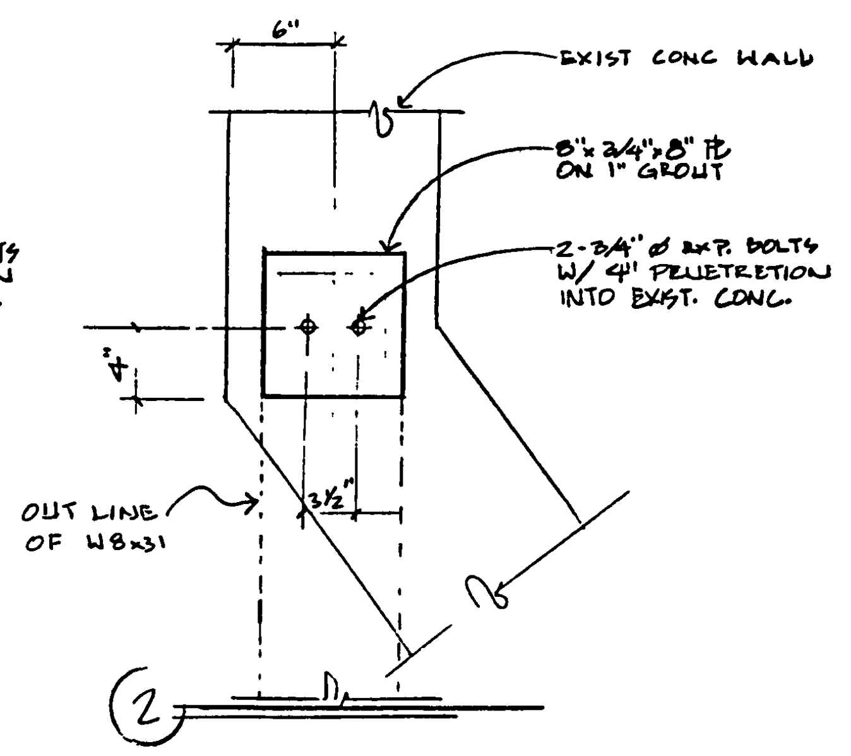
CITY OF WHITE CLOUD
Newaygo County, Michigan

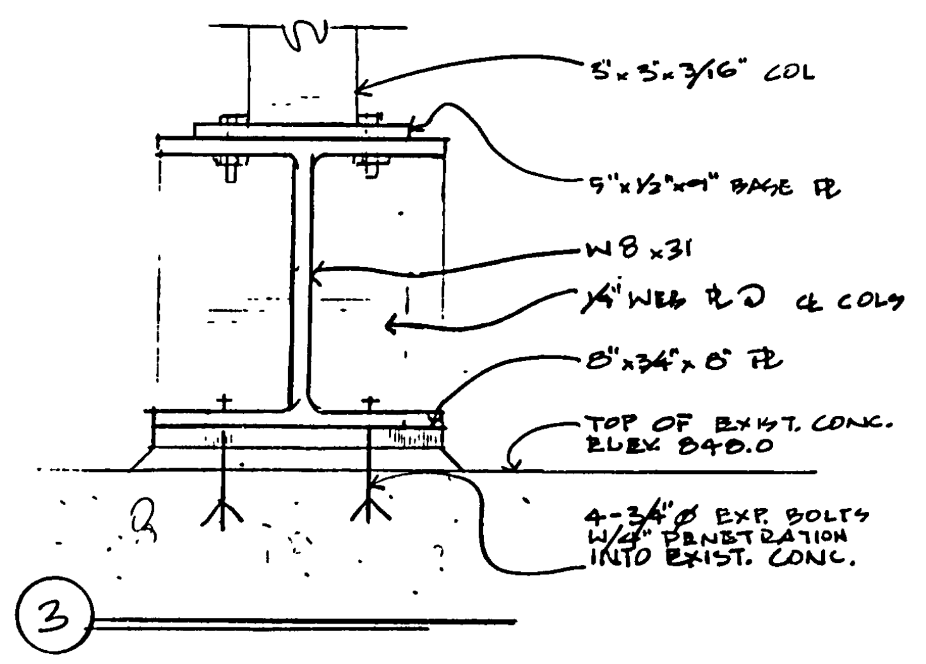
WHITE CLOUD DAM
RECONSTRUCTION

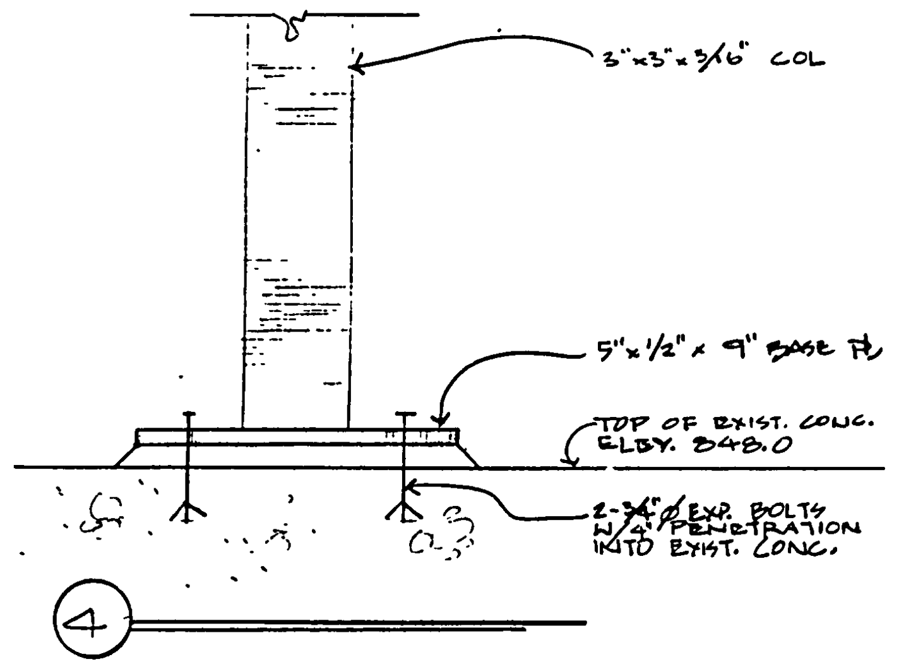
drawn by date
M C S 11/06/89
checked by date
D G M 11/06/89
project number
1355
sheet number
6 of 9

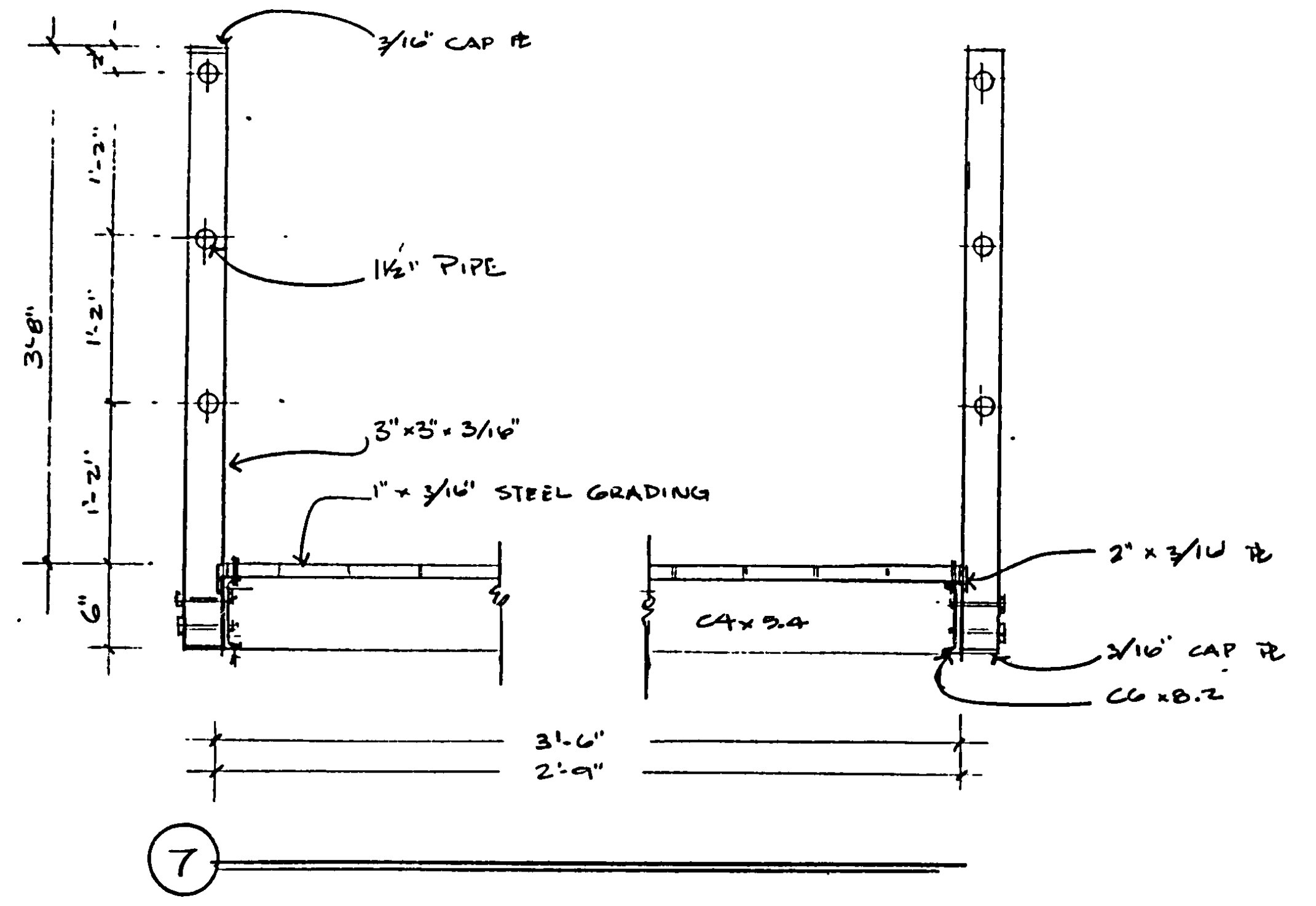
SLIDE GATES: PRINCIPAL SPILLWAY

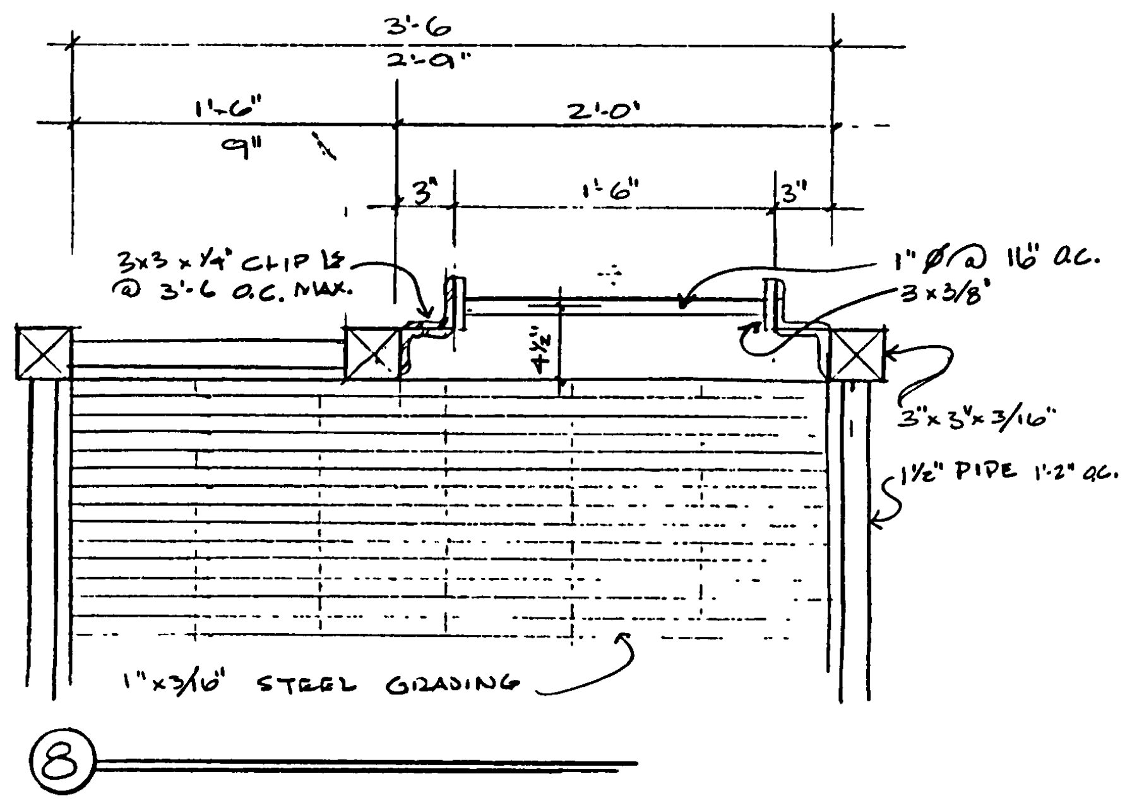


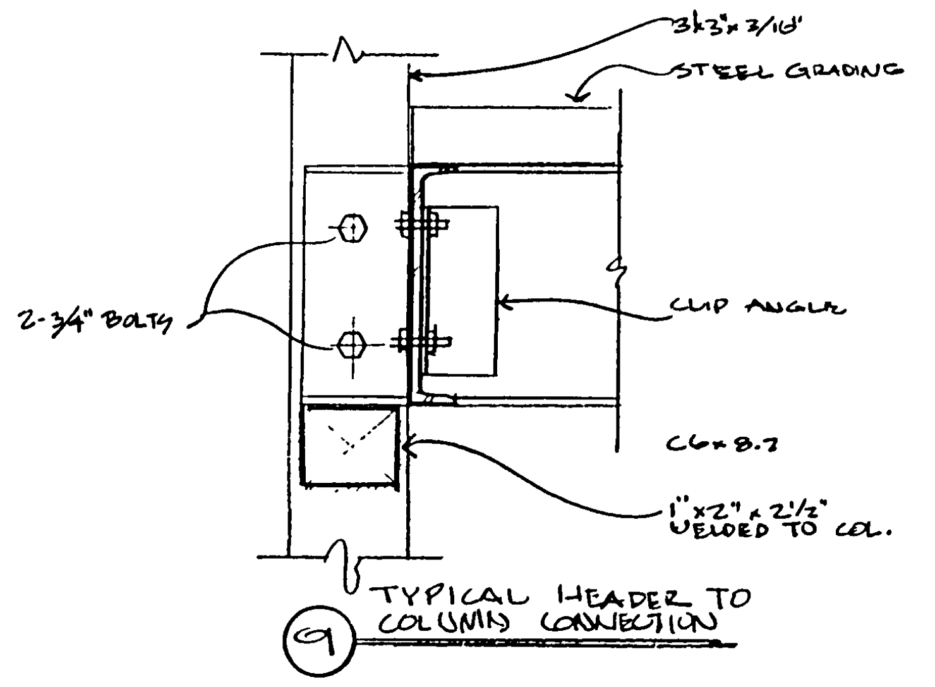




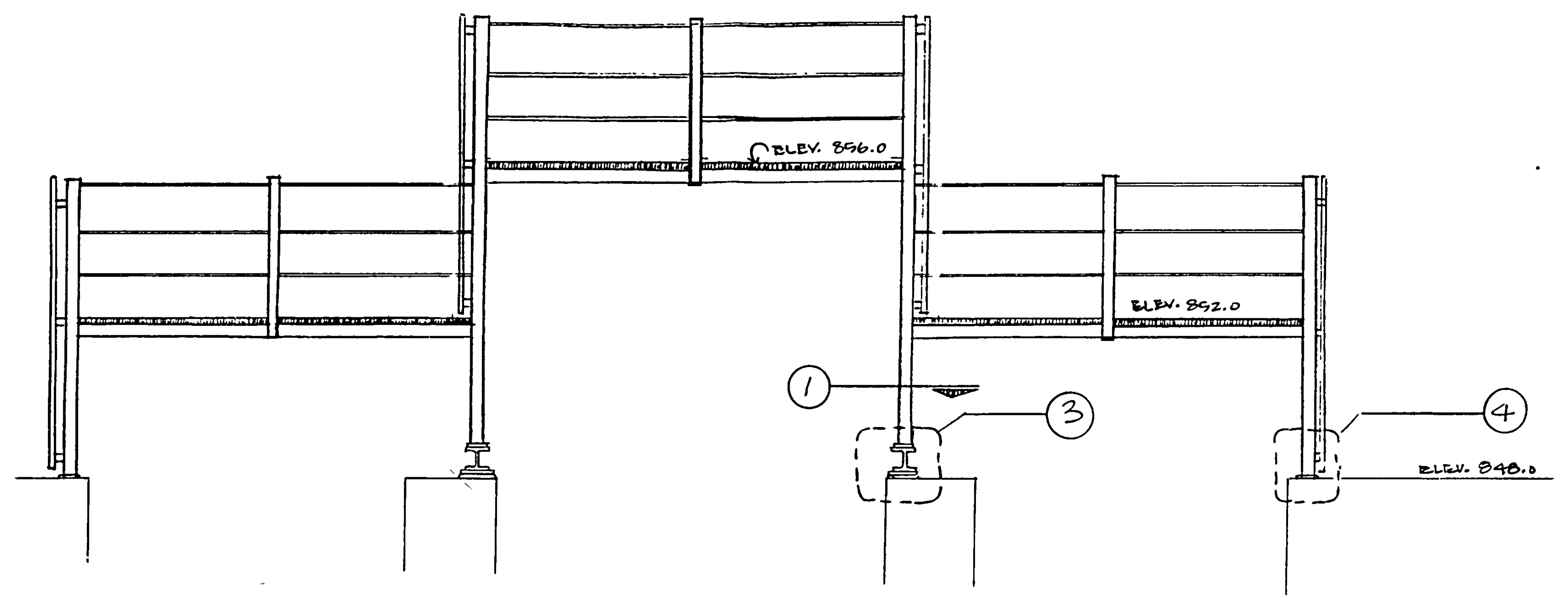




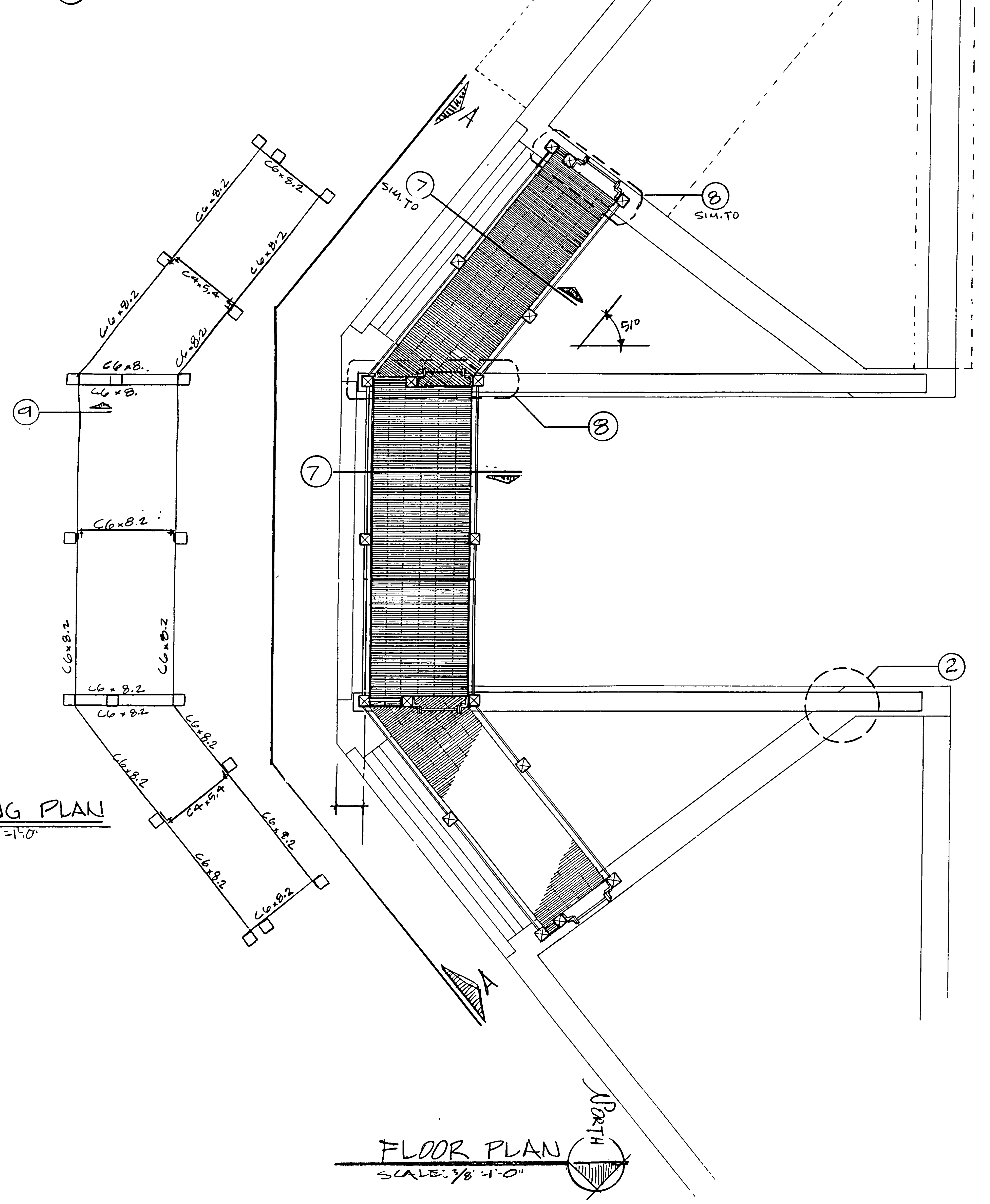




TYPICAL HEADER TO COLUMN CONNECTION



SECTION A



FRAMING PLAN
SCALE: 3/8"=1'-0"

FLOOR PLAN
SCALE: 3/8"=1'-0"

RECEIVED
SEP 0 8 2000
ENVIRONMENTAL QUALITY
LAND & WATER MANAGEMENT

Newhof & Winer
Grand Rapids, Michigan
Architects
Engineers
Planners

olson, meyers & may, inc.
civil engineers
1550 e. beltline se
grand rapids, mi. 49506
616-957-4350

sewer permit number
water permit number
approved by
name
date
revised in accordance w/construction records
by
date

| revisions | by | date |
|---|---|---|
| Revised December 1989 to reflect DNR review comments | E J | 12-89 |

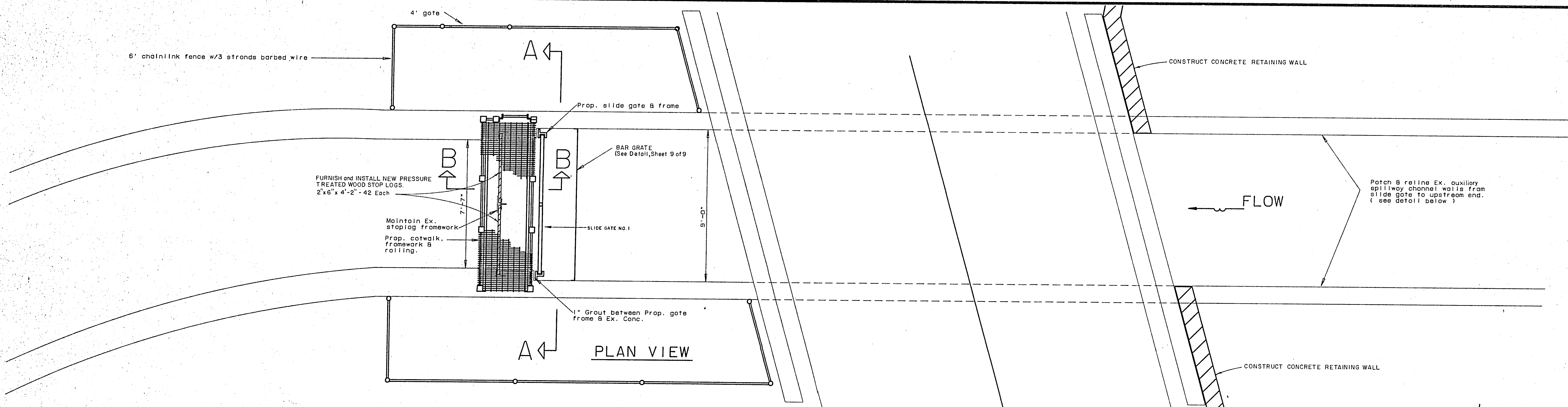
CITY OF WHITE CLOUD
Newaygo County, Michigan
WHITE CLOUD DAM
RECONSTRUCTION

| drawn by | date |
|---|---|
| N.& W. | 10-89 |
| checked by | date |
| D.G.M. | 10-89 |

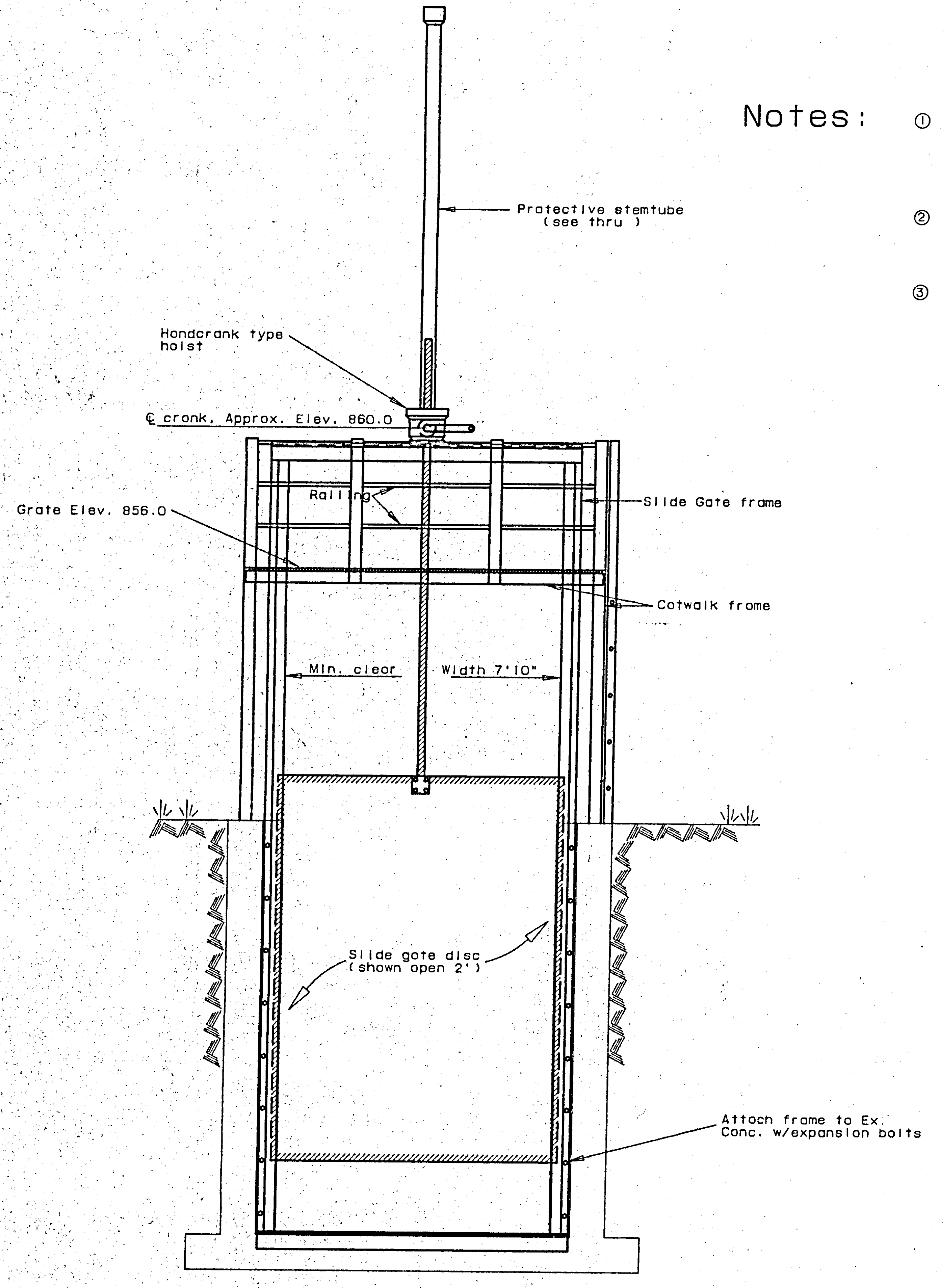
project number
01355

sheet number
7 of 9

CATWALKS: PRINCIPAL SPILLWAY



## PLAN VIEW



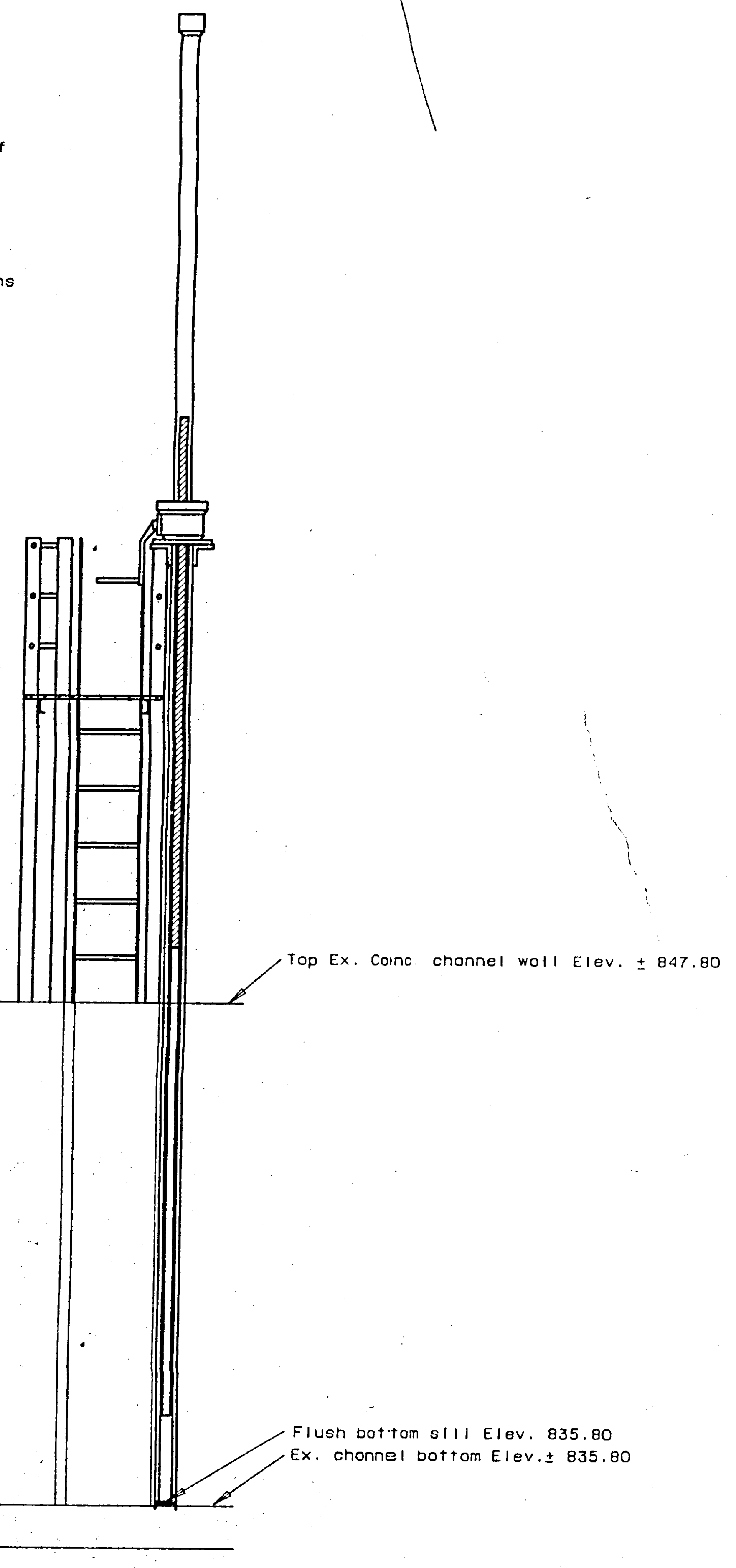
## SECTION A-A

Notes:

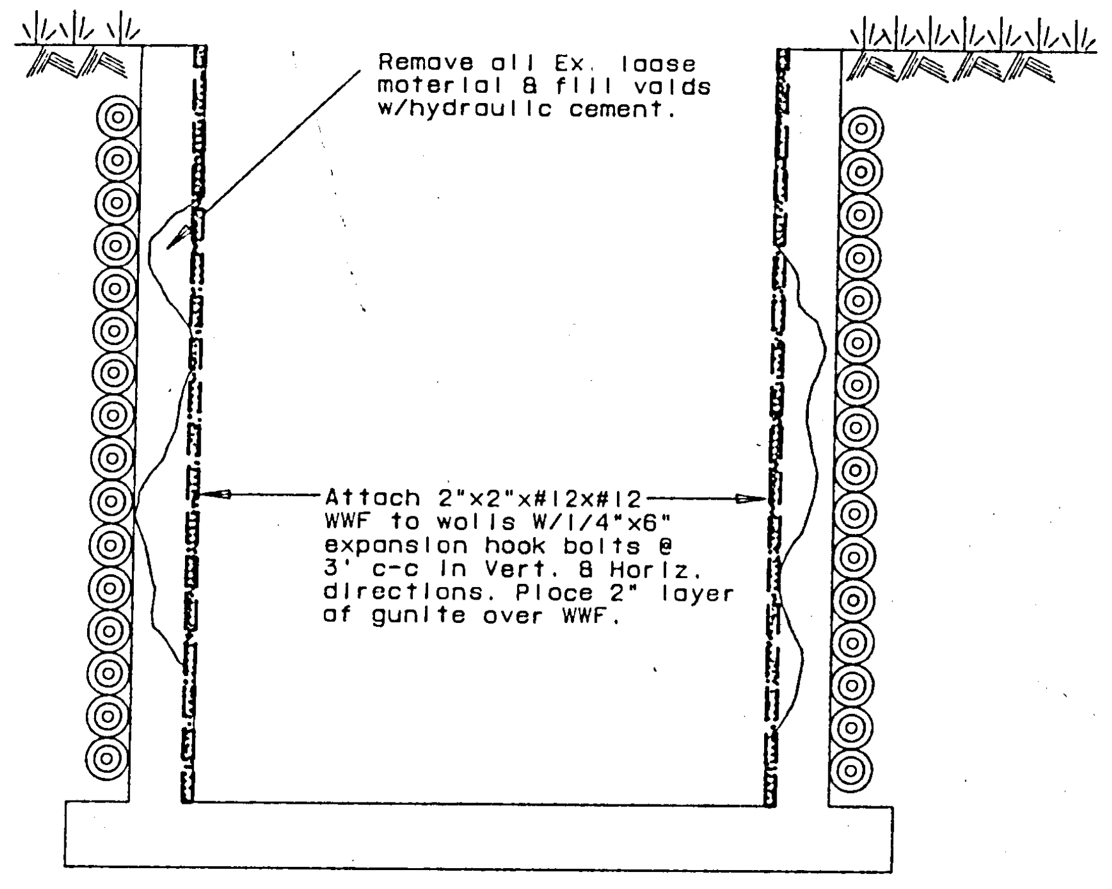
① Slide Gate, Frame & Hoist shown for the purpose of illustrating location, required opening dimensions and water levels. Actual details of member sizes, fabrication and installation to be supplied by manufacturer.

② Catwalk system shown to illustrate general layout and Elev. Actual steel member sizes and connections shall be similar to those shown on sheet #7 of 8 for principal spillway.

③ CONTRACTOR shall make field measurements as may be required by slide gate manufacturer to establish exact dimensions necessary for gate fabrication.



## SECTION B-B



## AUXILIARY SPILLWAY

Patching & Relining Details

RECEIVED
SEP 0 8 2000
ENVIRONMENTAL QUALITY
LAND & WATER MANAGEMENT

olson meyers & may, inc.
civil engineers/surveyors
1550 e. beltline se
grand rapids, mi. 49506
616-957-4350

sewer permit number
water permit number
approved by
name
date
revised in accordance by
w/construction records date

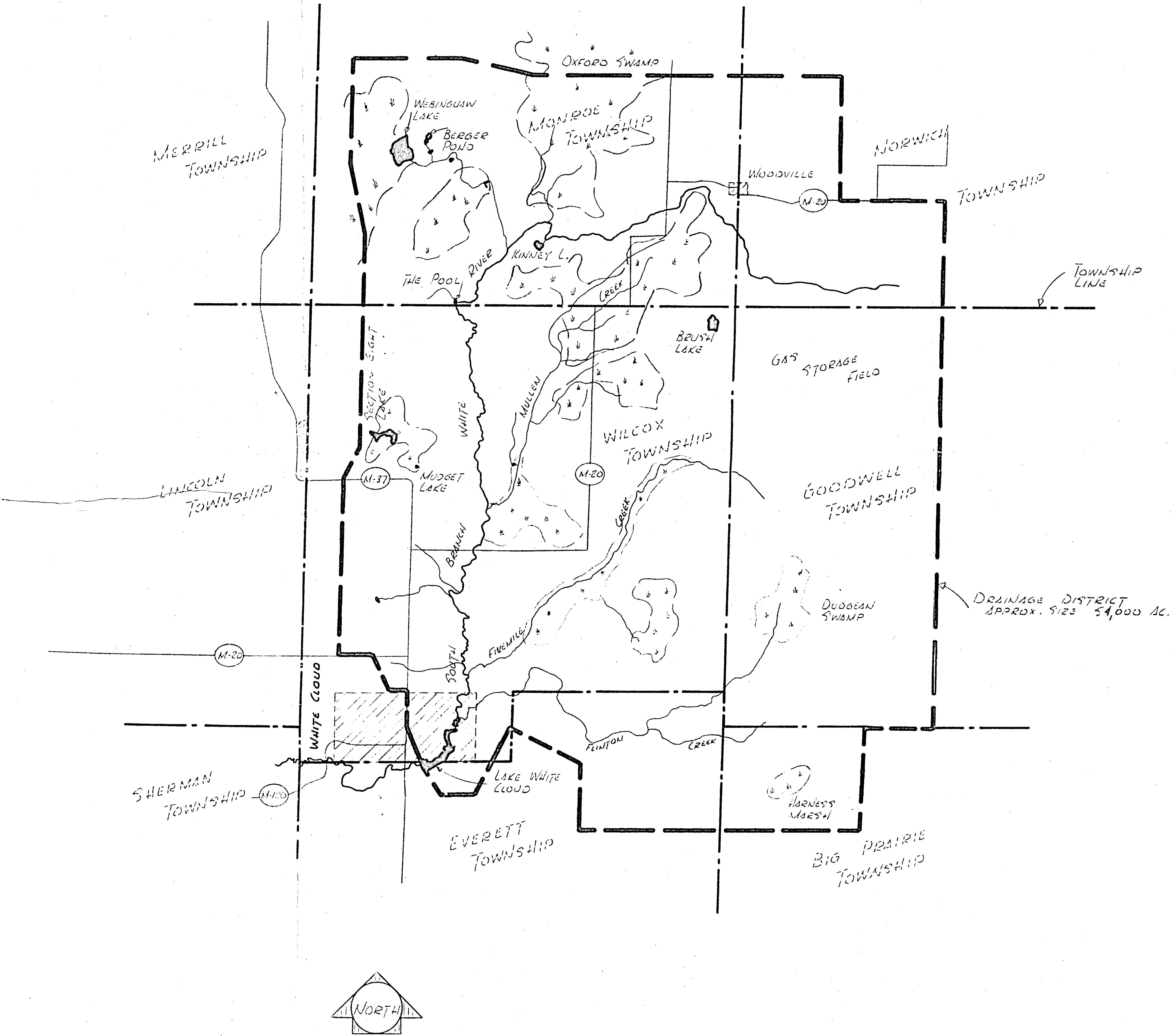
| revisions | by | date |
|---|---|---|
| Revised December, 1989 to reflect DNR review comments | E.J. | 12-89 |
| Add Stop Logs | E.J. | 1-90 |

CITY OF WHITE CLOUD
Newaygo County, Michigan

# WHITE CLOUD DAM RECONSTRUCTION

drawn by date
M C S 11/06/89
checked by date
D G M 11/06/89
project number
1355
sheet number
8 of 9

SLIDEGATE,CATWALK AND CHANNEL WALL REPAIRS: AUXILIARY SPILLWAY



SCALE: 1 INCH = 1 MILE

# IMPROVEMENT PLANS FOR
# LAKE WHITE CLOUD

WHITE CLOUD, MICHIGAN

**ENGINEER**

GERALD J. GRAY
PE & RLS 17741

PREPARED FOR
VILLAGE OF WHITE CLOUD
MICHIGAN

DESIGN BY:
DRAWN BY: R
JOB NO.: 7416
DATE: 10-29-74
APP'D. BY:
SHT 1 OF 4

**NEWAYGO ENGINEERING and SURVEYING**
P.O. BOX 35, 274 N. CHARLES ST. (M-37)
WHITE CLOUD, MICHIGAN 49349
616-689-2571

14 × 18 or 8½ × 11
60% reduction

## QUANTITIES

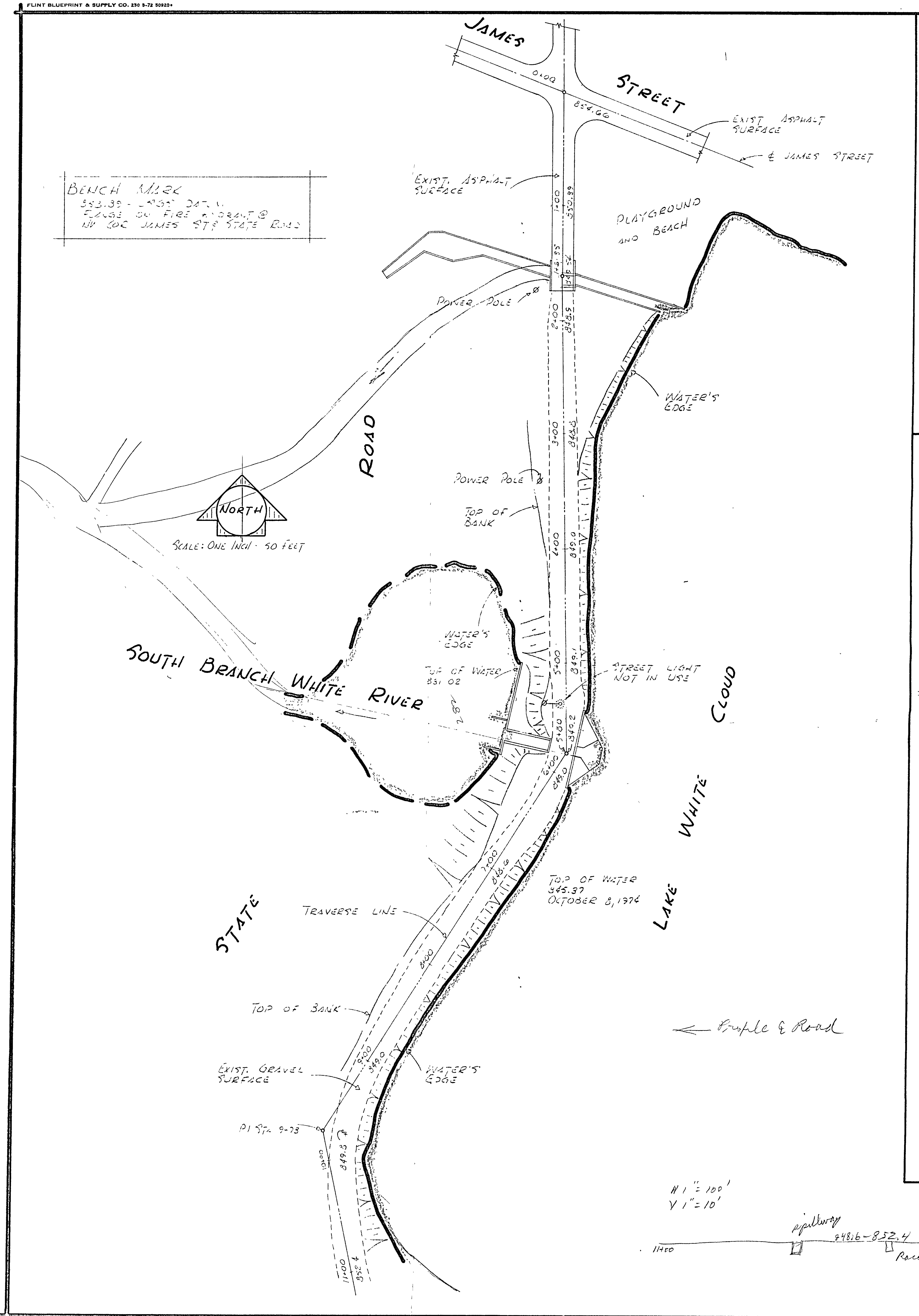
| | | |
|---|---|---|
| 100 | CU. YDS | - REINFORCED CONC. |
| 250 | CU. YDS | - 6" OR LARGER STONE |
| 105'± | LIN. FT. | - 6" C.I. WATERLINE |
| 30 | LIN FT | - 12" STEEL CASING |
| 450 | LIN FT. | - EXCAVATING, BACKFILLING & EARTHWORK AS REQUIRED |
| 515 | LIN FT | - CHAIN LINK FENCE |

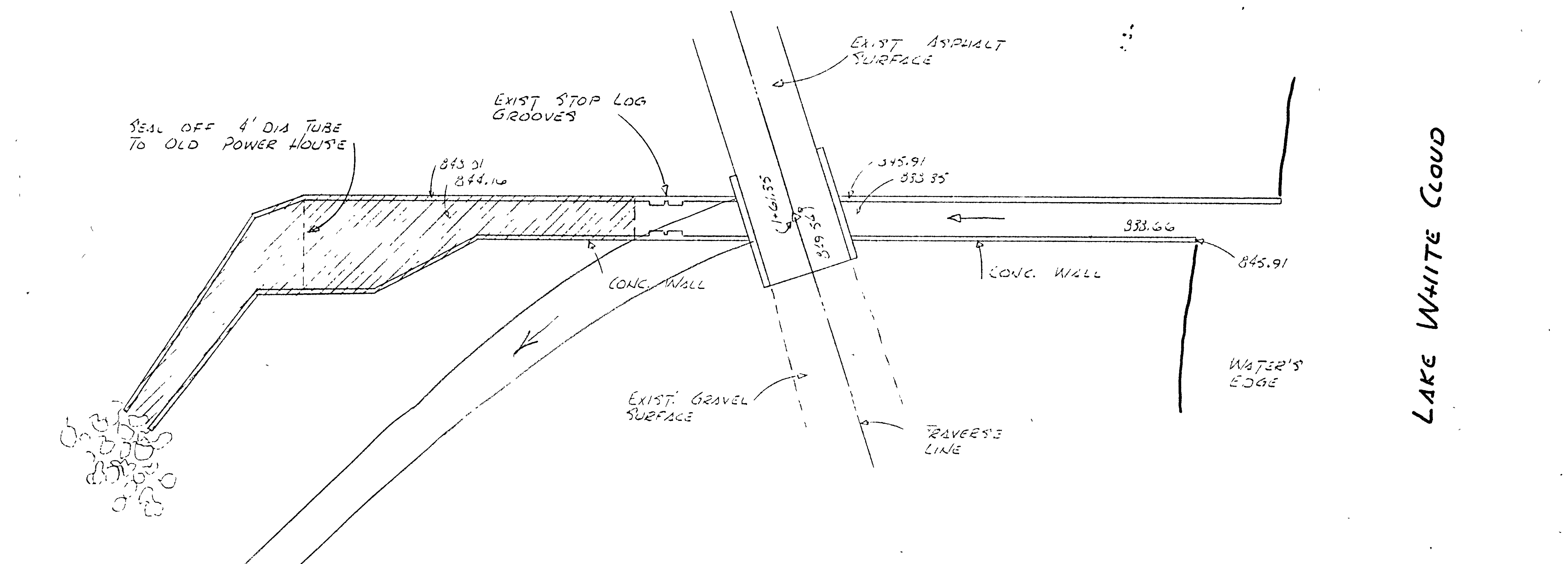
ALL QUANTITIES ARE ESTIMATES

IMPROVEMENT PLANS FOR
**LAKE WHITE CLOUD**
PREPARED FOR
**VILLAGE OF WHITE CLOUD**
MICHIGAN

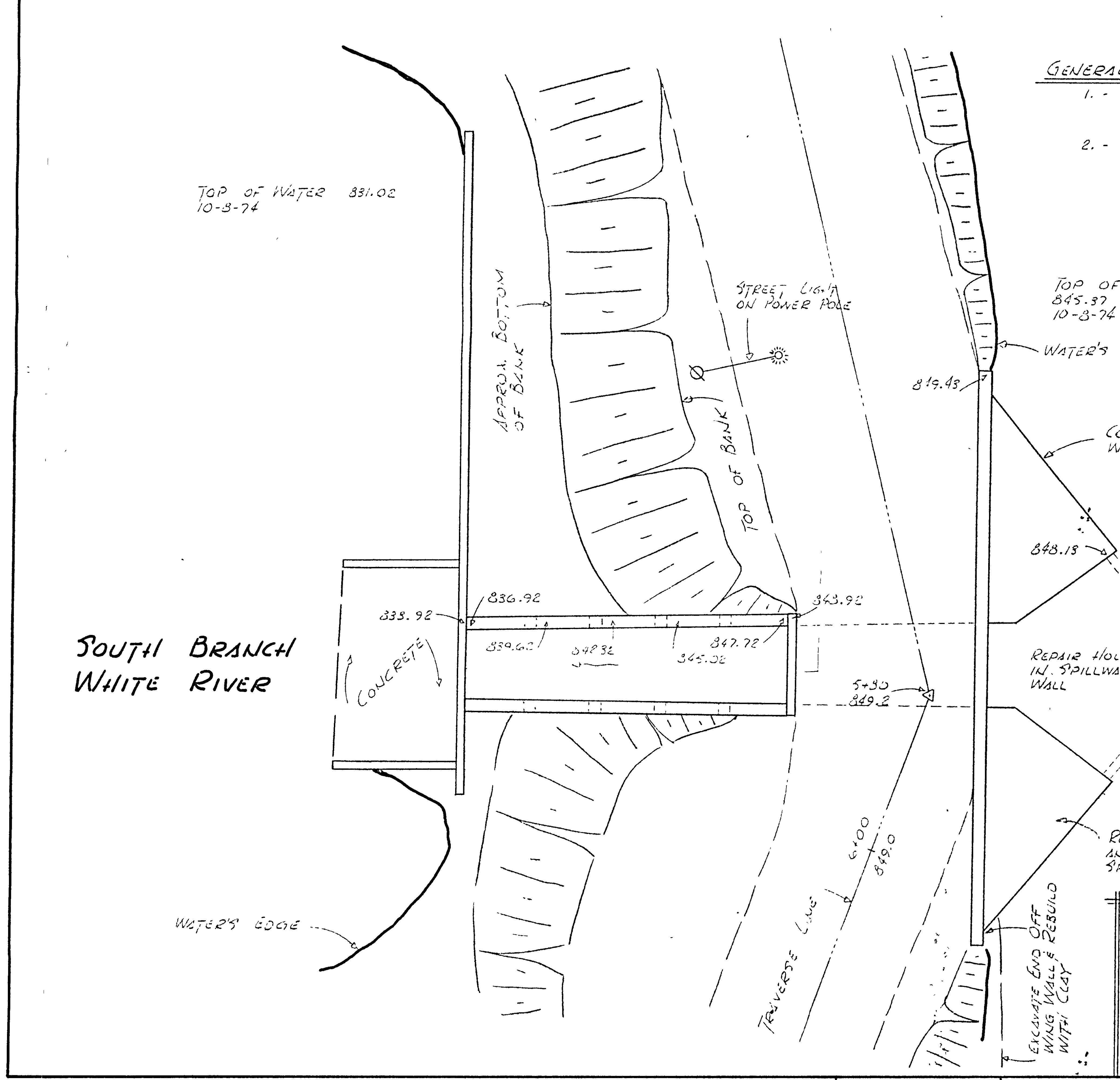
**NEWAYGO ENGINEERING AND SURVEYING**
274 N CHARLES ST. (M-37)
WHITE CLOUD, MICHIGAN
616-689-2571

DRAWN BY: R
SCALE: 1" = 20'
DATE: 12/18/75
JOB NO.
SHEET NO. 2 OF 4





## RACEWAY DETAILS
Scale: 1" = 20'



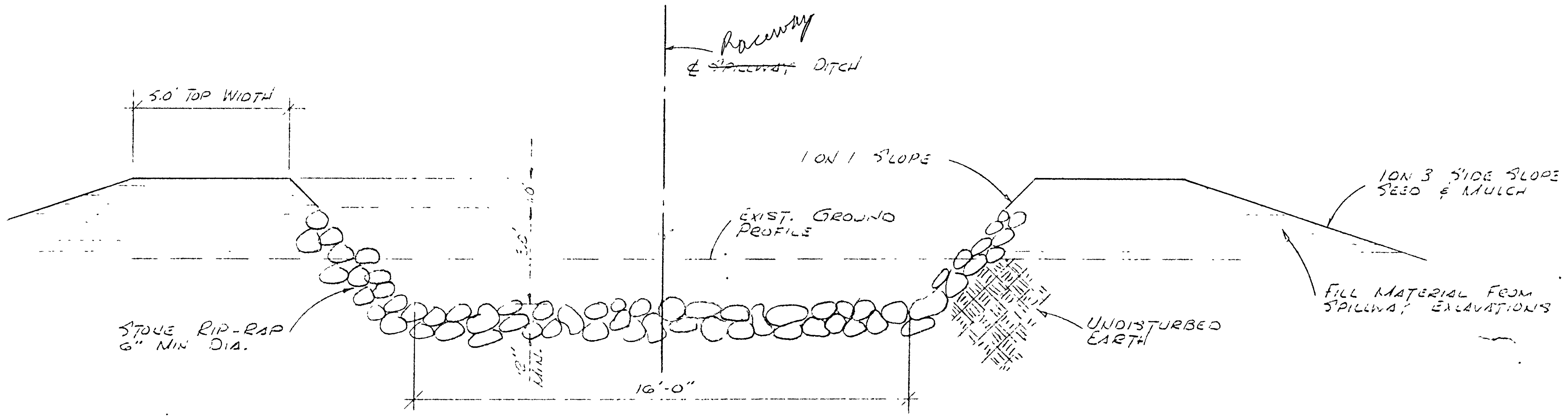
## GENERAL NOTES
1. - Drop water level of lake in 6 inch increments for further inspection.
2. - Seal all cracks in existing structure.

Detail of
## SPILLWAY STRUCTURE
For
## LAKE WHITE CLOUD
Scale: 1" = 10'

Prepared For
## VILLAGE OF WHITE CLOUD
Michigan

| | | | |
|---|---|---|---|
| DESIGN BY: | JOB NO.: 7416 | DATE: 10-29-74 | |
| DRAWN BY: R | | APP'D. BY: | SHT. 3 OF 4 |

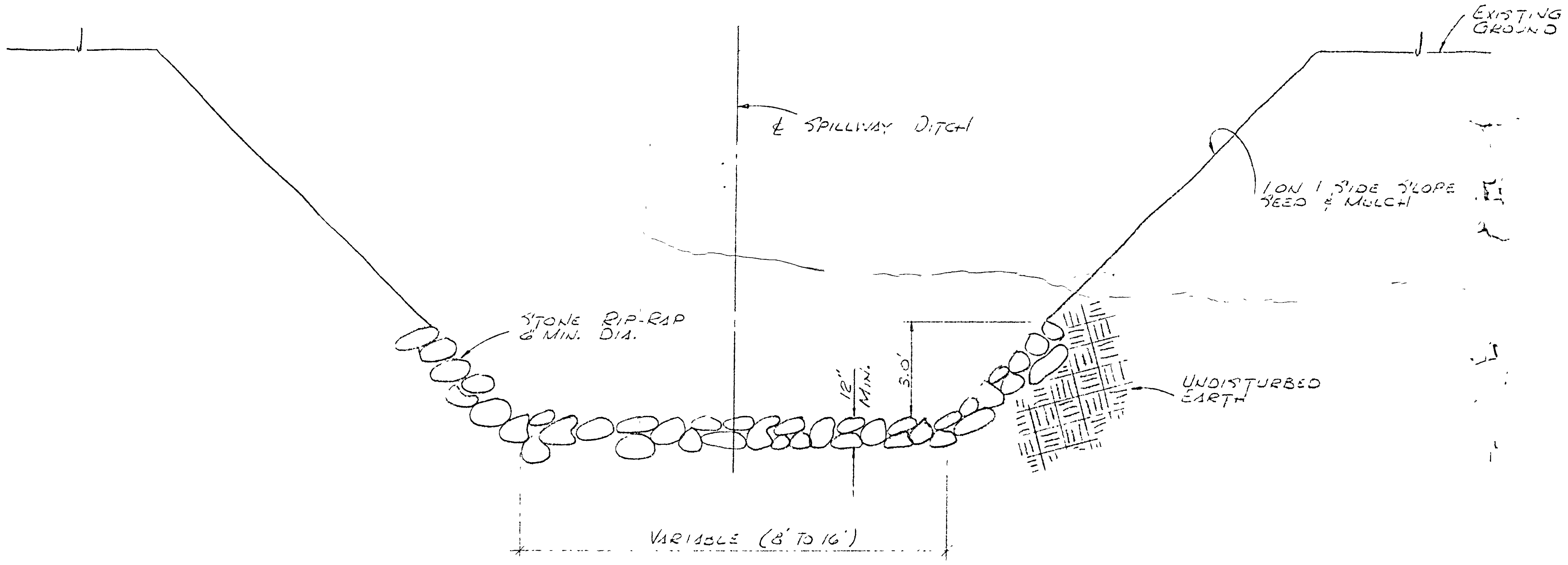
**NEWAYGO ENGINEERING and SURVEYING**
P.O. BOX 35, 274 N CHARLES ST. (M-37)
WHITE CLOUD, MICHIGAN 49349
616-689-2571



RACEWAY DITCH CROSS-SECTION

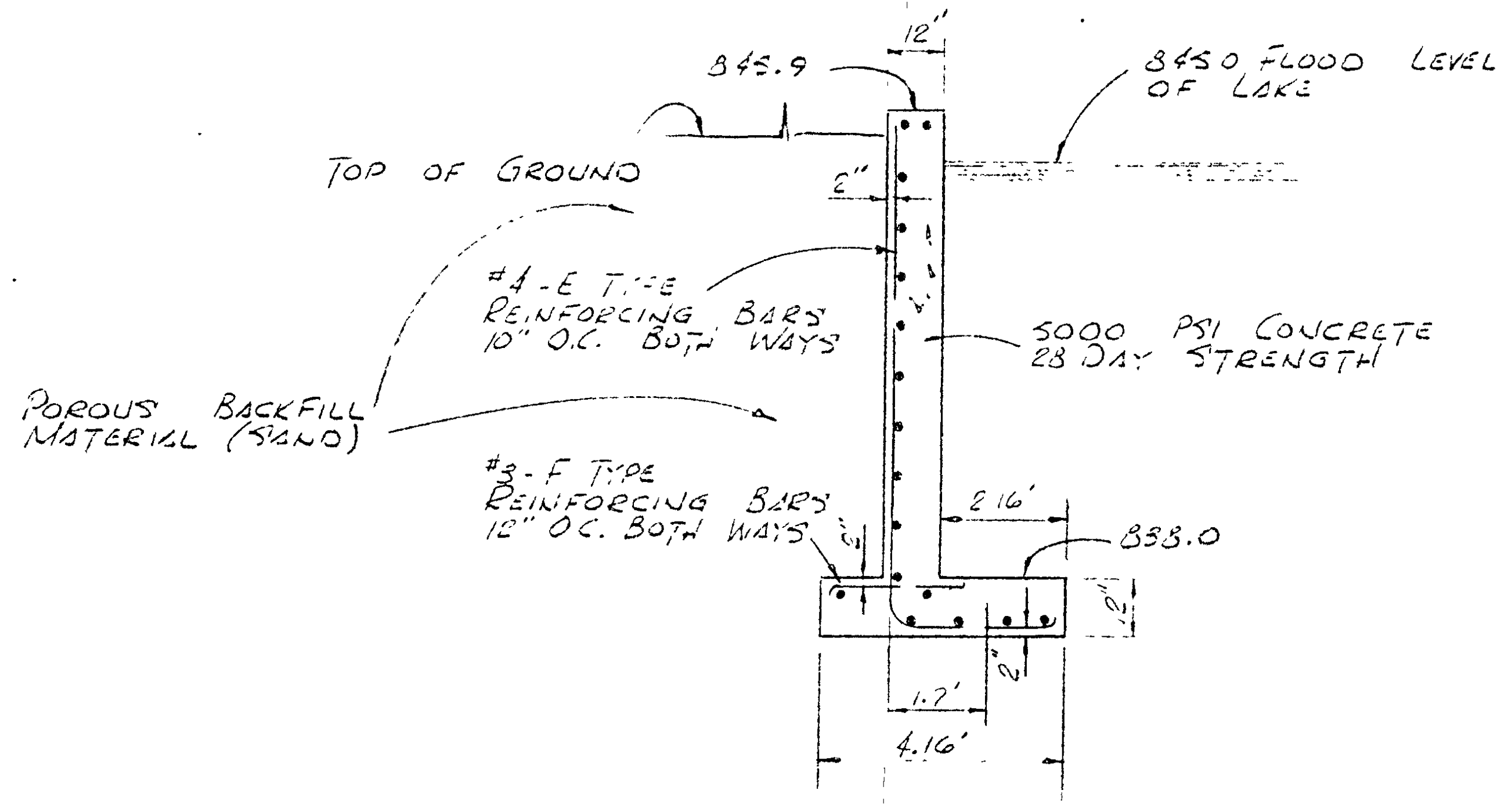
SCALE: 1"=3.0'

X B



RACEWAY DITCH CROSS-SECTION

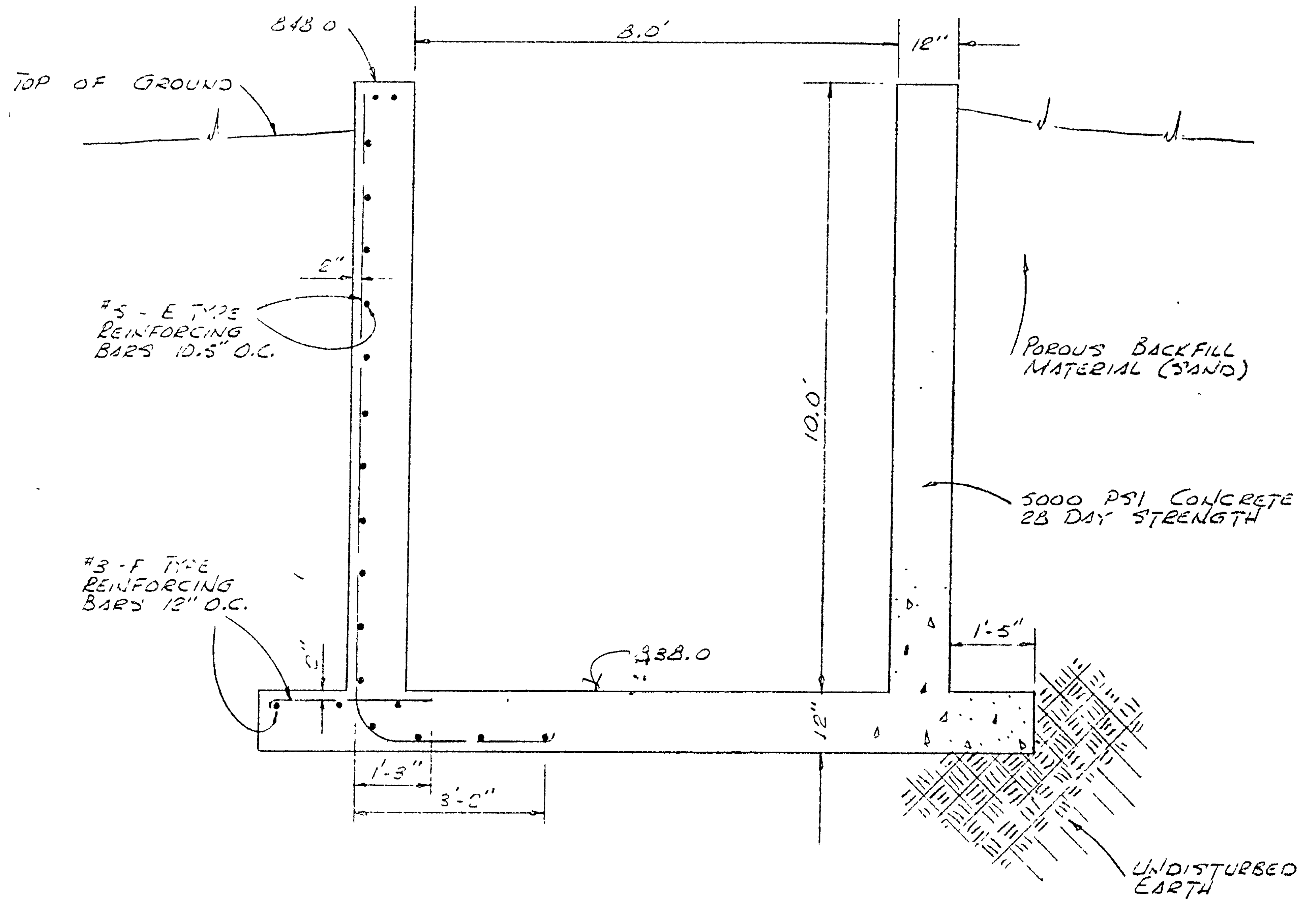
SCALE: 1"=3.0'



RETAINING WALL AT LAKE

SCALE: 1'=3.5'

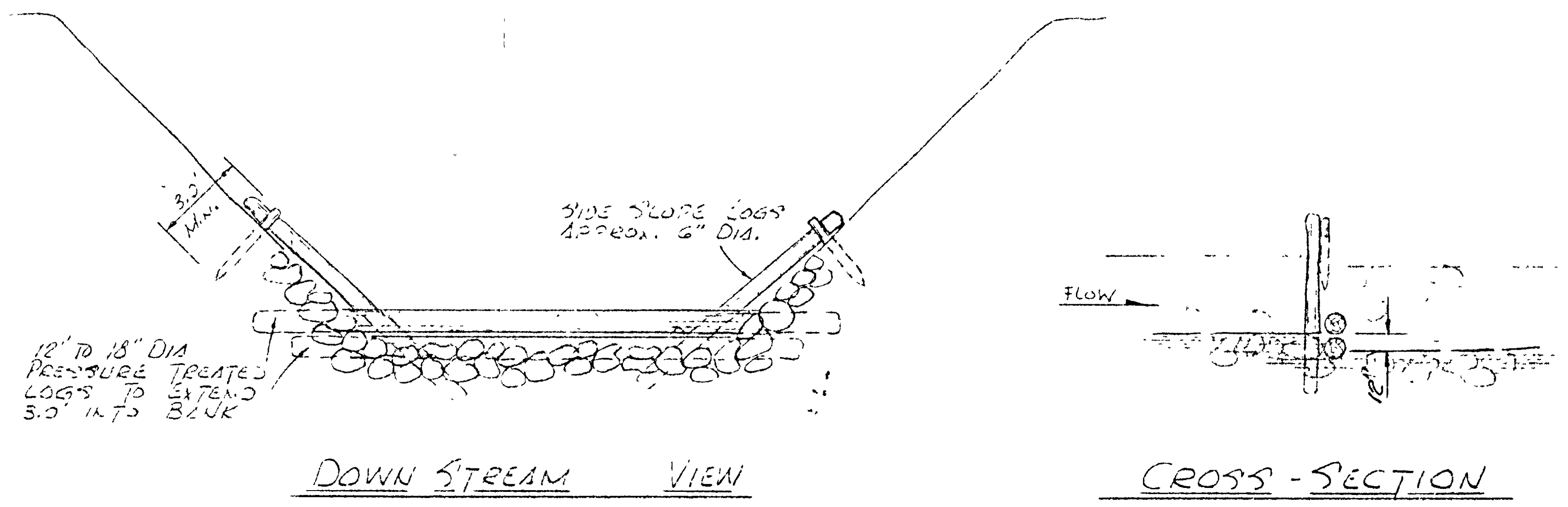
X C



RACEWAY CROSS-SECTION

SCALE 1"=2.0'

X A



DOWN STREAM VIEW

CROSS-SECTION

CHECK DAM DETAILS

NO SCALE

IMPROVEMENT PLANS FOR
**LAKE WHITE CLOUD**
PREPARED FOR
**VILLAGE OF WHITE CLOUD**
MICHIGAN

**NEWAYGO ENGINEERING**
**AND SURVEYING**
274 N CHARLES ST (M-37)
WHITE CLOUD, MICHIGAN
616-689-2571

| | |
|---|---|
| DRAWN BY: R | SCALE: AS SHOWN |
| DATE: 12/18/75 | |
| | SHEET NO 4 OF 4 |

# Appendix B – Site Observation Photo Log

# White Cloud Dam Inspection Photos

**Date:** 11/16/2023
**GEI Project No.:** 2302435
**Client:** City of White Cloud

# White Cloud Dam Inspection Photos

**Date:** 11/16/2023
**GEI Project No.:** 2302435
**Client:** City of White Cloud

Photo No. 1 – Auxillary Spillway – Downstream of Road Looking Downstream

Photo No. 2 – Auxillary Spillway – Operator Walkway Looking Upstream

Photo No. 3 – Auxillary Spillway – Downstream of Road Looking Downstream

Photo No. 4 – Auxillary Spillway – Gate and Operator Deck

Photo No.  5 – Auxillary Spillway – Gate and Operator Deck

Photo No.  6 – Auxillary Spillway – Trashrack Looking Downstream

# White Cloud Dam Inspection Photos

**Date:** 11/16/2023
**GEI Project No.:** 2302435
**Client:** City of White Cloud

Photo No. 7 – Auxillary Spillway – Trashrack Looking Right

Photo No. 8 – Auxillary Spillway – Concrete Deterioration Right Wall

Photo No.  9 – Auxillary Spillway – Erosion at Fence

Photo No.  10 – Auxillary Spillway – Upstream of Road Looking Upstream

Photo No. 11 – Auxillary Spillway – Upstream of Road Looking Upstream

Photo No. 12 – Auxillary Spillway – Upstream of Road Looking Upstream

Photo No.  13 – Auxillary Spillway – Upstream of Road Looking Upstream

Photo No.  14 – Auxillary Spillway – Upstream of Road Looking Downstream

Photo No.  15 – Auxillary Spillway – Upstream of Road Looking Downstream

Photo No.  16 – Auxiliary Spillway – Impoundment Looking Downstream

Photo No.  17 – Boat Launch, Auxiliary Spillway & Playground – Impoundment Looking Downstream

Photo No.  18 – Auxiliary Spillway, Playground and Beach – Impoundment Looking Downstream

Photo No. 19 – Boat Launch – Looking Upstream

Photo No. 20 – Boat Launch – Looking Left

Photo No.  21 – Primary Spillway – Right-Side from Boat Launch

Photo No.  22 – Right Embankment – Downstream Toe Looking Left

# White Cloud Dam Inspection Photos

**Date:** 11/16/2023
**GEI Project No.:** 2302435
**Client:** City of White Cloud

Photo No.  23 – Right Embankment – Downstream Toe Looking Upstream

Photo No.  24 – Right Embankment – Downstream Toe Seepage

Photo No. 25 – Right Embankment – Downstream Toe Looking Upstream

Photo No. 26 – Right Embankment – Downstream Toe Seepage

Photo No.  27 – Right Embankment – Downstream Toe Seepage

Photo No.  28 – Right Embankment – Downstream Toe Looking Right

Photo No. 29 – Right Embankment – Downstream Toe Looking Upstream

Photo No. 30 – Right Embankment – Downstream Toe Looking Upstream – End of Visible Seepage Right of Primary Spillway

Photo No.  31 – Downstream of Right Embankment – Looking Downstream

Photo No.  32 – Downstream of Right Embankment – Looking Downstream

Photo No. 33 – Right Embankment – Downstream Toe Looking Left

Photo No. 34 – Right Embankment – Downstream Toe Seepage

Photo No.  35 – Primary Spillway – Right Wall Downstream of Road

Photo No.  36 – Right Embankment and Right Primary Spillway Wall - Downstream of Road

Photo No.  37 – Outlet of Primary Spillway

Photo No.  38 – Erosion at Toe of RCC Left Downstream Embankment

Photo No.  39 – Erosion at Toe of RCC Left Downstream Embankment

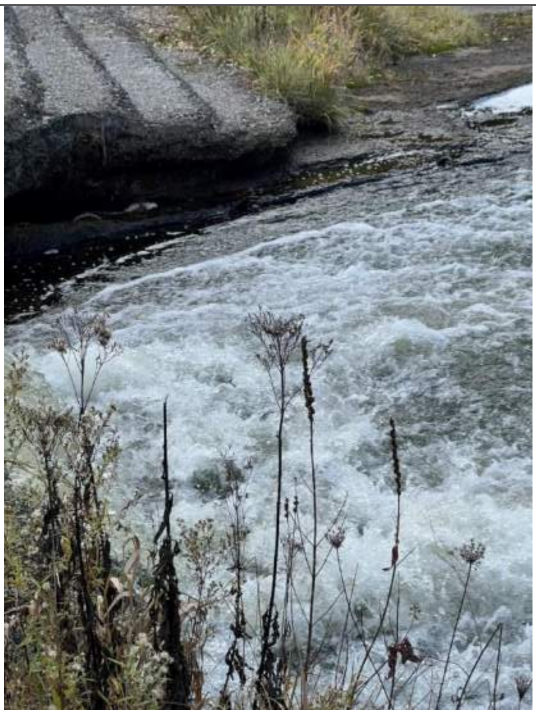


Photo No.  40 – Erosion at Toe of RCC Left Downstream Embankment

Photo No. 41 – Right Downstream Primary Spillway Wall

Photo No. 42 – Right Downstream Primary Spillway Wall – Efflorescence and Concrete Deterioration

Photo No.  43 – Outlet of Primary Spillway

z

Photo No.  44 – Outlet of Primary Spillway – Erosion at Right Spillway Wall

Photo No. 45 – Outlet of Primary Spillway – Looking Upstream

Photo No. 46 – Right Concrete Wall – Downstream of Primary Spillway Looking Downstream

Photo No. 47 – Downstream Toe of Right Embankment – Looking Downstream

Photo No. 48 – Right Upstream Flood Wall Looking Right

Photo No.  49 – Right Upstream Flood Wall – Looking Upstream

Photo No.  50 – Right Upstream Flood Wall at Primary Spillway Right Abutment

Photo No. 51 – Right Primary Spillway Abutment Looking Upstream

Photo No. 52 – Right Primary Spillway Abutment

Photo No.  53 – Roadway over Primary Spillway – Looking Left

Photo No.  54 – Cracking of Pavement over Primary Spillway – Looking Left

Photo No. 55 – Settlement of Pavement at Upstream Side of Right Primary Spillway Abutment – Looking Upstream

Photo No. 56 – Primary Spillway – Looking Upstream

Photo No. 57 – Right Primary Spillway Brace – Looking Upstream

Photo No. 58 – Downstream End of Right Primary Spillway Brace – Looking Right

Photo No.  59 – Left Primary Spillway Brace – Looking Upstream

Photo No.  60 – Middle Gate of Primary Spillway – Looking Upstream

Photo No. 61 – Right Gate of Primary Spillway – Looking Upstream

Photo No. 62 – Left Primary Spillway Abutment – Looking Right

Photo No. 63 – Primary Spillway – Impoundment Looking Downstream

Photo No. 64 – Right-Side of Primary Spillway – Impoundment Looking Downstream

Photo No. 65 – Primary Spillway – Impoundment Looking Downstream

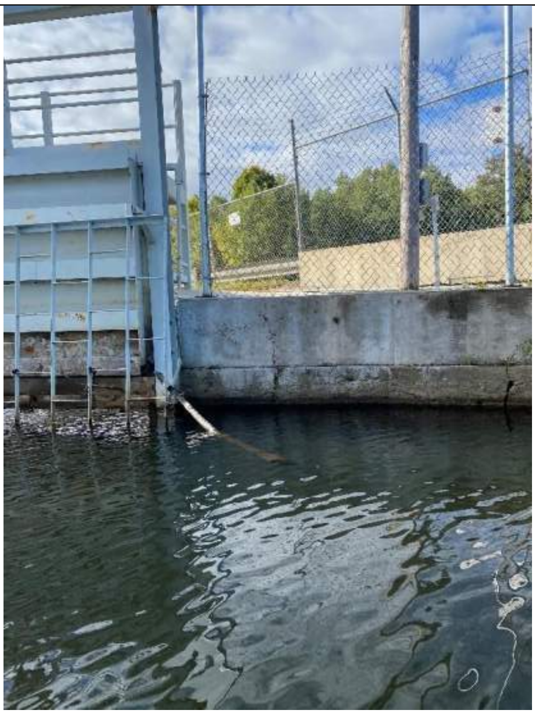


Photo No. 66 – Right Primary Spillway – Impoundment Looking Downstream

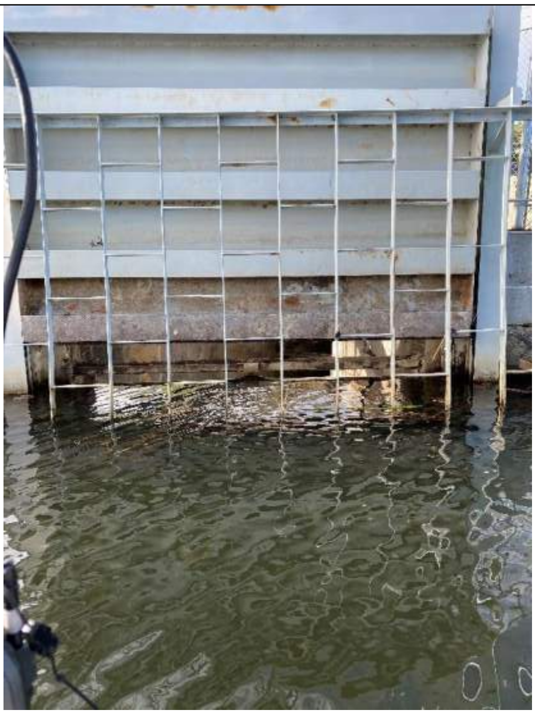


Photo No.  67 – Middle Primary Spillway – Impoundment Looking Downstream

Photo No.  68 – Left Primary Spillway – Impoundment Looking Downstream

Photo No. 69 – Left Primary Spillway – Impoundment Looking Downstream

Photo No. 70 – Left Primary Spillway – Impoundment Looking Downstream

Photo No.  71 – Upstream Side of Left Embankment at Overtopping Section – Looking Left

Photo No.  72 – Upstream Side of Left Embankment at Overtopping Section – Looking Left

Photo No. 73 – Interface of Upstream Side of Left Embankment at Overtopping Section and Flood Wall – Looking Left

Photo No. 74 – Upstream Left Embankment at Flood Wall Adjacent to Overtop Section – Looking Upstream

Photo No.  75 – Upstream Left Embankment Flood Wall - Looking Left

Photo No.  76 – Upstream Left Embankment Flood Wall Transition to Riprap and Wall Rotation - Looking Left

Photo No.  77 – Upstream Left Embankment Flood Wall Transition to Riprap - Looking Upstream

Photo No.  78 – Upstream Left Embankment Flood Wall - Looking Right

Photo No. 79 – Upstream Left Embankment Flood Wall Transition to Riprap and Wall Rotation - Looking Left

Photo No. 80 – Upstream Left Embankment Flood Wall - Looking Left

Photo No.  81 – Upstream Left Embankment Flood Wall - Looking Right

Photo No.  82 – Upstream Left Embankment Flood Wall - Looking Right

Photo No. 83 – Downstream Left Embankment RCC Slope

Photo No. 84 – Downstream Left Embankment RCC Slope – Looking Downstream

Photo No. 85 – Downstream Left Embankment RCC Slope – Looking Downstream

Photo No. 86 – Downstream Left Embankment RCC Slope – Looking Downstream

Photo No. 87 – Cracking in Pavement at Crest over RCC – Looking Downstream

Photo No. 88 – Downstream Left Embankment RCC Slope – Looking Downstream

Photo No.  89 – Downstream Left Embankment RCC Slope – Looking Downstream

Photo No.  90 – Downstream Left Embankment RCC Slope – Looking Left

Photo No. 91 – Downstream Left Embankment RCC Slope Deterioration – Looking Left

Photo No. 92 – Downstream Left Embankment RCC Slope Deterioration – Looking Left

Photo No. 93 – Primary Spillway Outlet – Looking Downstream

Photo No. 94 – Primary Spillway Outlet – Looking Downstream

Photo No. 95 – Guardrail Post Embedment at Primary Spillway Downstream Side of Crest – Looking Downstream

Photo No. 96 – Pavement Deterioration at Primary Spillway Downstream Side of Crest – Looking Downstream

Photo No. 97 – Concrete Deterioration at Right-Side Primary Spillway Downstream Side of Crest – Looking Downstream

# Appendix C – Sediment Lab Results

**IMPOUNDMENT SEDIMENT SAMPLING**
Michigan Direct Contact Criteria Comparison
White Cloud Feasibility Study, White Cloud, MI

| Hazardous Substance | Chemical Abstract Service Number | Statewide Default Background Levels | NonResidential Direct Contact Criteria (mg/kg) | Residential Direct Contact Criteria (mg/kg) | Site 1 | Site 2 | Site 3 | Site 4 | Site 5 |
|---|---|---|---|---|---|---|---|---|---|
| % Solids | | | | | 82% | 72% | 65% | 51% | 61% |
| Acenaphthene | 83329 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Acenaphthylene | 208968 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Anthracene | 120127 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Arsenic | 7440382 | 5800 | 0 | 0 | *ND* | *ND* | *ND* | 7.1 | 3.8 |
| Barium (B) | 7440393 | 75000 | 0 | 0 | *ND* | 13 | *ND* | 19 | 14 |
| Benzo(a)anthracene (Q) | 56553 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Benzo(b)fluoranthene (Q) | 205992 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Benzo(k)fluoranthene (Q) | 207089 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Benzo(g,h,i)perylene | 191242 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Benzo(a)pyrene (Q) | 50328 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Cadmium (B) | 7440439 | 1200 | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Chromium (VI) | 18540299 | NA | 0 | 0 | *ND* | 2.1 | *ND* | 3.9 | 2.2 |
| Chrysene (Q) | 218019 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Copper (B) | 7440508 | 32000 | 0 | 0 | *ND* | 1.2 | *ND* | 2.5 | 1.5 |
| Dibenzo(a,h)anthracene (Q) | 53703 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Fluorine (soluble fluoride) (B) | 7782414 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Indeno(1,2,3-cd)pyrene (Q) | 193395 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Lead (B) | 7439921 | 21000 | 900 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Mercury (Total) (B,Z) | Varies | 130 | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| 2-Methylnaphthalene | 91576 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Naphthalene | 91203 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Phenanthrene | 85018 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Pyrene | 129000 | NA | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Selenium (B) | 7782492 | 410 | 0 | 0 | *ND* | *ND* | *ND* | 0.81 | *ND* |
| Silver (B) | 7440224 | 1000 | 0 | 0 | *ND* | *ND* | *ND* | *ND* | *ND* |
| Zinc (B) | 7440666 | 47,000 | 0 | 0 | 4.6 | 9.2 | 6.2 | 13 | 7.6 |

*Sampled on 10/26/2023 by TRACE Analytics*

**IMPOUNDMENT SEDIMENT SAMPLING**
Aquatic PEC/TEC Comparison
White Cloud Feasibility Study, White Cloud, MI

| Hazardous Substance | Chemical Abstract Service Number | Consensus-Based TEC | Consensus-Based PEC | Site 1 (S-1) | Site 2 (S-2) | Site 3 (S-3) | Site 4 (S-4) | Site 5 (S-5) |
|---|---|---|---|---|---|---|---|---|
| Anthracene | 120127 | 57 | 845 | ND | *ND* | *ND* | *ND* | *ND* |
| Arsenic | 7440382 | 10 | 33 | ND | *ND* | *ND* | 7.1 | 3.8 |
| Benzo(a)anthracene (Q) | 56553 | 108 | 1,050 | ND | *ND* | *ND* | *ND* | *ND* |
| Benzo(a)pyrene (Q) | 50328 | 150 | 1,450 | ND | *ND* | *ND* | *ND* | *ND* |
| Cadmium (B) | 7440439 | 1 | 5 | ND | *ND* | *ND* | *ND* | *ND* |
| Chromium (III) (B,H) | 16065831 | 43 | 111 | ND | 2.1 | *ND* | 3.9 | 2.2 |
| Chromium (VI) | 18540299 | 43 | 111 | ND | 2.1 | *ND* | 3.9 | 2.2 |
| Copper (B) | 7440508 | 32 | 149 | ND | 1.2 | *ND* | 2.5 | 1.5 |
| Dibenzo(a,h)anthracene (Q) | 53703 | 33 |  | ND | *ND* | *ND* | *ND* | *ND* |
| Lead (B) | 7439921 | 36 | 128 | ND | *ND* | *ND* | *ND* | *ND* |
| Mercury (Total) (B,Z) | Varies | 0 | 1 | ND | *ND* | *ND* | *ND* | *ND* |
| Naphthalene | 91203 | 176 | 561 | ND | *ND* | *ND* | *ND* | *ND* |
| Zinc (B) | 7440666 | 121 | 459 | 4.6 | 9.2 | 6.2 | 13 | 7.6 |

*Sampled on 10/26/2023 by TRACE Analytics*

# Appendix D – Threatened and Endangered Species

MICHIGAN STATE UNIVERSITY | Extension

Zach Pitman
GEI Consultants
4472 Mount Hope Road, Suite A
Williamsburg, MI 49690

December 5, 2023

**Re: Rare Species Review #4731 – White Cloud Project, City of White Cloud, Newaygo County, MI**

Hello:

The location for the proposed project was checked against known localities for rare species and unique natural features, which are recorded in the Michigan Natural Features Inventory (MNFI) natural heritage database. This continuously updated database is a comprehensive source of existing data on Michigan's endangered, threatened, or otherwise significant plant and animal species, natural plant communities, and other natural features. Records in the database indicate that a qualified observer has documented the presence of special natural features. The absence of records in the database for a particular site may mean that the site has not been surveyed. The only way to obtain a definitive statement on the status of natural features is to have a competent biologist perform a complete field survey.

Under Act 451 of 1994, the Natural Resources and Environmental Protection Act, Part 365, Endangered Species Protection, "a person shall not take, possess, transport, …fish, plants, and wildlife indigenous to the state and determined to be endangered or threatened," unless first receiving an Endangered Species Permit from the Michigan Department of Natural Resources (MDNR), Wildlife Division. Responsibility to protect endangered and threatened species is not limited to the lists below. Other species may be present that have not been recorded in the database.

**Several at-risk species and/or natural communities have been documented within 1.5 miles of the project location and it is possible that adverse impacts will occur.** This response reflects a desktop review of the database and MNFI cannot fully evaluate this project without visiting the area. MNFI offers several levels of Rare Species Reviews, including field surveys which I would be happy to discuss with you.

Sincerely,

Michael Sanders

Michael Sanders
Environmental Review Specialist/Zoologist
Michigan Natural Features Inventory

**MSU EXTENSION**

**Michigan Natural Features Inventory**

PO Box 13036
Lansing MI 48901

(517) 284-6200
Fax (517) 373-9566

mnfi.anr.msu.edu

## Comments for Rare Species Review #4731:

It is important to note that it is the applicant's responsibility to comply with both state and federal threatened and endangered species legislation. Therefore, if a state listed species occurs at a project site, and you think you need an endangered species permit please contact: Amy Bleisch, DNR-Wildlife Division, DNR-StateTEPermit@michigan.gov. If a federally listed species is involved and, you think a permit is needed, please contact Jessica Pruden, U.S. Fish and Wildlife Service, East Lansing office, 517-351-8316, or Jessica_Pruden@fws.gov.

**NOTE:** Special concern species and natural communities are not protected under endangered species legislation, but efforts should be taken to minimize any or all impacts. Please consult MNFI's Rare Species Explorer for additional information on Michigan's rare plants and animals.

**Table 1: Occurrences of Threatened & Endangered Species within 1.5 miles of Project Site**

| Element Category | Scientific Name | Common Name | Federal Status | State Status | G Rank | S Rank | EO Rank | First Observed Date | Last Observed Date |
|---|---|---|---|---|---|---|---|---|---|
| Animal | *Glyptemys insculpta* | Wood turtle | | T | G3 | S2 | E | 1954 | 2013-08-22 |
| Animal | *Lycaeides melissa samuelis* | Karner blue | LE | T | G1G2 | S2 | E | 2004-07-28 | 2004-07-28 |
| Animal | *Myotis lucifugus* | Little brown bat | | T | G3G4 | S1 | H | 1974-05-13 | 1974-05-13 |

## Comments for Table 1:

### Little brown bat (*Myotis lucifugus*)

Known to occur in the area. Before white-nosed syndrome devastated bat populations, little brown bats were the most common bat species in the northern half of the Lower Peninsula in Michigan accounting for roughly 60 percent of all mist net captures. They occur in a variety of habitats and their abundance is linked closely to availability mines and caves suitable for hibernation. Upon emergence from hibernation they travel throughout the state and will set up maternity roosts in man-made structures, utilizing barns, houses, large buildings, and the underside of bridges. They also roost in tree hollows and under loose bark. Little brown bats often forage over streams and ponds.

#### Management Recommendations

Little brown bat are generalists. Maintaining forest for roosting in and around open water for foraging would benefit this species. Protecting hibernacula from vandalism during winter is critical. For more information, see the *Myotis lucifugus* species page on the MNFI website.

### Wood turtle (*Glyptemys insculpta*)

Known to occur in the White River. Wood turtles are found primarily in or near moving water and associated riparian habitats. They prefer clear, medium-sized (range 7-100 ft / 2.1-30.5 m), hard-bottomed streams and rivers with sand and/or gravel substrates and moderate flow. Wood Turtles also require partially shaded, wet-mesic herbaceous vegetation such as raspberries, strawberries, grasses, willows, and alders along or near the river for foraging. Forested floodplains (deciduous and coniferous) with numerous sunlit openings and a dense mixture of low herbs and shrubs seem to provide ideal habitat for this species. They also have been found in non-forested habitats such as willow and alder thickets, sphagnum bogs, swamps, wet meadows, and old fields within or near the floodplain. Wood Turtles also require sandy or sandy-gravelly areas along the river for nesting but will utilize gravel pits, railroad crossings, clearcuts, roadways, utility right-of-ways, and residential yards and gardens if natural nesting habitat is not available.

**Management Recommendations**

The most serious threat to this species is poaching for commercial pet trade and incidental collecting by the public. The public should be informed and educated that this species is protected under the Director's order and should not be collected or harmed. Maintaining good water quality, controlling sedimentation, restricting pesticide use near waterways, implementing minimum development set-back distances, and leaving buffer zones along streams during timber harvest, grazing, and agricultural operations can help preserve Wood Turtle habitat. Maintaining stream dynamics that create sandy areas along the river is crucial for providing suitable nesting habitat. Maintaining or creating small openings in floodplain forests can provide foraging, basking, and/or nesting habitat. Management practices such as sand traps, streambank stabilization, stream channelization and dams can eliminate or reduce good wood turtle habitat and should be avoided. Predator control may be necessary at nesting areas to enable successful reproduction or recruitment. Road construction near streams and rivers should be avoided or minimized.

For more information, see the *Glyptemys insculpta* species page on the MNFI website.

### Table 2: Occurrences of Special Concern Species and Natural Communities within 1.5 miles of Project Site

| Element Category | Scientific Name | Common Name | Federal Status | State Status | G Rank | S Rank | EO Rank | First Observed Date | Last Observed Date |
|---|---|---|---|---|---|---|---|---|---|
| Animal | *Merolonche dolli* | Doll's merolonche | | SC | G3G4 | S2S3 | H | 1968 | 1968 |
| Animal | *Emydoidea blandingii* | Blanding's turtle | | SC | G4 | S2S3 | E | 2003-08-04 | 2003-08-04 |
| Animal | *Lithobates palustris* | Pickerel frog | | SC | G5 | S3S4 | E | 1925-10-04 | 2005-07-12 |

**Comments for Table 2:**

## Blanding's turtle (*Emydoidea blandingii*)

Known to occur in the area. Blanding's turtles inhabit clean, shallow waters with abundant aquatic vegetation and soft muddy bottoms over firm substrates. This species is found in ponds, marshes, swamps, bogs, wet prairies, river backwaters, embayments, sloughs, slow-moving rivers, and lake shallows and inlets. Blanding's Turtles also occupy terrestrial habitats in the spring and summer during the mating and nesting seasons and in the fall to a lesser extent. Females nest in open uplands adjacent to wetland habitats, preferring sunny areas with moist but well-drained sandy or loamy soil. They will nest in lawns, gardens, plowed fields or even gravel road embankments if suitable natural nesting habitat is not available.

## Management Recommendations

The most critical conservation need for this species is protection and management of suitable wetland and adjacent upland habitats. Maintaining good water quality, restricting herbicide and pesticide use in or near wetlands, implementing minimum development set-back distances, leaving buffer zones during timber harvest, grazing and agricultural operations, and minimizing the construction of roads in or near suitable wetlands would be beneficial to this species. Timber harvesting can benefit this species by creating or maintaining open habitat conditions for thermoregulation and nesting. Minimizing adult mortality or removal is crucial for population viability given this species' life history. Thus, habitat management activities should be conducted in such a manner to minimize the potential for causing take of adults (e.g., timber harvesting during the inactive season). Minimizing road mortality and illegal collection also would be beneficial to this species. In some cases, on-site protection of nest sites and predator control may be necessary to facilitate or increase successful reproduction or population recruitment. For more information, see the *Emydoidea blandingii* species page on the MNFI website.

## Codes to accompany tables

**State Protection Status Code Definitions**
E = Endangered
T = Threatened
SC = Special concern

**Federal Protection Status Code Definitions**
LE = listed endangered
LT = listed threatened
LELT = partly listed endangered and partly listed threatened
PDL = proposed delist
E(S/A) = endangered based on similarities/appearance
PS = partial status (federally listed in only part of its range)
C = species being considered for federal status

**Global Heritage Status Rank Definitions (G RANK)**
The priority assigned by NatureServe's national office for data collection and protection based upon the element's status throughout its entire world-wide range. Criteria not based only on number of occurrences; other critical factors also apply. Note that ranks are frequently combined.
G1 = critically imperiled globally because of extreme rarity (5 or fewer occurrences range-wide or very few remaining individuals or acres) or because of some factor(s) making it especially vulnerable to extinction.
G2 = imperiled globally because of rarity (6 to 20 occurrences or few remaining individuals or acres) or because of some factor(s) making it very vulnerable to extinction throughout its range.
G3 = Either very rare and local throughout its range or found locally (even abundantly at some of its locations) in a restricted range (e.g. a single western state, a physiographic region in the East) or because of other factor(s) making it vulnerable to extinction throughout its range; in terms of occurrences, in the range of 21 to 100.
G4 = Apparently secure globally, though it may be quite rare in parts of its range, especially at the periphery.
G5 = Demonstrably secure globally, though it may be quite rare in parts of its range, especially at the periphery.
Q = Taxonomy uncertain

**State Heritage Status Rank Definitions (S RANK)**
The priority assigned by the Michigan Natural Features Inventory for data collection and protection based upon the element's status within the state. Criteria not based only on number of occurrences; other critical factors also apply. Note that ranks are frequently combined.
S1 = Critically imperiled in the state because of extreme rarity (5 or fewer occurrences or very few remaining individuals or acres) or because of some factor(s) making it especially vulnerable to extirpation in the state.
S2 = Imperiled in state because of rarity (6 to 20 occurrences or few remaining individuals or acres) or because of some factor(s) making it very vulnerable to extirpation from the state.
S3 = Rare or uncommon in state (on the order of 21 to 100 occurrences). S4 = apparently secure in state, with many occurrences.
S5 = demonstrably secure in state and essentially ineradicable under present conditions.
SX = apparently extirpated from state.

**EO Rank Codes**

Element Occurrence (EO) ranks refer to the viability or ecological integrity of the occurrence; they provide an assessment of the likelihood that if current conditions prevail the EO will persist for a defined period of time, typically 20-100 years.

A - Excellent estimated viability/ecological integrity
A? - Possibly excellent estimated viability/ecological integrity
AB - Excellent or good estimated viability/ecological integrity
AC - Excellent, good, or fair estimated viability/ecological integrity
B - Good estimated viability/ecological integrity
B? - Possibly good estimated viability/ecological integrity
BC - Good or fair estimated viability/ecological integrity
BD - Good, fair, or poor estimated viability/ecological integrity
C - Fair estimated viability/ecological integrity
C? - Possibly fair estimated viability/ecological integrity
CD - Fair or poor estimated viability/ecological integrity
D - Poor estimated viability/ecological integrity
D? - Possibly poor estimated viability/ecological integrity
E - Verified extant (viability/ecological integrity not assessed)
F - Failed to find
F? - Possibly failed to find
H - Historical
H? - Possibly historical
X - Extirpated
X? - Possibly extirpated
U - Unrankable
NR - Not ranked

**Section 7 Comments for Rare Species Review #4731**
**White Cloud Project**
**City of White Cloud, Newaygo County, MI**
**Zach Pitman**
**GEI Consultants**

,December 5, 2023

**For projects involving Federal funding or a federal agency authorization**

The following information is provided to assist you with Section 7 compliance of the Federal Endangered Species Act (ESA). The ESA directs all Federal agencies "to work to conserve endangered and threatened species. Section 7 of the ESA, called "Interagency Cooperation," is the means by which Federal agencies ensure their actions, including those they authorize or fund, do not jeopardize the existence of any listed species."

The project falls within the range of the following federally listed/proposed/candidate species which have been identified by the U.S. Fish and Wildlife Service (USFWS) to occur in Newaygo County, Michigan:

**Federally Endangered**
**Karner blue butterfly** – there appears to be suitable habitat within 1.5 miles of the project site. The federally endangered and state threatened Karner blue butterfly *(Lycaeides melissa samuelis)* was historically found in open-canopied barrens communities, including oak and oak-pine savanna or barrens found prior to European settlement. Since their historical habitat suffers from fire suppression efforts, the butterfly often occurs in openings, old fields, and rights-of-way. Karner blue larvae feed exclusively on wild lupine (*Lupinus perennis*), an early successional species that can become abundant after appropriate disturbances. Adults visit a wide variety of flowering plants for nectar.

The Karner blue has two generations per year, with the later, or summer, generation typically having three to four times the number of adults as the earlier, or spring, brood. Adults are active most of the day, decreasing activity during midday and during cool, rainy weather. Females can live up to two weeks in the field, but typically live an average of five days. Peak flight dates are mid-May through early June and mid-July through early August, with stragglers found between.

*Management and Conservation:* recommendations for management of Karner blue butterfly habitat will be pertinent only if the host plant, wild lupine (*Lupinus perennis*) is present. If lupine is present the following guidelines should be followed: (1) mower blades should be set no lower than 6 inches; (2) mowing should not occur before August 15th (i.e. no spring mowing at all!); (3) no burning of habitat where lupine exists, and; (4) contact us if planting or logging will occur in lupine areas.

**Northern long-eared bat** - Northern long-eared bat (*M. septentrionalis*) numbers in the northeast US have declined up to 99 percent. Loss or degradation of summer habitat, wind turbines, disturbance to hibernacula, predation, and pesticides have contributed to declines in Northern long-eared bat populations. However, no other threat has been as severe to the decline as White-nose Syndrome (WNS). WNS is a fungus that thrives in the cold, damp conditions in caves and mines where bats hibernate. The disease is believed to disrupt the hibernation cycle by causing bats to repeatedly awake thereby depleting vital energy reserves. This species was federally listed in May 2015 primarily due to the threat from WNS.

Although no known hibernacula or roost trees have been documented within 1.5 miles of the project areas, this activity occurs within the designated WNS zone (i.e., within 150 miles of positive counties/districts impacted by WNS. In addition, there appears to be suitable habitat within 1.5 miles of the project site.

Also called northern bat or northern myotis, this bat is distinguished from other *Myotis* species by its long ears. In Michigan, northern long-eared bats hibernate in abandoned mines and caves in the Upper Peninsula; they also commonly hibernate in the Tippy Dam spillway in Manistee County. This species is a regional migrant with migratory distance largely determined by locations of suitable hibernacula sites.

Northern long-eared bats typically roost and forage in forested areas. During the summer, these bats roost singly or in colonies underneath bark, in cavities or in crevices of both living and dead trees. Roost trees are selected based on the suitability to retain bark or provide cavities or crevices. Common roost trees in southern Lower Michigan include species of ash, elm and maple. Foraging occurs primarily in areas along woodland edges, woodland clearings and over small woodland ponds. Moths, beetles and small flies are common food items. Like all temperate bats this species typically produces only 1-2 young per year.

*Management and Conservation:* **when there are no known roost trees or hibernacula in the project area, we encourage you to conduct tree-cutting activities and prescribed burns in forested areas during October 1 through March 31. When that is not possible, we encourage you to remove trees prior to June 1 or after July 31, as that will help to protect young bats that may be in forested areas but are not yet able to fly.**

**Federally Threatened**

**Eastern massasauga rattlesnake (EMR)** - **the project falls outside Tier 1 and Tier 2 EMR habitat as designated by the U.S. Fish & Wildlife Service (USFWS).** The federally threatened and state special concern eastern massasauga rattlesnake *(Sistrurus catenatus)* is Michigan's only venomous snake and occurs in a variety of wetland habitats including bogs, fens, shrub swamps, wet meadows, marshes, moist grasslands, wet prairies, and floodplain forests. Eastern massasaugas occur throughout the Lower Peninsula but are not found in the Upper Peninsula. Populations in southern Michigan are typically associated with open wetlands, particularly prairie fens, while those in northern Michigan are better known from lowland coniferous forests, such as cedar swamps. These snakes normally overwinter in crayfish or small mammal burrows often close to the groundwater level and emerge in spring as water levels rise. During late spring, these snakes move into adjacent uplands they spend the warmer months foraging in shrubby fields and grasslands in search of mice and voles, their favorite food.

Often described as "shy and sluggish", these snakes avoid human confrontation and are not prone to strike, preferring to leave the area when they are threatened. However, like any wild animal, they will protect themselves from anything they see as a potential predator. Their short fangs can easily puncture skin and they do possess potent venom. Like many snakes, the first human reaction may be to kill the snake, but it is important to remember that all snakes play vital roles in the ecosystem. Some may eat harmful insects. Others like the massasauga consider rodents a delicacy and help control their population. Snakes are also a part of a larger food web and can provide food to eagles, herons, and several mammals.

*Management and Conservation:* protection of extant populations and suitable wetland and adjacent upland habitats is crucial for successful conservation of the Eastern Massasauga. Maintaining or restoring open habitat conditions is critical for this species. Fragmentation of suitable wetland-upland habitat complexes by roads or other barriers should be avoided or minimized. Land management practices such as timber harvesting, mowing, disking or prescribed burning should be conducted in such a manner so as to minimize the potential for adverse impacts to massasaugas (e.g., conducting management activities during the snakes' inactive season (November through early March) or on days when snakes are less likely to be active on the surface during the active season). Protecting suitable hibernation sites also is critical. Hydrological alterations such as drawdowns should be conducted prior to or after hibernation to reduce the potential for causing winter mortality due to desiccation or freezing. Sudden and/or permanent increases or decreases in water levels during the active season also can cause adverse impacts.

USFWS Section 7 Consultation Technical Assistance can be found at:

https://www.fws.gov/service/esa-section-7-consultation

The website offers step-by-step instructions to guide you through the Section 7 consultation process with prepared templates for documenting "no effect." as well as requesting concurrence on "may affect, but not likely to adversely affect" determinations.

# Appendix E – Hydrologic and Hydraulic Analysis

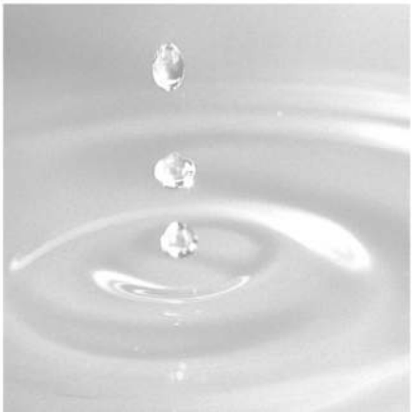
# Hydrology and Hydraulic Report for White Cloud Dam

White Cloud, Michigan

**Submitted to:**
The City of White Cloud
12 North Charles Street
White Cloud, MI 49349

**Submitted by:**
GEI Consultants of Michigan, P.C.
9282 General Drive, STE 180
Plymouth, MI 48170

February 14, 2024
Project No. 2302435



Janeen M. McDermott

Janeen McDermott, P.E.
Project Manager

Emma Giese

Emma Giese
Water Resources Engineer

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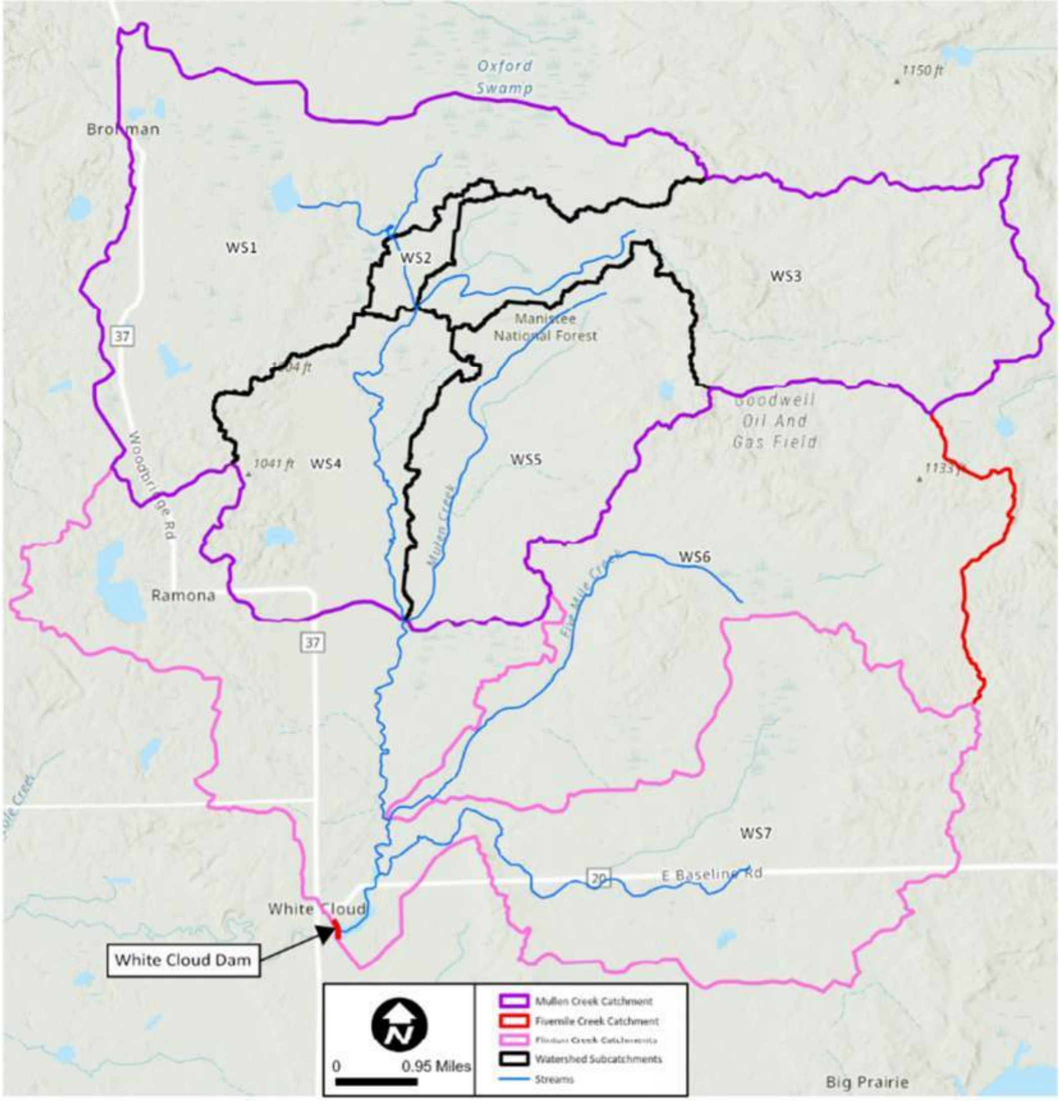
AR/EG/JMM
B:\Working\CITY OF WHITE CLOUD\2302435 White Cloud Dam Disposition Feasibility Study\ENG\H&H\HEC-HMS\White Cloud Hydrology Report_FINAL

# 1. Background

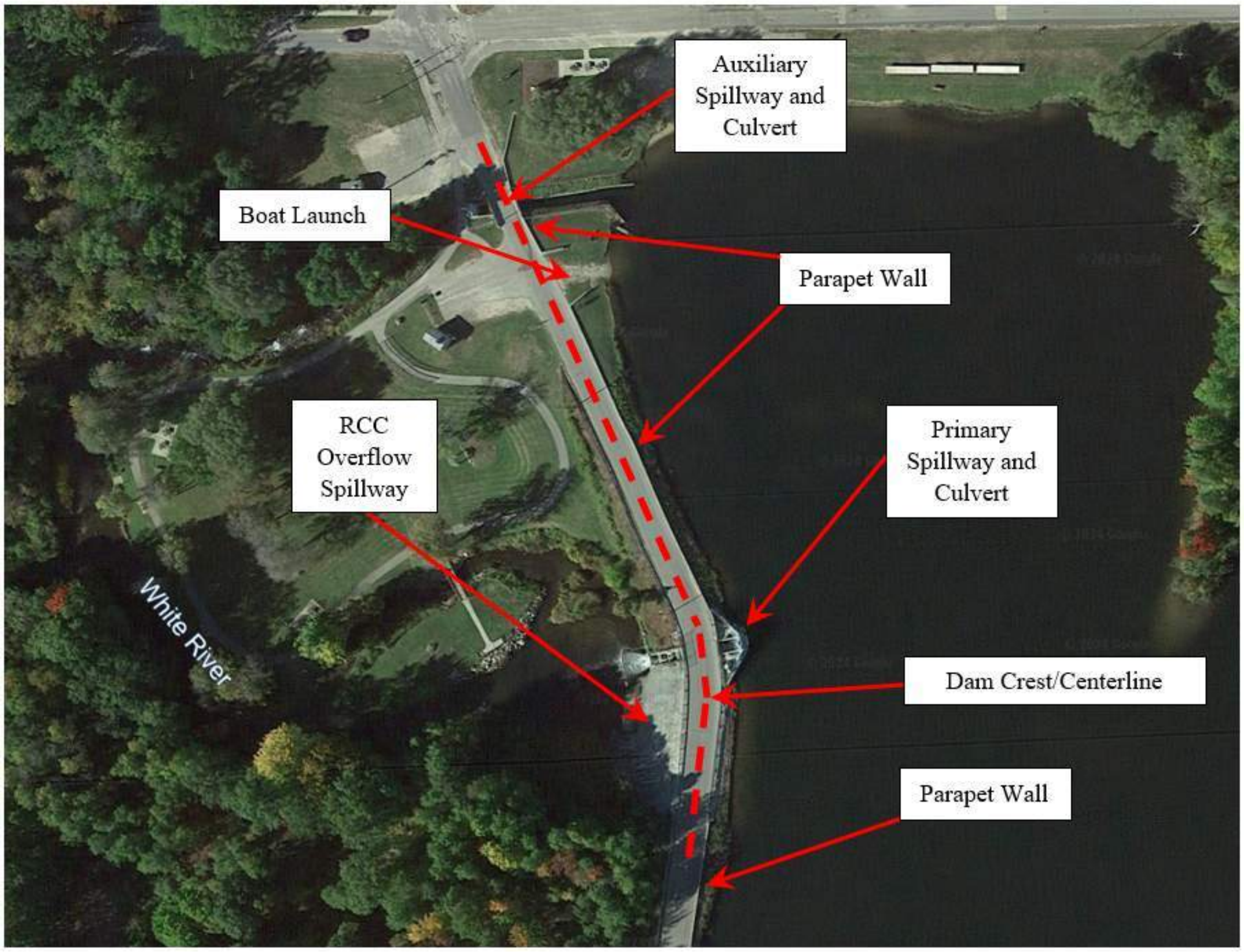
## 1.1. Background

The White Cloud Dam located on the White River in White Cloud, Michigan is owned and operated by The City of White Cloud. Due to numerous deficiencies that have been identified during dam safety and a structural inspection report including frequent overtopping events, the dam has received a poor condition rating from EGLE Dam Safety. The scope of this report includes the methodology of the hydrologic modelling involved in the simulation of the watershed seen in **Figure 1-1** as well as the inflow results for the hypothetical frequency storm events, and methodology and results of hydraulic modeling to evaluate the hydraulic capacity of the existing structures. A detailed image of the dam structure layout is shown in **Figure 1-2**, below.

**Figure 1-1. White Cloud Dam Watershed – HEC-HMS Model Area**



## Figure 1-2. Layout of White Cloud Dam Structures



# 2. Hydrology Model Development

## 2.1. Precipitation and Gauge Data

Publicly available precipitation data sources and known dam overtopping or near-overtopping events were used to validate the model (**Table 2-1**). The nearest USGS stream gauge located on the White River was evaluated and considered to be too far outside the project boundaries to be an accurate representation of stream flow on site. There was one Weather Underground hourly precipitation gauge (KMIWHITE64) located within the watershed. This source utilizes an Ambient Weather WS-2902 device that records rainfall data in 5-minute intervals. The nearest National Oceanic and Atmospheric Administration (NOAA) precipitation gauge reports daily rainfall data and is located outside the watershed at the Big Rapids Water Treatment Plant (WTP).

A list of near-overtopping and overtopping events was generated using the records from the City of White Cloud and Michigan's Environment, Great Lakes, and Energy (EGLE) dam safety unit. The incidents used for this model include one near-overtopping event on August 10th, 2021 as well as three overtopping events which occurred in early October of 2019, mid-March of 2019, and mid-April of 2014. KMIWHITE64 only had available rainfall data for the August 10th, 2021 near-overtopping event. The Big Rapids WTP gauge was used to compare the hourly gauge during the near-overtopping event and all other earlier events within the period of record in **Table 2-1**.

**Table 2-1. Summary of Nearby Gauges**

| Gage Name | Source | Data Type | Lat/Long | Timestep | Time Period Evaluated |
|---|---|---|---|---|---|
| Shop - KMIWHITE64 | Weather Underground | Precipitation | 43.57 °N, 85.78 °W | Hourly<br>5-min | July 31st, 2021 - August 12th,2021 |
| Big Rapids Water Treatment Plant | Midwest Regional Climate Center | Precipitation | 43.7072/ -85.4819 | Daily | April 6th, 2014 - August 12th,2021 |

## 2.2. Probability Storm Events

In addition to the known events, probability storms were simulated to generate flood flow hydrographs. Probability storms refer to storms with the likelihood of a storm of a certain size and duration occurring every year. For example, a 24-hour, 200-year storm is a day-long storm that has a 0.5% chance of occurring in any given year. These storm events were simulated using NOAA Atlas-14 (NOAA, 2013) 24-hour rainfall estimates (**Table 2-2)**, and an SCS Type II distribution. These storm events were modelled in HEC-HMS to generate 1- year to 500-year, 24-hour flood flow hydrographs.

**Table 2-2. Precipitation in Inches During Frequency Storm Events (NOAA Atlas-14, 2013)**

| **Duration** | **Average recurrence interval (years)** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | **1** | **2** | **5** | **10** | **25** | **50** | **100** | **200** | **500** | **1000** |
| **5-min** | 0.286 | 0.345 | 0.444 | 0.531 | 0.658 | 0.761 | 0.869 | 0.982 | 1.14 | 1.26 |
| **10-min** | 0.42 | 0.504 | 0.65 | 0.778 | 0.964 | 1.11 | 1.27 | 1.44 | 1.67 | 1.85 |
| **15-min** | 0.512 | 0.615 | 0.793 | 0.949 | 1.18 | 1.36 | 1.55 | 1.75 | 2.04 | 2.26 |
| **30-min** | 0.715 | 0.865 | 1.12 | 1.34 | 1.67 | 1.93 | 2.2 | 2.49 | 2.89 | 3.21 |
| **60-min** | 0.946 | 1.12 | 1.44 | 1.73 | 2.17 | 2.54 | 2.93 | 3.35 | 3.96 | 4.45 |
| **2-hr** | 1.18 | 1.38 | 1.76 | 2.12 | 2.67 | 3.14 | 3.65 | 4.22 | 5.02 | 5.68 |
| **3-hr** | 1.34 | 1.55 | 1.95 | 2.34 | 2.97 | 3.52 | 4.13 | 4.81 | 5.8 | 6.62 |
| **6-hr** | 1.62 | 1.83 | 2.27 | 2.72 | 3.47 | 4.15 | 4.92 | 5.78 | 7.07 | 8.16 |
| **12-hr** | 1.89 | 2.12 | 2.6 | 3.11 | 3.97 | 4.76 | 5.65 | 6.67 | 8.18 | 9.46 |
| **24-hr** | 2.15 | 2.42 | 2.98 | 3.56 | 4.52 | 5.41 | 6.41 | 7.55 | 9.24 | 10.7 |
| **2-day** | 2.4 | 2.75 | 3.43 | 4.1 | 5.2 | 6.18 | 7.28 | 8.51 | 10.3 | 11.8 |
| **3-day** | 2.6 | 3 | 3.77 | 4.51 | 5.67 | 6.68 | 7.8 | 9.04 | 10.8 | 12.3 |
| **4-day** | 2.79 | 3.22 | 4.03 | 4.79 | 5.98 | 7 | 8.12 | 9.34 | 11.1 | 12.6 |
| **7-day** | 3.38 | 3.8 | 4.6 | 5.35 | 6.52 | 7.53 | 8.64 | 9.85 | 11.6 | 13 |
| **10-day** | 3.91 | 4.34 | 5.15 | 5.9 | 7.08 | 8.08 | 9.18 | 10.4 | 12.1 | 13.5 |
| **20-day** | 5.38 | 5.96 | 6.97 | 7.86 | 9.16 | 10.2 | 11.3 | 12.5 | 14.2 | 15.5 |
| **30-day** | 6.59 | 7.32 | 8.53 | 9.55 | 11 | 12.1 | 13.3 | 14.4 | 16 | 17.3 |
| **45-day** | 8.15 | 9.03 | 10.5 | 11.6 | 13.2 | 14.4 | 15.6 | 16.8 | 18.3 | 19.5 |
| **60-day** | 9.5 | 10.5 | 12 | 13.3 | 15 | 16.3 | 17.5 | 18.7 | 20.2 | 21.3 |

## 2.3. Hydrologic Computer Model

The U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center's - Hydrologic Modeling System (HEC-HMS) Version 4.12 computer model was used to model the existing watershed.

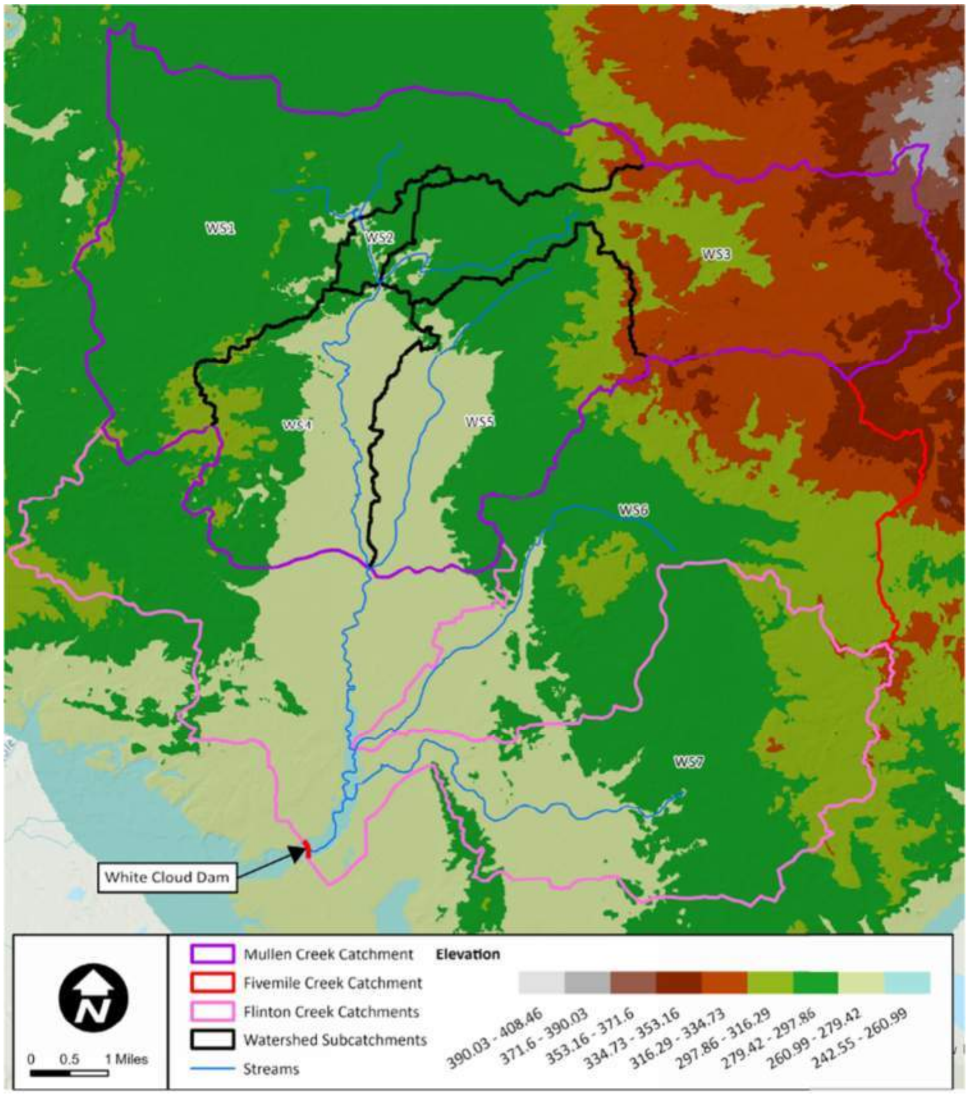
## 2.4. Watershed Delineation

Using the Environmental Protection Agency's Watershed Assessment, Tracking & Environmental Results System (WATERS), the watershed was delineated between the three 12-digit Hydrologic Unit Code (HUC) subwatersheds that feed into the White Cloud Dam; Flinton Creek, Fivemile Creek, and Mullen Creek. As Mullen Creek was approximately 47 mi$^2$ in size, it was further split into groups of 14-digit HUC subcatchments within the Mullen Creek subwatershed based on the various streams located within the project area. The subcatchments were labelled WS1 through WS7 as shown in **Table 2-3.** The primary stream for the watershed is the South Branch White River, which reaches from near the top of the Mullen Creek subwatershed to the White Cloud Dam. The other major streams which flow into the South Branch White River include James Creek located in WS1, Mullen Creek in WS5, Fivemile Creek in W6, Flinton Creek in WS7, and an unnamed stream located in WS3. The subcatchments ranged in size from 0.87 mi$^2$ to 29.39 mi$^2$, with a total watershed area of 94.57 mi$^2$. Additionally, elevations for the watershed are in NAVD88 vertical datum and were found using the United States Geological Survey's (USGS) National Map Viewer and LiDAR datasets. The location and the elevations of the subcatchments can be seen in **Figure 2-1.**

**Table 2-3. Watershed Subcatchments**

| Subcatchment Name | Area (mi²) |
|---|---|
| WS1 - James Creek | 16.09 |
| WS2 - Post-James Creek | 0.87 |
| WS3 - Unnamed River | 12.64 |
| WS4 - South Branch White River | 7.36 |
| WS5 - Mullen Creek | 9.54 |
| WS6 - Fivemile Creek | 18.68 |
| WS7 - Flinton Creek-South Branch White River | 29.39 |

**Figure 2-1. Subcatchments and Elevation Map**



## 2.5. Infiltration Loss Rates

A primary component of the simulation was the loss that occurs during rainfall events due to soil infiltration. For this project, the Green and Ampt model was used to simulate loss in the watershed.

### *2.5.1. Parameterization of Loss Rates*

The Green and Ampt model utilizes soil parameters based on the soil type of the surrounding area to simulate the amount of water that the soil can hold. The first parameter for the Green and Ampt model is the initial moisture content. The initial moisture content -or antecedent condition- indicates the amount of saturation in the soil layer at the start of the simulation. This parameter was set in HEC-HMS at normal conditions (i.e. not completely saturated or completely dry) as the weighted average of the water content at field capacity, or water content at one-third bar, for each subcatchment obtained from the United States Department of Agriculture (USDA) Web Soil Survey.  The next three parameters seen in **Table 2-4** are related to the types of soil in the watershed.

**Table 2-4. Soil Type**

| Texture | Saturated Hydraulic Conductivity (in/hr) | Wetting Front Suction Head (in) | Effective Porosity ($in^3/in^3$) |
|---|---|---|---|
| Sand | 4.60 | 1.90 | 0.42 |
| Loamy Sand | 1.20 | 2.40 | 0.40 |
| Loam | 0.10 | 3.50 | 0.43 |
| Sandy Loam | 0.40 | 4.30 | 0.41 |
| Silt Loam | 0.30 | 6.60 | 0.49 |
| Clay Loam | 0.04 | 8.20 | 0.31 |
| Sandy Clay Loam | 0.06 | 8.60 | 0.33 |
| Sandy Clay | 0.02 | 9.40 | 0.32 |
| Silty Clay Loam | 0.04 | 10.70 | 0.43 |
| Silty Clay | 0.02 | 11.50 | 0.42 |
| Clay | 0.01 | 12.50 | 0.39 |

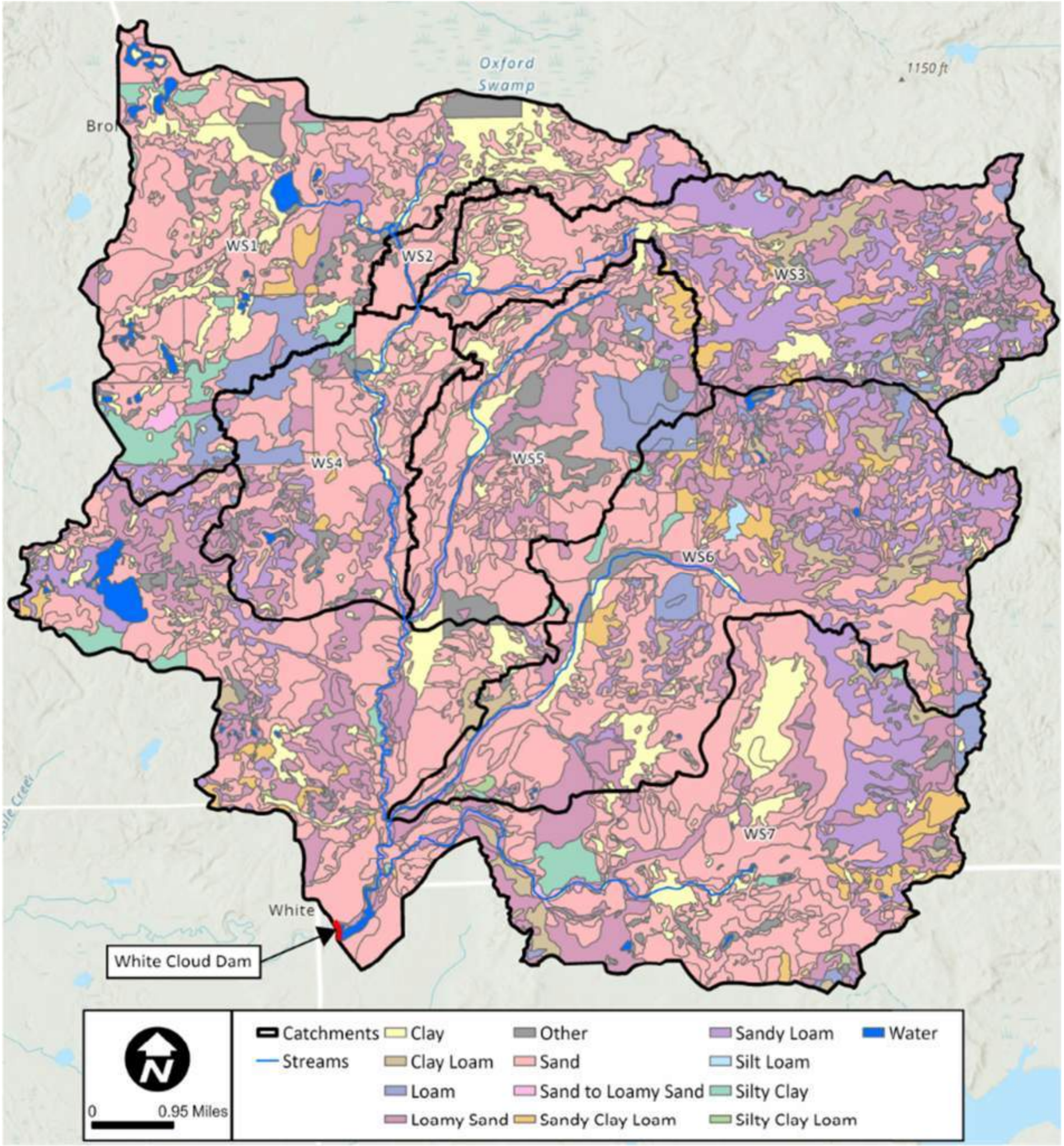
Source: *Soil Textures and Effective Porosity, Wetting Front Suction Head, and Saturated Hydraulic Conductivity* (Rawls, Brakensiek, and Miller, 1983)

The saturated hydraulic conductivity indicates the minimum rate that rainfall will infiltrate the soil column after the soil has been completely saturated. Sand, for example, has relatively large pore spaces which can infiltrate large amounts of water whereas clay has relatively small pore spaces and therefore a low saturated hydraulic conductivity. Wetting front suction head is a measure of how water moves downward through the soil column, with clay having the highest value and sand the lowest. Finally, effective porosity is defined as the amount of interconnected pore volume in the soil.

The combination of these parameters in the Green and Ampt Model allow for the mapping of how much runoff each soil type would generate during storm events. For example, with a low saturated hydraulic conductivity and effective porosity as well as a high wetting front suction head, clay will generate more runoff during a storm event than sand would.

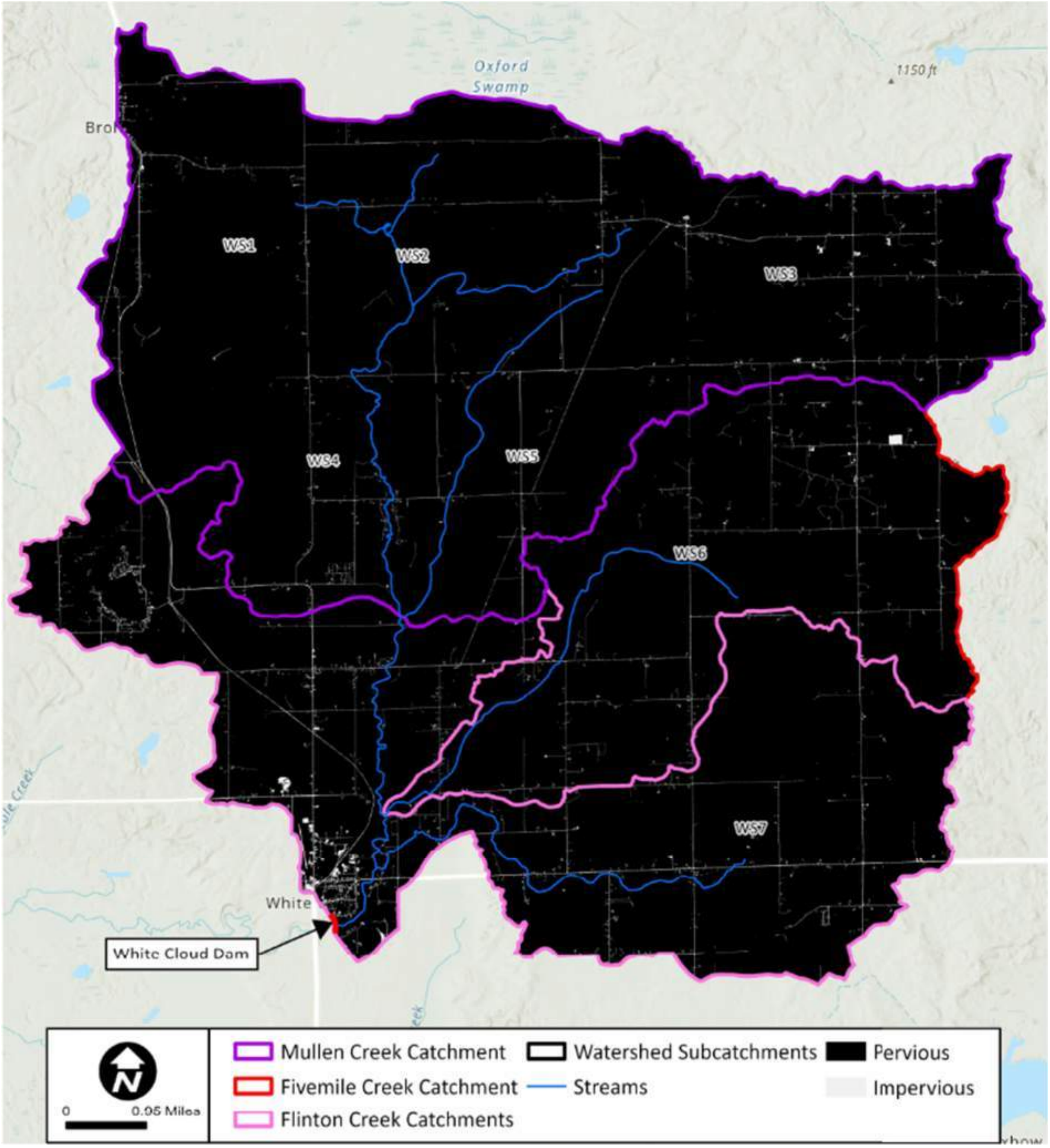
Therefore, as seen in **Figure 2-2**, the site was split into 13 soil types using the USDA's Web Soil Survey to calculate a weighted average of each Green and Ampt parameter by subcatchment. This includes the 11 soil types listed in **Table 2-4** as well as "other" and "sand to loamy sand". The majority of the watershed is sand, loamy sand, and sandy loam. The label "other" denotes non-standard soil textures such as peat or variable soil which does not have a defined set of values for saturated hydraulic conductivity, wetting front suction head, and effective porosity, so this area was not included in the weighted average calculation. The "other" category was found to be between 2.3% to 10.5% of each subcatchment, with the highest being in Mullen Creek due to a higher amount of peat content. Muck was counted as saturated clay for the purposes of this simulation due to their similar characteristics. Finally, sand to loamy sand indicates areas that are split between the two types of soil, so an average of the sand and loamy sand parameter values was used for the Green and Ampt Model.

**Figure 2-2. Soils Map**



The final parameter for this loss method is the percentage of impervious area within the watershed. This characteristic of the watershed was found using NOAA's Coastal Change Analysis Program (C-CAP) impervious data (2020). This dataset offers precise satellite data by state with a resolution of 1 to 2.4 meters. The percentage of open water for each individual subcatchment was calculated from the 2023 National Land Cover Database (NLCD) and was then added to the total impervious with the final values shown in **Table 2-5**.

**Figure 2-3. Land Cover Map**



The results for the weighted average parameters in each subcatchment are shown in **Table 2-5**. Additionally, the percentage of "other" soil types which were excluded from the weighted average calculations for each subcatchment.

**Table 2-5. Soil Data For Green and Ampt Model**

| Catchment Names | Effective Porosity Weighted Average (in3/in2) | Wetting Front Suction Head Weighted Average (in) | Saturated Hydraulic Conductivity Weighted Average (in/hr) | Impervious Land Use (%) | "Other" Soil Types (Peat, Variable, etc.) (%) | Initial Moisture Content (%) |
|---|---|---|---|---|---|---|
| **WS1 - James Creek** | **0.4129** | **4.4499** | **2.9472** | **3.2%** | **6.6%** | **13.0** |
| **WS2 - Post-James Creek** | **0.4186** | **2.1614** | **4.3760** | **1.1%** | **3.5%** | **10.1** |
| **WS3 - Unnamed River** | **0.4035** | **3.9038** | **1.8846** | **1.8%** | **2.9%** | **15.6** |
| **WS4 - South Branch White River** | **0.4132** | **2.9088** | **3.1260** | **1.6%** | **4.4%** | **13.9** |
| **WS5 - Mullen Creek** | **0.4135** | **3.0517** | **3.0845** | **1.2%** | **10.5%** | **15.3** |
| **WS6 - Fivemile Creek** | **0.4031** | **3.5279** | **2.3609** | **1.5%** | **2.3%** | **16.7** |
| **WS7 - Flinton Creek** | **0.4073** | **3.6166** | **2.7234** | **3.7%** | **2.9%** | **18.6** |

## 2.6. Transform Method

After accounting for infiltration losses, the transform method simulates the actual runoff calculations which would occur within the watershed. For this project, the method chosen was the Soil Conservation Service (SCS) Unit Hydrograph Method.

### 2.6.1. *Parameterization of Transform Method*

The SCS Unit Hydrograph Method is based on the Time of Concentration for each subwatershed. Time of Concentration ($T_C$) was calculated for each subwatershed based on the sum of the travel times for sheet flow, shallow flow, and channel flow using TR-55 methods (SCS, 1986).

$$T_C = T_{sheet} + T_{shallow} + T_{channel} \quad [1]$$

The first, sheet flow, is the flow over the land surface. Sheet flow has four parameters seen in **Table 2-6**.

The length of sheet flow was estimated to be around 300 feet for all the subbasins. The land slope was then found by using USGS LiDAR elevation data. The Manning's n value for sheet flow was found by analyzing the land type in Google Earth Pro and estimating the roughness coefficient based on the compiled data from TR-55 Urban Hydrology for Small Watersheds (1986). The majority of the watershed has either dense grasses or wooded areas, resulting in Manning's n values between 0.32 and 0.4. Some subcatchments such as Mullen Creek had a mix of both dense grass and wooded areas around the main reach and so the Manning's n value was an estimated weighted average based on the various land surface types. The 2-year, 24-hour rainfall data was found using the National Oceanic and Atmospheric Administration (NOAA) Precipitation Frequency Data Server. Using these factors, the travel time for sheet flow was computed using the Manning's equation (Overtop and Meadows, 1976).

**Table 2-6. Sheet Flow Parameters**

| Subbasin | Manning's n | Flow Length (ft) | 2-yr 24-hr Rainfall (in) | Land slope (ft/ft) | $T_{sheet}$ (hr) |
|---|---|---|---|---|---|
| WS1 - James Creek | 0.400 | 300 | 2.42 | 0.0125 | 1.20 |
| WS2 - Post-James Creek | 0.400 | 300 | 2.42 | 0.0210 | 0.97 |
| WS3 - Unnamed River | 0.360 | 300 | 2.42 | 0.0080 | 1.31 |
| WS4 - South Branch White River | 0.400 | 300 | 2.42 | 0.0220 | 0.95 |
| WS5 - Mullen Creek | 0.347 | 300 | 2.42 | 0.0080 | 1.28 |
| WS6 - Fivemile Creek | 0.320 | 300 | 2.42 | 0.013 | 0.98 |
| WS7 - Flinton Creek | 0.400 | 300 | 2.42 | 0.0100 | 1.31 |

After approximately 300ft of sheet flow, shallow concentrated flow was assumed to begin. Shallow concentrated flow has three parameters shown in **Table 2-7**. The flow length for shallow flow was estimated to be the distance between the point where sheet flow ends to where the channel begins as mapped on USGS maps or visible in aerial photographs. The shallow channel slope was estimated using the same technique as the sheet flow with the USGS LiDAR elevation data. The average velocity for shallow flow is calculated based on the channel slope. Finally, the time of travel for shallow flow can then be found by dividing the length by the velocity and adjusting the units from seconds to hours.

**Table 2-7. Shallow Flow Parameters**

| Subbasin | Flow Length (ft) | Channel Slope (ft/ft) | Average Velocity (ft/s) | $T_{shallow}$ (hr) |
|---|---|---|---|---|
| WS1 - James Creek | 18958 | 0.002672 | 1.0855 | 4.77 |
| WS2 - Post-James Creek | 5851 | 0.001102 | 0.6973 | 2.21 |
| WS3 - Unnamed River | 38747 | 0.002461 | 1.0418 | 10.25 |
| WS4 - South Branch White River | 6121 | 0.004853 | 1.4629 | 1.11 |
| WS5 - Mullen Creek | 12667 | 0.003276 | 1.2019 | 2.86 |
| WS6 - Fivemile Creek | 35841 | 0.001403 | 0.7866 | 12.55 |
| WS7 - Flinton Creek | 31292 | 0.001359 | 0.7741 | 11.12 |

Finally, channel flow begins after shallow flow with the channel length starting where the channel becomes visible in aerial photograph, or where both USGS topographic data and EPA WATERS data start to indicate the stream by marking it with a blue line. Channel flow has seven major parameters shown in **Table 2-8**. The cross-sectional flow area required multiple steps to find for the streams. First, an approximate top width for the stream was obtained from Google Earth Pro. The channel was then assumed to be trapezoidal with 3:1 horizontal side slopes, except for James Creek, which was assumed to have a 2:1 side slope due to its small width. The depth was estimated based on regional reference curves using **Equation 2** (Stantec, 2015) with the associated drainage area (*DA*) in square miles of the reach that runs through the subcatchments.

$$depth = 1.1 * DA^{0.17} \quad [2]$$

With these components, the bottom width was calculated for each of the streams using the assumed trapezoidal geometry. The cross-sectional flow area was computed using these values. The wetted perimeter, which includes the sides and the base of the channel, was also determined for each of the streams. The hydraulic radius (*R*) is the cross-sectional flow area divided by the wetted perimeter. The channel slope (*s*) is calculated using a similar method to the other flow types with the USGS LiDAR elevation data. Manning's n (*n*) was chosen for the streams based on engineering judgment and Chow, 1959. The velocity (*V*) for the channel was then calculated using **Equation 3**.

$$V = \frac{1.49 * R^{\frac{2}{3}} * s^{\frac{1}{2}}}{n} \quad [3]$$

Similar to the shallow flow travel time calculation, the channel flow travel time can be computed by dividing the flow length by the velocity with a correction for units from seconds to hours.

**Table 2-8. Channel Flow Parameters**

| Subbasin | Flow Length (ft) | Cross-sectional Flow Area (ft$^2$) | Wetted Perimeter (ft) | Hydraulic Radius (ft) | Channel Slope (ft/ft) | Manning's n | V (ft/s) | $T_{channel}$ (hr) |
|---|---|---|---|---|---|---|---|---|
| WS1 - James Creek | 9930 | 10.43 | 10.33 | 1.01 | 0.000895 | 0.040 | 1.12 | 2.46 |
| WS2 - Post-James Creek | 4912 | 11.67 | 12.46 | 0.94 | 0.000583 | 0.045 | 0.77 | 1.78 |
| WS3 - Unnamed River | 18670 | 15.88 | 14.61 | 1.09 | 0.001041 | 0.045 | 1.13 | 4.59 |
| WS4 - South Branch White River | 26582 | 36.29 | 24.01 | 1.51 | 0.000358 | 0.040 | 0.93 | 7.95 |
| WS5 - Mullen Creek | 27233 | 17.92 | 15.88 | 1.13 | 0.000904 | 0.045 | 1.08 | 7.01 |
| WS6 - Fivemile Creek | 32846 | 17.70 | 15.39 | 1.15 | 0.000884 | 0.045 | 1.08 | 8.44 |
| WS7 - Flinton Creek | 32984 | 28.36 | 21.85 | 1.30 | 0.000642 | 0.045 | 1.00 | 9.18 |

The final steps include adding the travel times from sheet, shallow and channel flow to get the total time of concentration (**Table 2-9**). The SCS transform method uses lag time ($T_p$) which is approximately 60% of the time of concentration. The lag time in minutes was then implemented into the HEC-HMS model.

The peak rate factor of 250 was selected based on EGLE guidance for Michigan streams (EGLE, 2010).

**Table 2-9. Time of Concentration**

| Subbasin | $T_{sheet}$ (hr) | $T_{shallow}$ (hr) | $T_{channel}$ (hr) | $T_C$ (hr) | $T_p$ (min) |
|---|---|---|---|---|---|
| WS1 - James Creek | 1.20 | 4.77 | 2.46 | 8.55 | 303 |
| WS2 - Post-James Creek | 0.97 | 2.21 | 1.78 | 5.15 | 179 |
| WS3 - Unnamed River | 1.31 | 10.25 | 4.59 | 16.28 | 582 |
| WS4 - South Branch White River | 0.95 | 1.11 | 7.95 | 10.10 | 360 |
| WS5 - Mullen Creek | 1.28 | 2.86 | 7.01 | 11.25 | 401 |
| WS6 - Fivemile Creek | 0.98 | 12.55 | 8.44 | 22.14 | 791 |
| WS7 - Flinton Creek | 1.31 | 11.12 | 9.18 | 21.77 | 778 |

## 2.7. Baseflow

Baseflow estimates were developed from the EGLE low flow database, which estimates a harmonic mean low flow of 89 cfs. This estimate was distributed to each subwatershed based on relative drainage area, as shown in Table 2-10.

**Table 2-10. Baseflow By Subcatchment**

| Subcatchment Name | Weighted Average Baseflow by Subcatchment (CFS) |
|---|---|
| WS1 - James Creek | 15.2 |
| WS2 - Post-James Creek | 0.8 |
| WS3 - Unnamed River | 11.9 |
| WS4 - South Branch White River | 6.9 |
| WS5 - Mullen Creek | 9 |
| WS6 - Fivemile Creek | 17.6 |
| WS7 - Flinton Creek-South Branch White River | 27.7 |

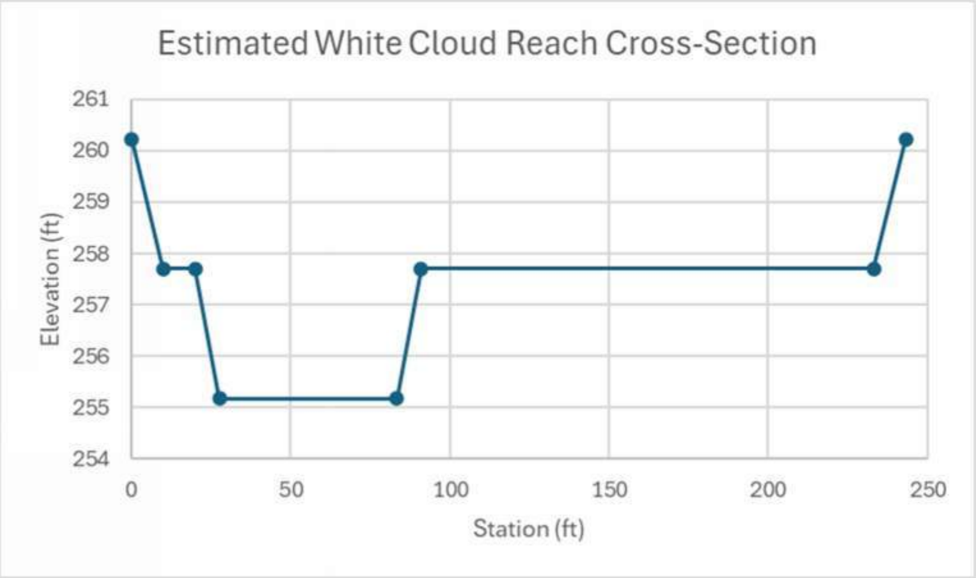
## 2.8. Reach Routing

The Normal Depth method was used to model the travel time through the remaining reaches. This approach was used for the entirety of the South Branch of White River. The other streams have been accounted for using the SCS Unit Hydrograph calculations.

There are six main parameters that are required for the Normal Depth method as seen in **Table 2-11**. The length of each reach was found using the WATERS data on Google Earth. The slope of the reaches in the watershed were found by using the upstream and downstream elevation from USGS LiDAR data and dividing the difference by the total length. The next step was to analyze both the floodplain and stream type from Google Earth Pro to select an appropriate Manning's n for the channels and floodplains (Chow, 1959). For example, Mid-South Branch Reach has medium to dense brush in the summer within the floodplain so the Manning's n value was set to 0.100. The stream seems to be clean and winding with pools which is associated with a Manning's n value of 0.040.

The shape of the stream was represented with an Eight Point cross section as shown in **Figure 2-4**. The shape utilizes the previous calculation from the transform method, such as the depth that came from **Equation 1** and the top width. Additionally, the final size of the floodplain was estimated to be two times the channel based on the available Federal Emergency Management Agency (FEMA) floodplain data. The elevation of the top of the floodplain was approximated using a 1:4 slope. The figure below shows an example floodplain cross-section. Based on the FEMA map, the floodplain for the White Cloud Reach is all on the east side. For the mid- and lower South Branch White River reaches, the floodplain is split evenly on the east and west sides. As these parameters were estimates, different sized floodplains and top of floodplain elevations were tested and were found to have very low sensitivity as it resulted in little to no change in the final flow.

**Figure 2-4. Estimated White Cloud Reach Cross-Section**



The index flow was found to be unique to each storm event as it indicates the maximum expected flow within the reach. The index flow was found using an iterative method of inputting the initial peak flow estimates from EGLE and adjusting based on the resulting modeled peak flows. For example, the 24-hour, 200-year storm event was initially input as 2600 cubic feet per second (cfs) index flow for each reach. After running the model, the resulting peak flows were implemented as the new index flows for each reach. The model was then run again to test the changes and if the modeled peak flow results for each reach were within 0.2 cfs of the initial results, the index flows were finalized in the model. For simulating observed events, a similar method was used, however the starting flow values were GEI estimates instead of EGLE flow estimates. The final parameters for the reach routing method are shown in the table below.

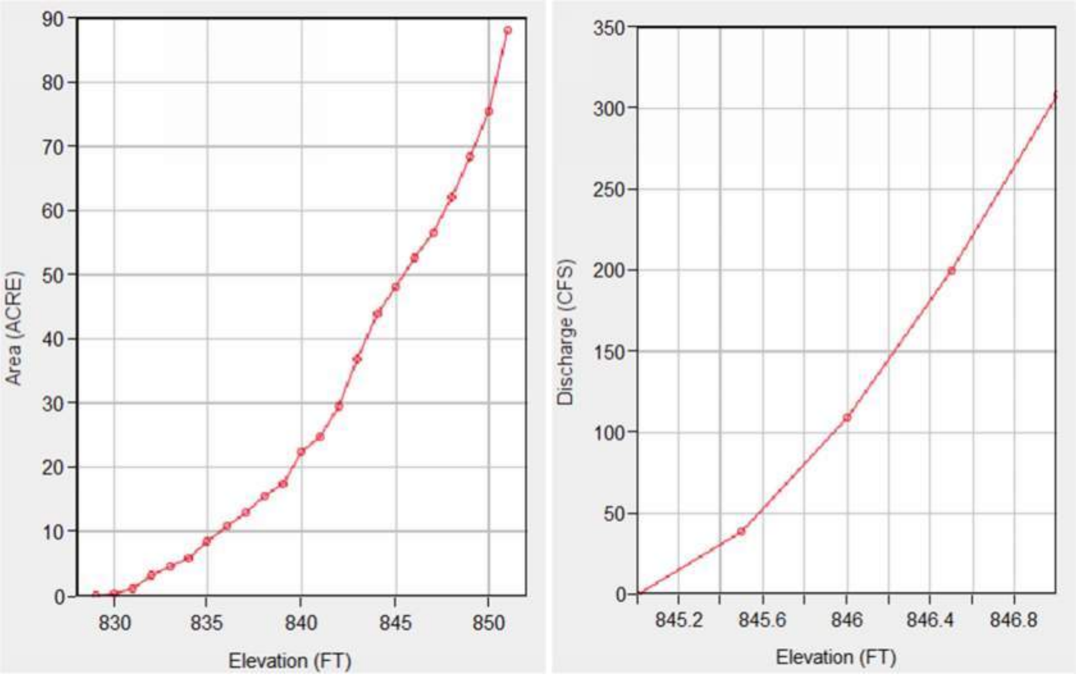
**Table 2-11. Normal Depth Reach Routing Method For A 200-Year Storm**

| Reach Name | Reach Length (ft) | Slope (ft/ft) | Channel Manning's n | Index Flow (cfs) | Shape | Floodplain Manning's n |
|---|---|---|---|---|---|---|
| Post James Creek – South Branch White River | 4912 | 0.000583 | 0.045 | 784.2 | Eight Point | 0.150 |
| Upper South Branch White River | 26582 | 0.000358 | 0.040 | 1285.6 | Eight Point | 0.150 |
| Middle South Branch White River | 16649 | 0.000424 | 0.040 | 1879.4 | Eight Point | 0.100 |
| Lower South Branch White River | 4889 | 0.000234 | 0.040 | 2377.2 | Eight Point | 0.060 |
| White Cloud Dam – South Branch White River | 923 | 0.000623 | 0.030 | 3156.6 | Eight Point | 0.060 |

## 2.9. Bathymetry and Combined Spillway Data

A bathymetry survey was conducted by GEI for the Lake White Cloud Impoundment in October 2023. Using Arc-MAP and the survey data, the acreage for each elevation at one-foot intervals was calculated. These values were then used to create a stage-storage curve seen in **Figure 2-5** for the model. Additionally, the combined spillway discharge curve was developed by calculating the amount of discharge through the primary and auxiliary spillways at 0.5-foot elevation intervals up to the RCC elevation using the weir equation and assuming normal stoplog elevation. These two features were then implemented into the dam component of the model to simulate the water surface elevation within the impoundment at each timestep.

**Figure 2-5. White Cloud Dam Stage-Storage Curve (Left) and Combined Spillway Discharge Curve (Right)**



## 2.10. Validation Approach

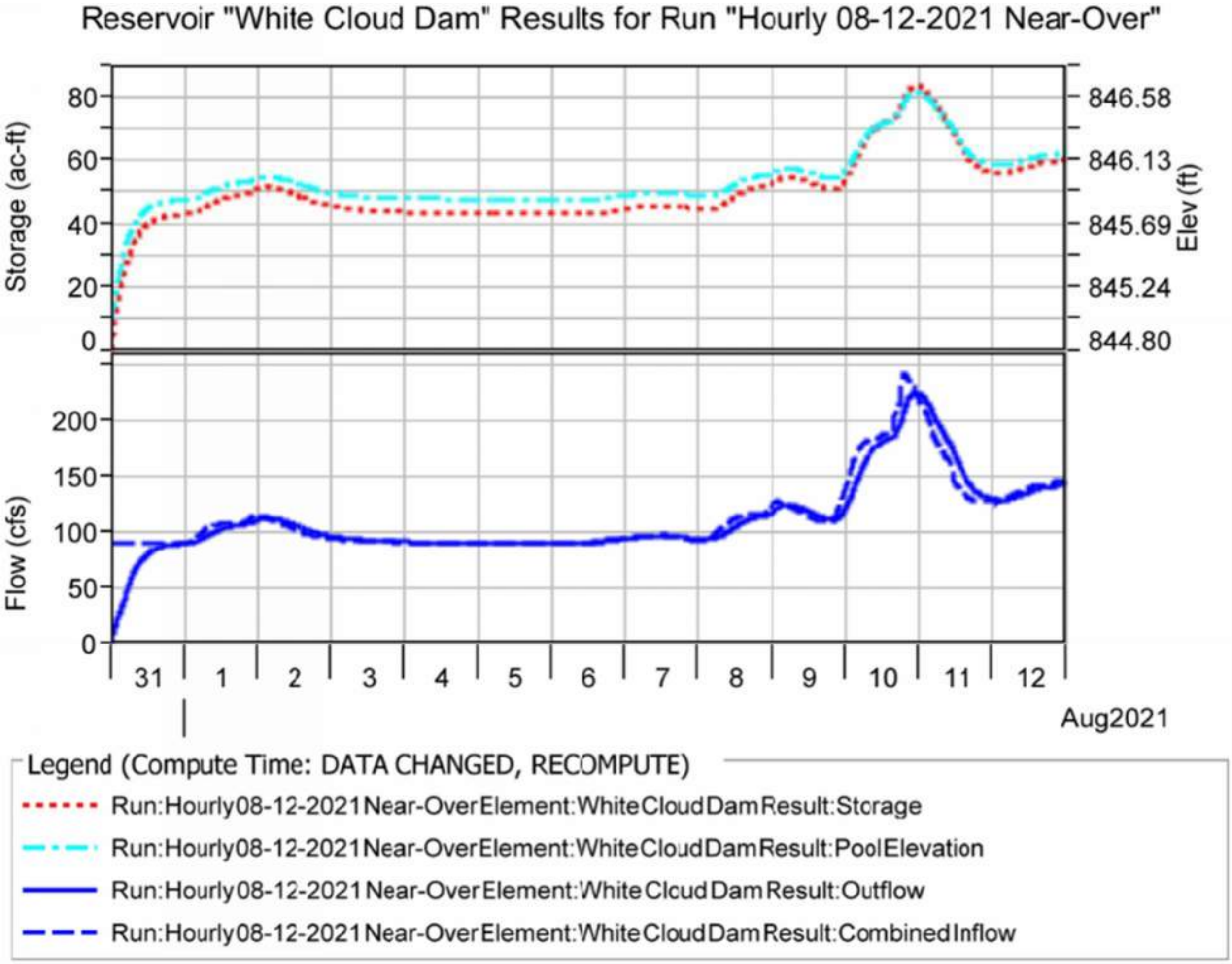
The validation approach was to use the rainfall data from the gauges during known near-overtopping and overtopping events to generate simulated runs of the watershed. As the August 10th, 2021 event has the most data available to use, it was used as the primary validation event for the model. During this event, the total rainfall measured at the Weather Underground gage was approximately 3.3 inches over 2 days, with approximately 2.5 inches falling within 24 hours, and the water surface elevation in the reservoir nearly reached the 847’ RCC spillway. As such, the simulation using the hourly gauge data and model parameters should give an output that nearly reaches an elevation of 847’. For all the other comparison events, the dam was overtopped meaning that the simulation should give an output that is over the 847’ elevation during the peak of the event. This approach was used to fine tune the model’s parameters so that all the known events gave results that would align with the known conditions of the dam.

## 2.11. Final Validation Results

The daily time-step validation events resulted in lower than observed water surface elevations. This was assumed to be due to the daily time-step data not representing the maximum rainfall intensities observed during those events. The daily time-step rain gage was also located outside of the watershed, so the rainfall totals within the watershed may have been slightly different. Hourly time-step data should better represent the maximum rainfall intensities that drive extreme events, and the hourly gage was located within the watershed. The hourly August 10th, 2021 near-overtopping event had a modeled peak inflow of 240.6 cubic feet per second (cfs) and a final modeled peak elevation of 846.6’ as shown in

**Figure 2-6**. As the RCC spillway invert is 847', this shows that the model is reasonably simulating the observed near-overtopping event.

**Figure 2-6. Hourly August 10th, 2021 Near-Overtopping Event**

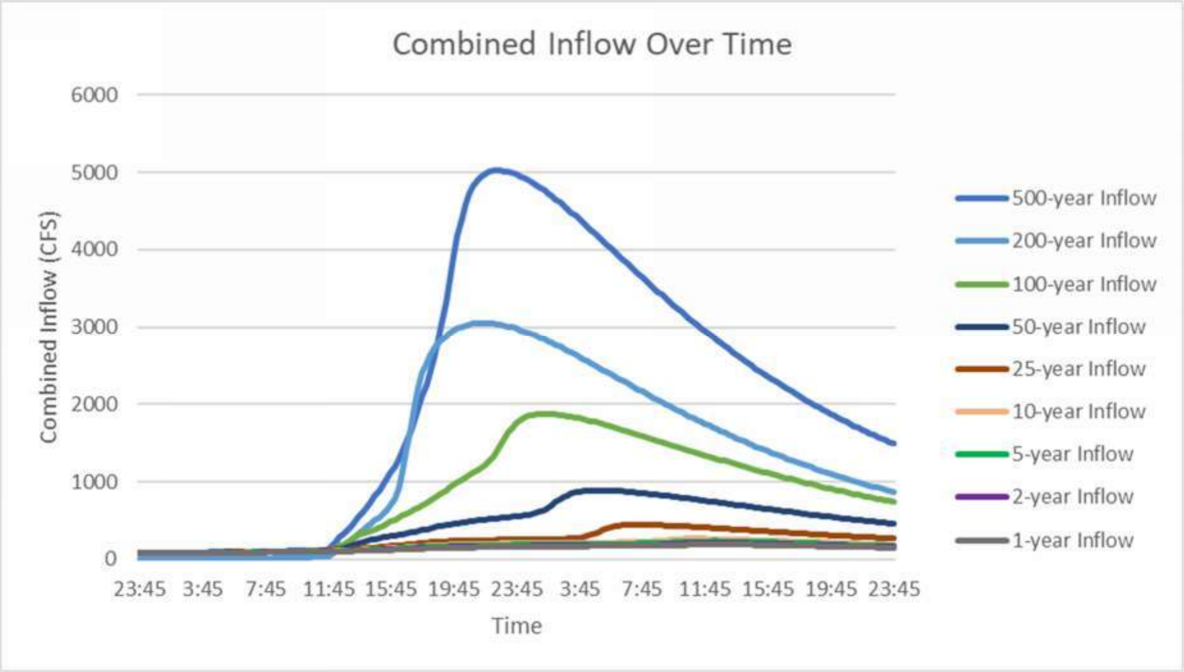


# 3. Hydrology Model Results

## 3.1. Storm Event Peak Flows

Using the methodology and probability storm events discussed in the model development section, the 24-hour events for the 1-, 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year probability storms were simulated. For all events, the HEC-HMS model was set to simulate 48 hours, with the first 24 hours being the rainfall in the storm event and the second 24 hours simulating the excess runoff generated. An example of the results generated in a graph format can be found in **Figure 3-1**. Additionally, it was assumed for all the results that inflow would equal discharge.

**Figure 3-1. Simulated hydrographs For All 24-Hour Storm Events**



EGLE has available peak flow data for the White Cloud dam that were generated through a regression equation. The difference between the HEC-HMS simulated peak flows and the EGLE data are provided in **Table 3-1.** The peak flows simulated in HEC-HMS are expected to be based on a more detailed representation of watershed characteristics and hydrologic processes than those developed from regression equations. These storm event hydrographs were then used to conduct the hydraulic analysis, which is described in the next section of this report.

### Table 3-1. Model Results Compared to EGLE Peak Flow Values

| Storm Event | 24-Hour Rainfall (inches) | EGLE Flow (CFS) | HEC-HMS Flow (CFS) | Difference (CFS) |
|---|---|---|---|---|
| 1-year | 2.15 | -- | 192 | -- |
| 2-year | 2.42 | -- | 205 | -- |
| 5-year | 2.98 | -- | 234 | -- |
| 10-year | 3.56 | 1200 | 267 | 933 |
| 25-year | 4.52 | -- | 446 | -- |
| 50-year | 5.41 | 1900 | 891 | 1009 |
| 100-year | 6.41 | 2300 | 1878 | 422 |
| 200-year | 7.55 | 2600 | 3157 | -557 |
| 500-year | 9.24 | 3200 | 5022 | -1822 |

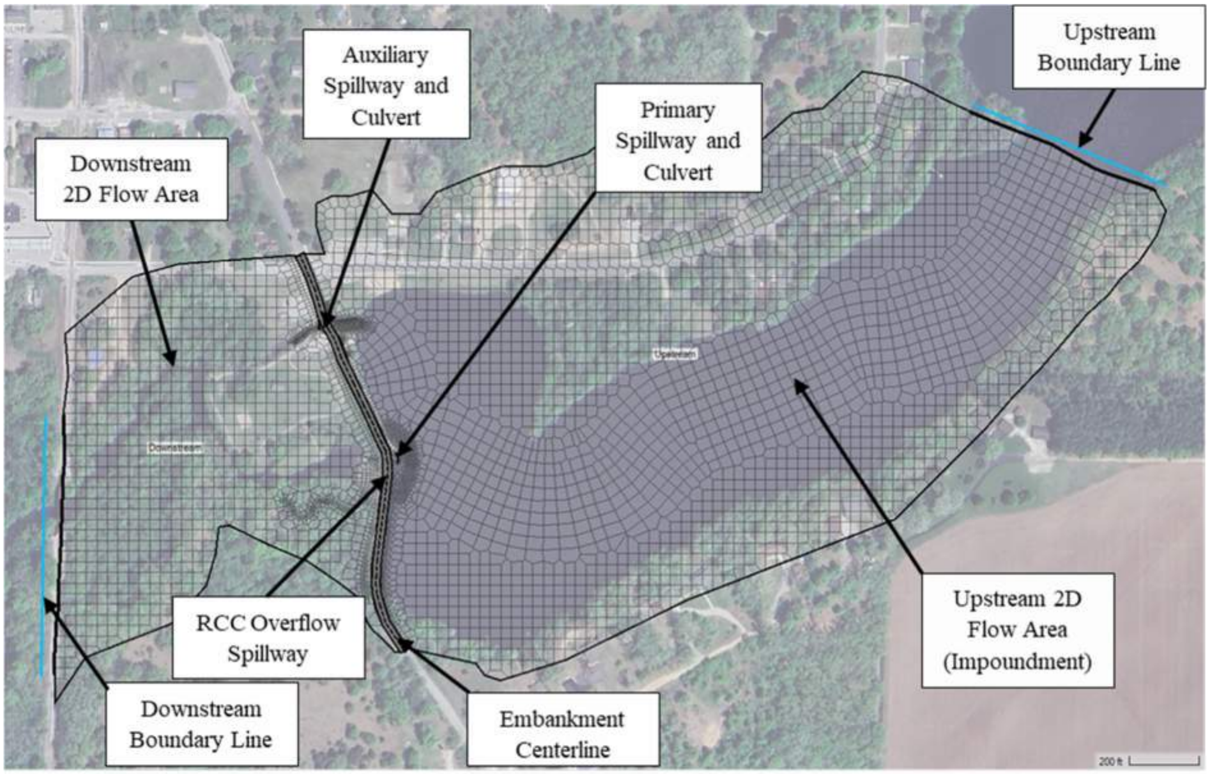
# 4. Hydraulic Model Development

The purpose for the hydraulic model is to evaluate the capacity of the existing dam structure in the 200-yr flood scenario, which is the design flood event for the dam, and evaluate possible stoplog management options for storm events. The following sections describe the hydraulic modeling methods and results.

## 4.1. Existing Conditions Model

EGLE provided GEI with a HEC-RAS version 6.2 model originally developed by Holland Engineering, formerly OMM, in Summer 2023. The model was originally developed to meet the 2022 EGLE dam safety inspection requirement for updated hydraulic capacity calculation for the Dam. The model was then modified by GEI Consultants to improve the accuracy and stability of the model for use in hydraulic evaluation of the structure. The model was updated to HEC-RAS version 6.5. **Figure 4-1** illustrates the model setup.

**Figure 4-1. HEC-RAS Model Setup**



## 4.2. Boundary Conditions

The model upstream boundary is located approximately 2,200 feet upstream of the dam. This was determined by OMM Engineering in the initial model creation. The boundary condition at this location is the inflow hydrograph.

The model downstream boundary is located approximately 950 feet downstream of the dam. The boundary is set at the railroad crossing over the White River to the west and is set as a normal depth using a slope of 0.01 ft/ft.

## 4.3. 2D Flow Areas

The original model from OMM Engineering represented the impoundment as a 1D storage area and the downstream section as a 2D flow area. To improve accuracy and ensure proper representation of the upstream conditions, especially those nearest the embankment, GEI converted the 1D storage area upstream into a 2D flow area.

## 4.4. Structures

The primary spillway, auxiliary spillway, and the road embankment including the RCC overflow spillway and floodwall were modeled as Storage Area/2D connections, which connect the upstream 2D area to the downstream 2D area using the weir equation. The primary spillway and auxiliary spillway were modeled with a  weir coefficient = 3.2 to represent the stop logs as a sharp crested weir. The primary and auxiliary spillways were modeled as “open air gates” to allow for multiple stop log configurations to be tested (**Figure 4-2**). All slide gates were assumed to be raised well above the water level so as not to restrict flow during the entire duration of storm events simulated. The road embankment including the RCC was modeled with a weir coefficient = 2.6 to represent a typical broad crested weir (**Figure 4-3**). Each of these structures were originally defined by OMM Engineers using existing structure drawings as well as site survey data. Following additional survey collection in 2024, GEI determined that the 1989 as-built drawings for the dam appear to be in NGVD29 vertical datum, and as a result, converted the elevations from the drawings to NAVD88 using the following datum transformation: NAVD88 = NGVD29 -0.37ft. GEI also updated the floodwall elevations based on the 2024 survey data. There is a gap in the flood wall to the left of the auxiliary spillway for a boat launch into the impoundment. The Dam’s Emergency Action Plan states that sandbags are to be installed at the boat launch during flood events to prevent overtopping, so the sandbags were assumed to be in place for all model runs.

**Figure 4-2. Schematic View of the HEC-RAS Primary (top) and Auxiliary (bottom) Spillways with All Stoplogs Removed**

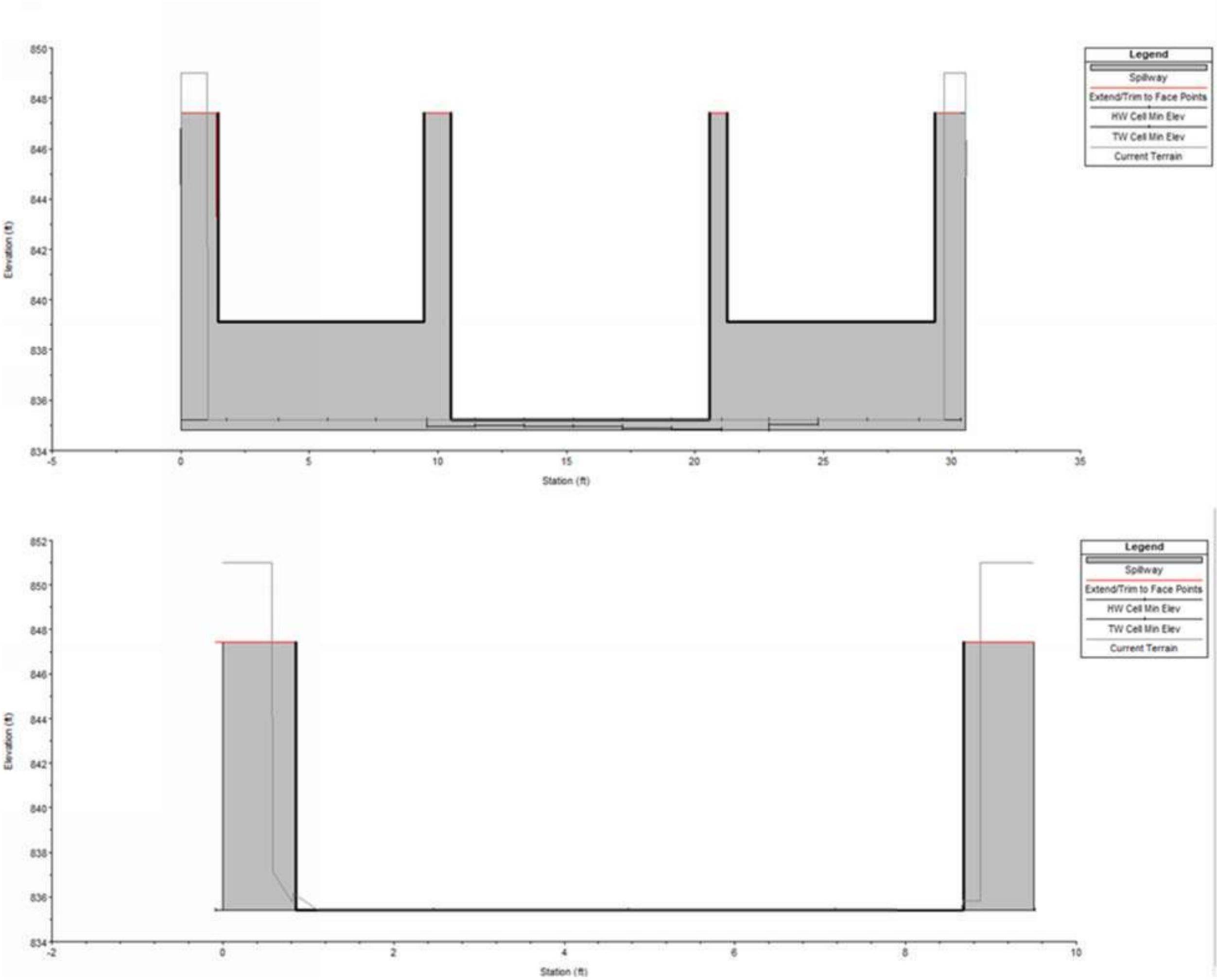
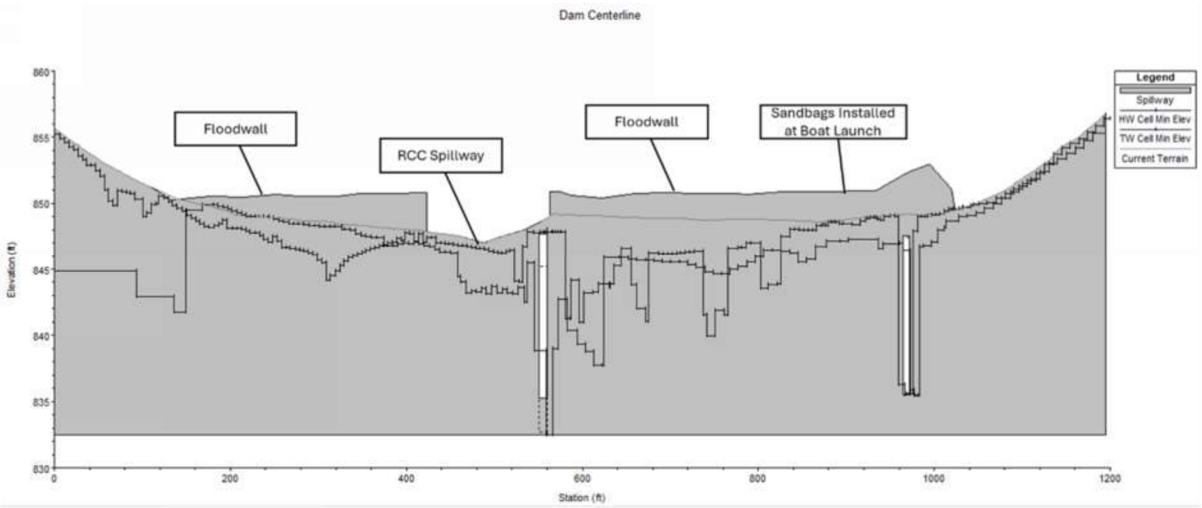


**Figure 4-3. Schematic View of the HEC-RAS Dam Centerline**



## 4.5. Flow Hydrographs

The 24-hour storm event hydrographs developed in HEC-HMS and described earlier in this report were used as the input flow hydrographs in the HEC-RAS model.

## 4.6. Computational Methods

Because both the upstream and downstream areas of the model are represented as 2D flow areas, all simulations were run with an unsteady flow hydrograph and a variable timestep controlled by the Courant condition, which balances model stability with computation time. In model setup and troubleshooting, it was determined that the downstream flow area be modeled using diffusion wave equations, and the upstream flow area be modeled using the Shallow Water Equations – Eulerian-Lagrangian Method (SWE-ELM). This was done to maximize model stability and accuracy on the upstream side of the dam, while reducing simulation times.

## 4.7. Scenarios

The model was run under several storm event scenarios. The first set of scenarios assumed that the impoundment was at normal summer pool elevation (845’) at the beginning of the event and the stoplogs were at their normal level and were not manipulated during the event. The second set of scenarios assumed that all stoplogs were removed before the water level in the impoundment began to rise, which effectively lowered the impoundment 1-ft below summer pool elevation before the storm began. The third set of scenarios tested various stoplog manipulations during a 10-year event to prevent overtopping the RCC spillway. The final set of scenarios tested various stoplog manipulations during 25-year and 50-year events to prevent overtopping the boat launch (without sandbags installed). All scenarios assumed that the slide gates were completely raised for the duration of the storm event. All scenarios assumed that sandbags were effectively installed at the boat launch located on the right embankment up to the flood wall elevation. All scenarios assumed the auxiliary spillway walls between the road and the pedestrian bridge were raised to elevation 851’. GEI understands this work to raise the auxiliary spillway walls was completed in 2024.

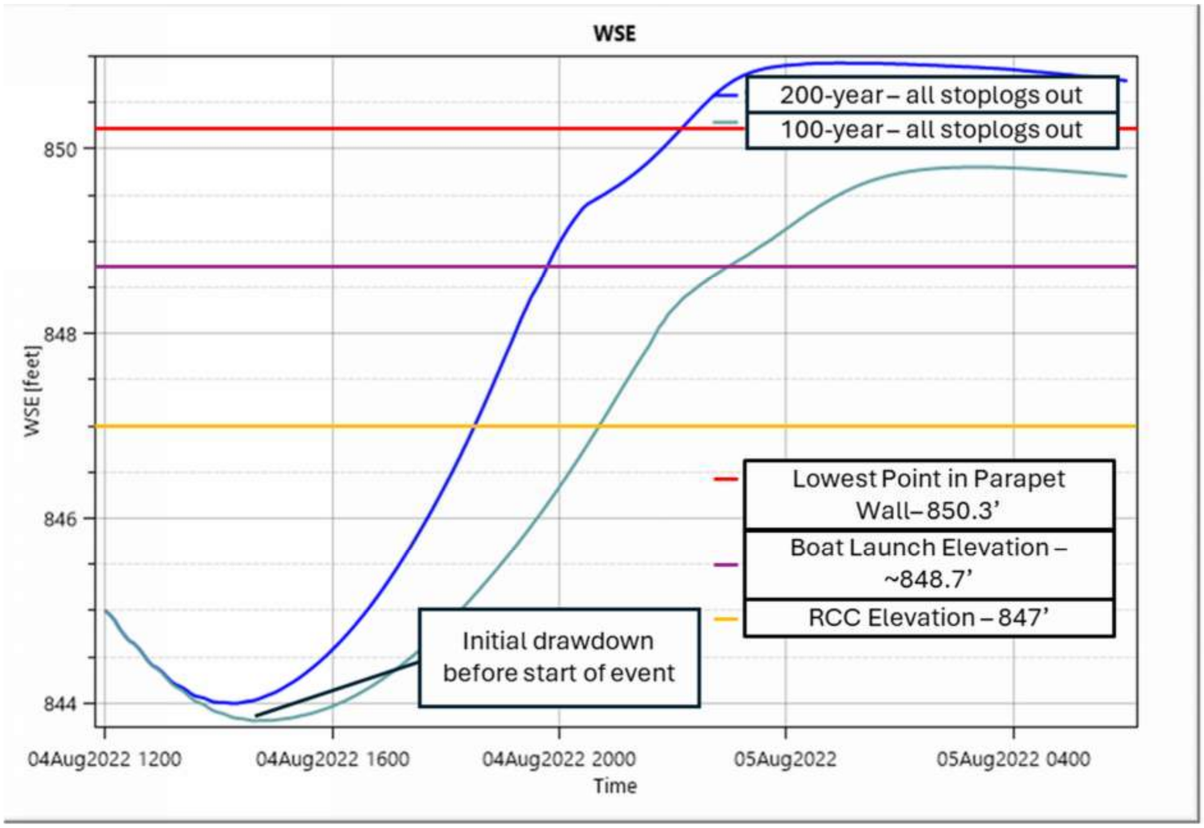
# 5. Hydraulic Model Results

The existing conditions model was run with various flood event hydrographs. Generally, the water surface computation errors were low, however the largest events including the 50-year, 100-year and 200-year produced higher water surface computation errors during the rising limb of the event. These errors occurred after the RCC was overtopped but before the peak of the event, so these errors likely did not affect the final maximum water surface results.

The results of the hydraulic analysis show that the floodwall, which is 850.3’ at the lowest point, was overtopped during the 200-yr storm event regardless of stoplog operation, even when all stoplogs were removed before the flood hydrograph arrived, which effectively lowered the pond 1-ft below the typical summer pool (845’) before the storm arrived (see **Figure 5-1**).

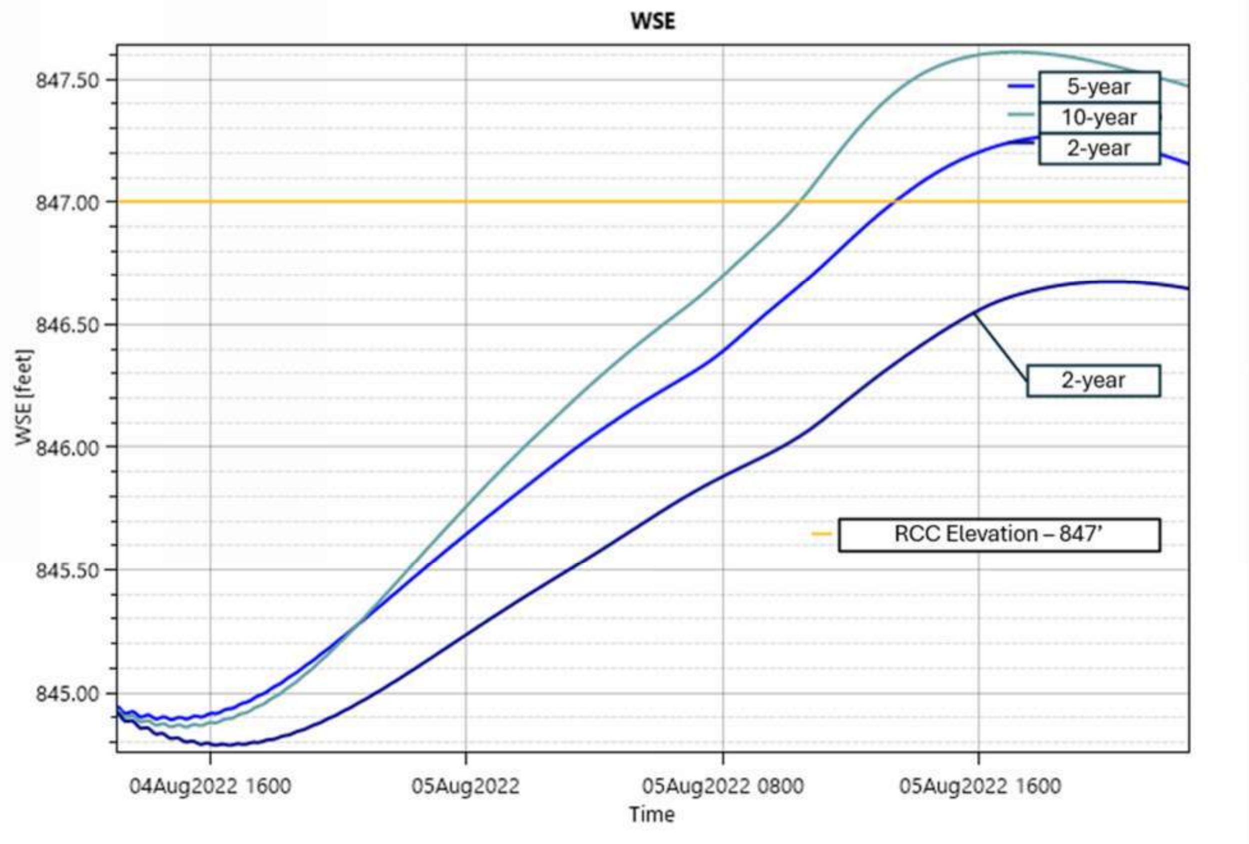
The simulated 100-yr storm event overtopped the RCC spillway and had minimal freeboard (0.5’) to the floodwall when all stoplogs were removed before the flood hydrograph arrived, which effectively lowered the pond to 1-ft below the summer pool before the storm arrived.

**Figure 5-1. 100-year and 200-year Simulated Water Levels with Stoplogs Out**



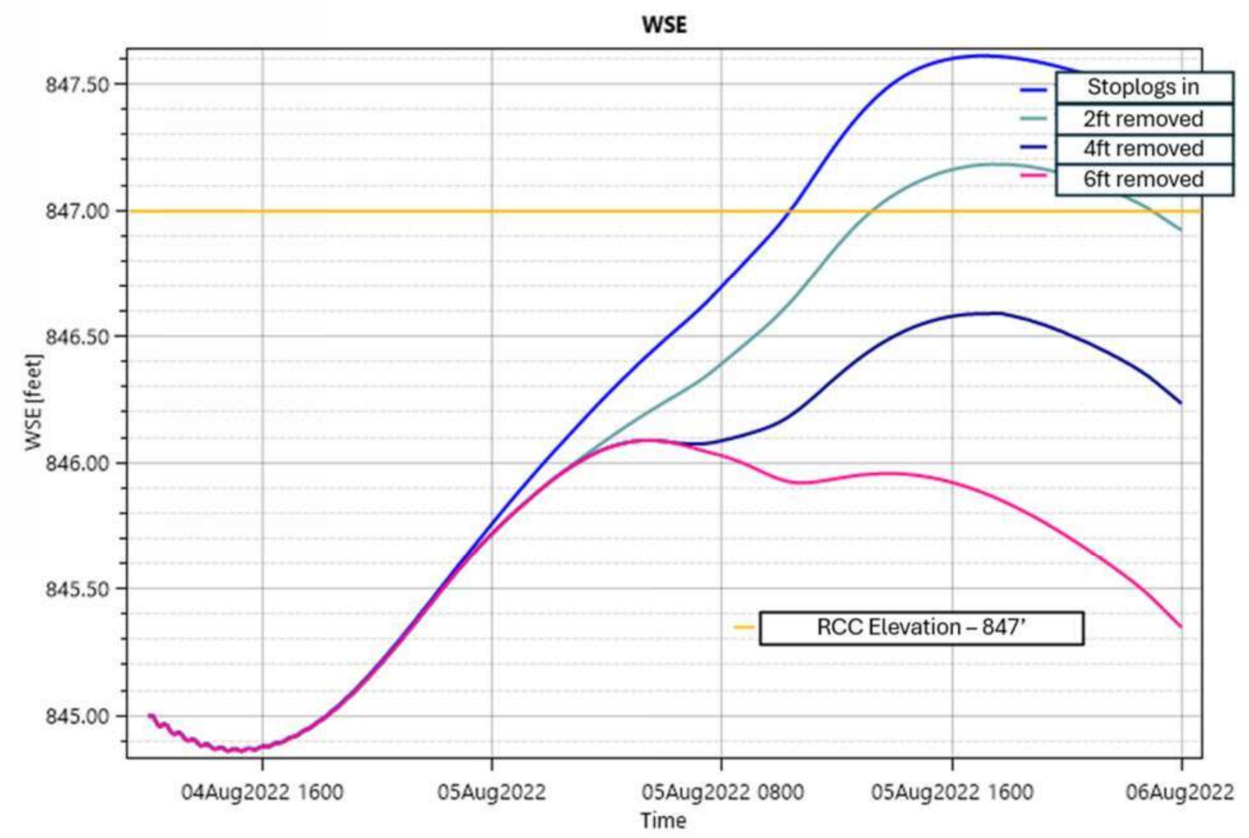
At normal summer pool levels, with no manipulation of stoplogs before or during the event, the simulated 2-year event had approximately 0.3’ of freeboard to the RCC spillway, and the 5-year and larger events overtopped the RCC spillway (**Figure 5-2**).

**Figure 5-2. 2-Year, 5-Year, and 10-Year Simulated Water Levels with Stoplogs In**



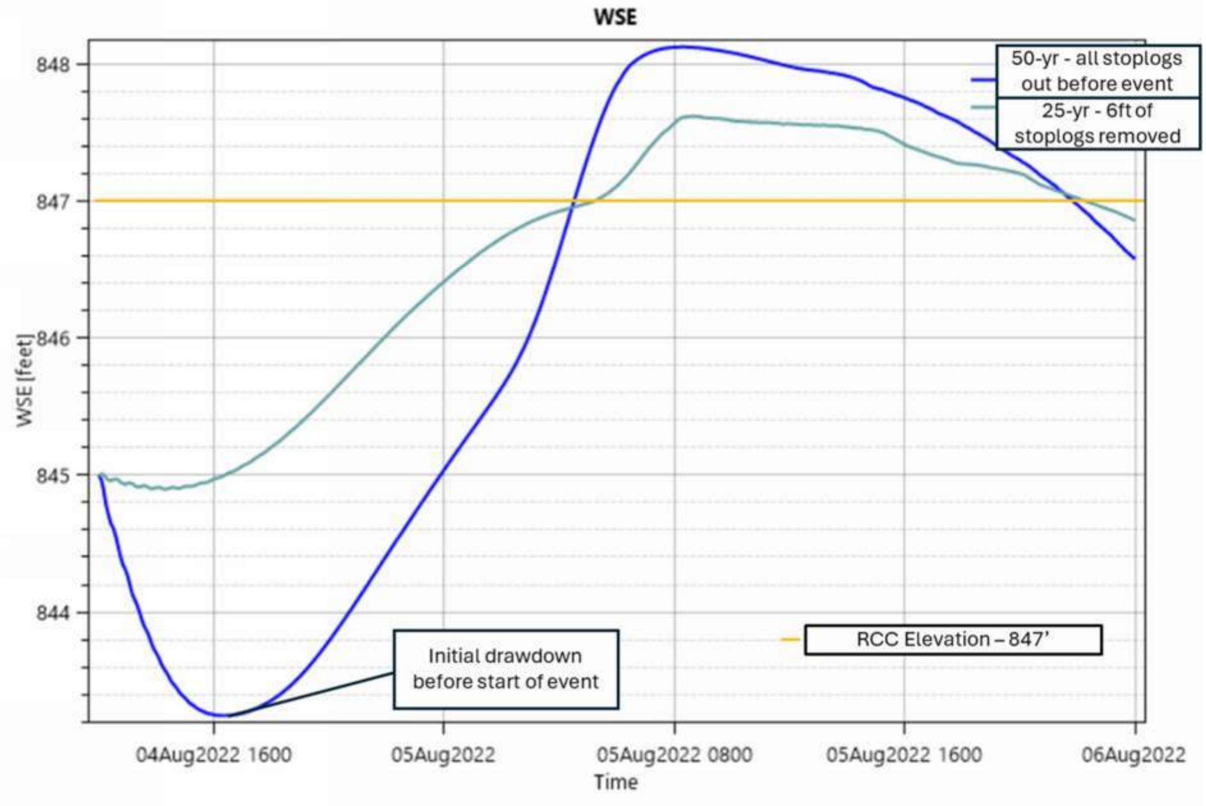
To prevent activation of the RCC spillway during these smaller events, the stop logs must be manipulated during the storm event. The 10-yr event was prevented from overtopping the RCC spillway (with 0.4' freeboard) by removing a total of 4 feet of stoplogs from both sides of the auxiliary spillway and all three primary spillway gates at a rate of one 6-inch board per hour from each gate after the reservoir rose 0.5' above normal summer pool at the beginning of the event (**Figure 5-3**). Removing 6 feet of stoplogs during the 10-yr event resulted in 0.9' freeboard to the RCC spillway.

**Figure 5-3. Stoplog Scenarios for 10-Year Event**



Storm events larger than a 50-year event will require sandbags to be installed at the boat launch to prevent overtopping regardless of stoplog manipulation, since the boat launch is overtopped at approximately 848.7' (Figure 5-1). Stoplogs should be removed for 50-year events and smaller to prevent overtopping at the boat launch (Figure 5-4).

**Figure 5-4. Stoplog Scenarios for 25-Year and 50-year Events to Prevent Overtopping at the Boat Launch**



An important note is that all storm hydrographs developed in HEC-HMS assumed normal antecedent conditions leading up to the modeled event. In other words, the watershed soils were not completely saturated or completely dry. If conditions leading up to an actual real-world event had saturated soils from recent rainfall, the incoming hydrographs would be larger than those shown here, and additional stoplogs should be removed.

The slide gates at both the primary and auxiliary spillways were assumed to be kept fully open and above the water level for the entire duration of all flood events so as to not restrict flows. The White Cloud dam operations manual states that the slide gates can restrict the flow during a flood and allow the stoplogs to be accessed for removal or replacement. Based on the results of this hydraulic analysis, it is recommended to only lower the slide gates briefly if needed to remove stoplogs and then raise the gates above the water level again after stoplogs have been removed.

Based on these results, the White Cloud Dam does not appear to have adequate spillway capacity for the 200-year event. Even with all stoplogs removed prior to the start of the event, there is a risk of overtopping the floodwalls. The impoundment should be managed in part by using weather forecast information. Forecasted 10-year rainfall events and smaller can be managed by removing at least 4 to 6 feet of stoplogs as soon as the impoundment rises approximately 0.5' as shown in Figure 3-3 to avoid overtopping the RCC spillway. Forecasted events larger than the 10-year should be managed by removing all stoplogs before the impoundment begins to rise to prevent or minimize the amount of floodwall overtopping.

# 6. Summary

The hydrologic modeling provides a more detailed analysis of flood flows coming into the White Cloud Dam impoundment and differs from the existing EGLE peak flow estimates. The differences result in lower flows for the 10-year – 100-year recurrence interval events, but a larger peak flow for the 200-year Part 315 regulatory event.

The updated hydraulic modeling with the updated flood flows from the hydrologic model show that the dam can safely convey up to the 2-year flood event without activating the RCC Spillway.

For flood events larger than the 2-year event, the stop logs at the primary and auxiliary spillway must be manipulated to prevent activation of the RCC spillway and/or uncontrolled overtopping of the dam.

Sandbags should be installed at the boat launch are required for storm events larger than a 50-year event because, regardless of stoplog manipulation, storm events larger than the 50-year event will overtop the boat launch at approximately 848.7’. For storm events less than the 50-year event, the stoplogs at the primary and auxiliary spillway should be manipulated to prevent impoundment levels from overtopping the boat launch.

Even with full removal of the stoplogs from the primary and auxiliary spillway prior to the flood hydrograph arrival, the dam’s parapet wall is overtopped during the 200-year design event and either the floodwall needs to be raised or the hydraulic capacity of the dam’s structures need to be increased.

# 7. References

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# Appendix F – Geotechnical Analysis

Consulting
Engineers and
Scientists

### Geotechnical Data Report

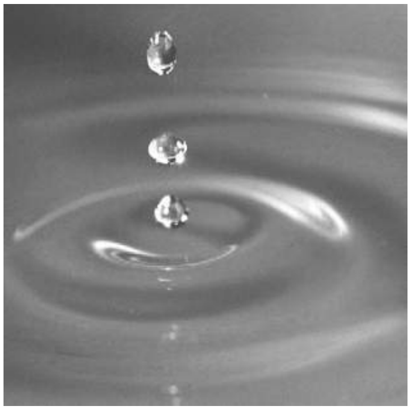
# White Cloud Dam

Newaygo County, Michigan
Dam I.D. No. 526

**Prepared for:**
**City of White Cloud**
12 N. Charles Street
White Cloud, MI 49349

**Prepared by:**
GEI Consultants of Michigan, P.C.
109 W. Baraga Avenue
Marquette MI, 49855

January 31, 2025
GEI Project 2302435



Morgan E Carden
Morgan E. Carden, P.E.
Project Engineer

Micheal D. Carpenter, P.E.
Vice President

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## Tables

## Figures

## Appendices

# 1. Introduction

GEI Consultants of Michigan, P.C. (GEI) contracted with the City of White Cloud, Michigan, to provide a Geotechnical Data Report to better understand the subsurface conditions at the White Cloud Dam. This report presents the results of our field explorations and laboratory testing. A Site Location Diagram is included as **Figure 1**.

## 1.1 Background and Site History

The White Cloud Dam, in White Cloud, Michigan, is a 950-foot-long and roughly 19-foot-high earth fill embankment with three concrete spillways. It creates a head of 14.4 feet and impounds about 50 acres. South State Street runs along the crest of the dam. The primary spillway is located near the center of the embankment with three lift gates on the upstream side. The center gate is 11 feet wide and the two side gates are 9 feet wide. These gates have both slide gates and stop log bays at each opening. Downstream of the gates, flow passes through a 10-foot-wide chute under the road and down to the river. To the left of the principal spillway is a 140-foot-long overflow spillway constructed of roller compacted concrete (RCC). An 8-foot-wide auxiliary spillway is located near the right abutment of the dam. This auxiliary is controlled by a gate on the downstream side of the roadway (crest) before discharging to a chute conveying flow down to the river. A parapet wall has been constructed along the upstream side of the crest to prevent overtopping of the dam during flood events. There is a break in the parapet wall at each of the spillways and immediately left of the auxiliary spillway for a boat launch into the impoundment. During flooding events, sandbags are installed at the boat launch to prevent overtopping. Figure 1 shows many of these features.

The dam was originally constructed in 1872. In 1910, the dam was destroyed by a flood and was reconstructed the same year. The embankment crest was later increased by 3 feet in 1975, and the auxiliary spillway was added in 1978. During the major floods in September 1986, the dam failed again. It was reconstructed again in 1990 with the addition of the RCC overflow spillway for increased spillway capacity.

In recent years, numerous deficiencies have been identified during dam safety inspections by the Michigan Department of Environment, Great Lakes, and Energy (EGLE) and structural inspections by the City's former engineer (OMM, Inc.). These deficiencies include, but are not limited to, uncertainty about spillway capacity, concrete and steel deterioration, seepage on the downstream slope of the right embankment, and woody vegetation on the embankment. Additionally, there were several overtopping events reported along the auxiliary spillway and a failure of stop logs on the primary spillway. These deficiencies have led to the dam receiving a poor condition rating from EGLE Dam Safety.

Given the poor condition rating and the challenges of maintenance and upkeep on the dam, the City has begun having public discussions about the long-term disposition of the dam and trying to understand the risk, liability, and financial impacts. In 2022, Trout Unlimited prepared a preliminary dam removal feasibility study. This study explored the design considerations associated with dam removal and river restoration and provided perspectives for restoration aesthetics and options for recreation and natural resource improvements. Unfortunately, this document did not provide financial evaluations for dam-in or dam-removal scenarios.

## 1.2 Purpose

The purpose of this subsurface exploration program is to provide subsurface soil and groundwater information to for an alternatives analysis and support potential repair, reconstruction, or removal design options for the White Cloud Dam site.

## 1.3 Data Review

The following reference documents and information were reviewed during the writing of this report:

- Construction Plans for White Cloud Dam Reconstruction, Olson, Meyers & May, Inc., December 1989.
- Dam Safety Inspection Report: White Cloud Dam – Dam ID No. 526, Lucas A. Trumble, P.E., Michigan Department of Environment, Great Lakes, and Energy, December 22, 2022.

Pertinent historical information related to this Geotechnical Data Report can be found in **Appendix D**.

## 1.4 Scope of Work

GEI performed the following tasks for this Geotechnical Data Report:

- Reviewed available site data related to site structures, soil, and groundwater conditions.
- Engaged Pearson Drilling, Inc. (Pearson) as a drilling subcontractor to drill three borings along the crest of the dam.
- Performed geotechnical laboratory testing on soil samples obtained from the borings to estimate the parameters of the soils for use in proposed future analyses.
- Prepared this Geotechnical Data Report.

## 1.5 Elevation Datum, Horizontal Coordinates & Stationing

Elevations cited in this report are in feet and are referenced to the North American Vertical Datum of 1988 (NAVD88) unless otherwise noted. The horizontal coordinate system used for this project is the Michigan State Plane coordinate system (MI-83CIF) which is based on the North American Datum of 1983 (NAD 83). A new stationing system was established for this project; see **Figure 1** for more detail.

# 2. Subsurface Exploration and Testing Procedures

The 2024 subsurface exploration program included advancement of geotechnical soil borings at three locations along the crest alignment of the dam. The borings were advanced from the crest, through the existing asphalt pavement, and through the roller compacted concrete (RCC) spillway at the left embankment.

## 2.1 Boring Selection and Layout

Boring locations were selected based on accessibility and to sample and characterize the various strata of geotechnical interest. Borings are identified as B-01 through B-03 for the embankment Borings. Information about the proposed borings is shown in Table 3-1, including proposed depths and ground surface elevations. The final boring locations are illustrated on **Figure 1**.

## 2.2 Drilling and Sampling

Pearson mobilized an Acker Renegade track mounted rig. Borings were started, completed, and backfilled on October 23, 2024. Borings were advanced using hollow-stem augers (HSA) in general accordance with the recommendations in U.S. Army Corps of Engineers Engineering Manual ER 1110-1-1807 (USACE, 2014). Borings were sampled using a conventional split-barrel sampler at 2-foot intervals from the surface through the embankment material then at 5-foot intervals thereafter to the boring completion depth. Split spoon samples were performed in accordance with ASTM D1586 “Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.” Blow counts were recorded during advancement of the split spoon. Fill and native soil samples recovered during drilling were classified in the field by a geotechnical professional. Samples were retained in jars supplied by Pearson. All jar soil samples collected during this field exploration were transported to the GEI laboratory in Marquette, Michigan, for laboratory testing.

## 2.3 Boring Logs

Field boring logs were completed by GEI personnel. Information collected on the field logs included:

- Project name, number, and location.
- Boring name and start and completion dates.
- Driller and drilling company identification and equipment used.
- Sample identification, depth, and type (split spoon or Shelby tube).
- Sample recovery and blow counts.

- Sample description (field description augmented by laboratory test results) and corresponding Unified Soil Classification System (USCS) designation.
- Depth of water encountered during sampling.

Boreholes were tremie-filled with a cement/bentonite grout mixture and asphalt road surface was restored after borings were completed. Borehole abandonment details are provided on the boring logs in **Appendix A.1**.

Upon completion of the field work, the logs were entered into gINT software to standardize the collected information into a presentable graphic form. Copies of the completed boring logs are included in **Appendix A.1**.

## 2.4 Laboratory Testing

Index testing on selected soil samples from the borings were performed to evaluate index properties for classification purposes and to evaluate the visual descriptions of the soils classified. Soil samples were tested at GEI's Marquette, Michigan, geotechnical laboratory. Results of the analyses are discussed in **Section 3** of this report. Tests included:

- Soil Classification per ASTM D2487
- Moisture Content per ASTM D2216
- Combined Sieve and Hydrometer per ASTM D6913

# 3. Exploration and Testing Results

## 3.1 Site and Regional Geology

Multiple advances and retreats of continental glaciers over the State of Michigan during the Pliestocene epoch (beginning approximately 1.8 million years ago) have left a thick sequence of glacial tills, outwash, and lacustrine deposits. The most recent glacial advance was the Wisconsinan glaciation, which ended in lower Michigan approximately 12,000 years ago. These glaciers left behind debris consisting of gravels, sands, silts, and clays. The distribution of various soil types is dependent on the depositional environment related to the proximity of the glacier. In Newaygo County, the thickness of the glacial deposits range from approximately 100 to 600 feet. Glacial deposits in the area consist of lacustrine deposits, glacial till, and outwash sand and gravels. Both sand-and-gravel dominate and clay-and-silt dominate lacustrine deposits are present in the county.

The bedrock beneath the glacial deposits consists of Jurassic “red beds” and the Saginaw Formation (Farrand and Bell, 1982). The Jurassic “red beds” are generally 50 to 150 feet thick and are considered to be a confining unit in Michigan.

## 3.2 Historical Borings

There are no known previous subsurface explorations conducted at the site.

## 3.3 Soil Conditions

The generalized subsurface conditions encountered in the borings are described below. The boring logs contained in **Appendix A.1** should be referenced for detailed descriptions of the subsurface conditions encountered at each boring. Variations in the soil profile should be anticipated throughout the proposed site. A photolog documenting site conditions at the time of drilling are included in **Appendix C**.

**Table 3-1: Soil Borings**

| ID | Type | Location | Approximate Ground Surface Elevation | Proposed Depth |
|---|---|---|---|---|
| GEI-B-01 | SPT Soil Boring | Crest of Left Primary Embankment | 847.2 | 40 |
| GEI-B-02 | SPT Soil Boring | Crest of Right Primary Embankment | 848.9 | 50 |
| GEI-B-03 | SPT Soil Boring | Crest of Right Primary Embankment | 848.9 | 40 |

Embankment Soils – Fill soils were encountered to an average elevation of 18 feet within the embankment borings. Encountered fill soils primarily consisted of fine to coarse sand, with gravel and silt included at varying depths. N values ranged from 2 to 77 in fill soils; however, they averaged to 20.

Foundation Soils – The embankment fill soils were underlain by foundation soils that primarily consisted of fine sands and silty fine to medium sands, with N values ranging from 1 to 31, and averaging around 18.

See **Table 3-2** for summary of ground surface and embankment fill elevations.

**Table 3-2: Generalized Subsurface Profile**

| ID | Ground Surface Elevation (ft) | Bottom of Fill Elevation (ft) |
|---|---|---|
| GEI-B-01 | 847.2 | 830.7 |
| GEI-B-02 | 848.9 | 820.5 |
| GEI-B-03 | 848.9 | 839.9 |

## 3.4 Groundwater Conditions

Groundwater was encountered during sampling within each of the borings. The groundwater elevations indicated on the soil boring logs in **Appendix A.1** represent conditions at the time and location indicated. Fluctuations in groundwater levels should be expected seasonally and annually due to variations in precipitation, evaporation, ground surface runoff, and changes in lake level.

**Table 3-3: Subsurface Water Level Elevations**

| ID | Ground Surface Elevation (ft) | Water Level Elevation (ft) |
|---|---|---|
| GEI-B-01 | 847.2 | ~833 |
| GEI-B-02 | 848.9 | 836.9 |
| GEI-B-03 | 848.9 | 813.9 |

## 3.5 Laboratory Testing Results

Laboratory testing was performed on several disturbed samples collected during the exploration. Testing was completed by GEI's laboratory in Marquette, Michigan. A brief description and summary of the laboratory test data is provided in the following sections. The laboratory test results are included in **Appendix B**. Soils are nonplastic. A full summary of test results is included in **Appendix B.1.** A summary of the index testing is provided in **Table 3-3** below.

**Table 3-4: Summary of Index Properties**

| Boring | Sample | Average Elevation (ft) | USCS | Moisture Content (%) | Grain Size Analysis | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | Gravel (%) | Sand (%) | Fines | | |
| | | | | | | | Total (%) | Silt (%) | Clay (%) |
| B-01 | S-6 | 834.2 | SM | 6.3 | 24.3 | 58.2 | 17.5 | 14.2 | 3.3 |
| B-01 | S-9 | 828.2 | SM | 10.2 | 3.5 | 59.5 | 37.0 | 30.3 | 6.7 |
| B-02 | S-7 | 836 | SM | 16.5 | 30.9 | 56.1 | 13.0 | 8.7 | 4.3 |
| B-02 | S-13 | 815 | SM | 9.8 | 8.5 | 62.5 | 29.0 | 24.1 | 4.9 |
| B-03 | S-11 | 824.9 | SM | 10.1 | 2.8 | 60.3 | 36.9 | 32.7 | 4.2 |

### *3.5.1 Soil Classification (Unified Soil Classification System, USCS)*

The soil samples were visually field classified in general accordance with ASTM D2488. Select samples were visually classified in the lab in general accordance with ASTM D2478. The visual classifications are included on the final boring logs in **Appendix A.1**.

### *3.5.2 Moisture Content*

Moisture contents were performed in general accordance with ASTM D2216. Moisture content results are included in the summary table in **Appendix B.1.** Moisture content is also displayed on the soil boring logs in **Appendix A.1**.

### *3.5.3 Sieve and Hydrometer (Combined) Analysis*

Grain size and hydrometer combined tests were completed in general accordance with ASTM D6913. Sieve analysis results included in **Appendix B.3**.

# 4. Limitations

This report has been prepared in general accordance with our proposal dated May 30, 2023.This report follows generally accepted geotechnical engineering practices to aid in the evaluation of this site and to assist the owner and/or engineer in the design of this project. No other warranty, either expressed or implied, is made. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects relevant to the geotechnical characteristics. This report is intended to satisfy the requirements of the Whitecloud geotechnical investigation and is not intended as a preliminary or final design document. Design would be completed in subsequent phases of this project.

The observations provided in this report are based on data obtained from soil borings performed at locations indicated on the location diagram and from information discussed in this report. This report does not reflect any variations which may occur between borings. In the performance of subsurface explorations, specific information is obtained at specific locations at specific times. However, it is a well-known fact that variations in soil and rock conditions exist on most sites between boring locations, and that seasonal and annual fluctuations in groundwater levels will likely occur. The nature and extent of variations may not become evident until the course of construction. If variations then appear evident, it will be necessary for a re-evaluation of recommendations contained in this report after performing on-site observations during the construction period and noting characteristics of the variations.

# 5. References

(Farrand and Bell, 1982) Farrand W.R. and Bell D.L. 1982. Quaternary geology of northern and southern Michigan: Lansing, Mich., Michigan Geological Survey Division, scale 1:500,000, 1982.

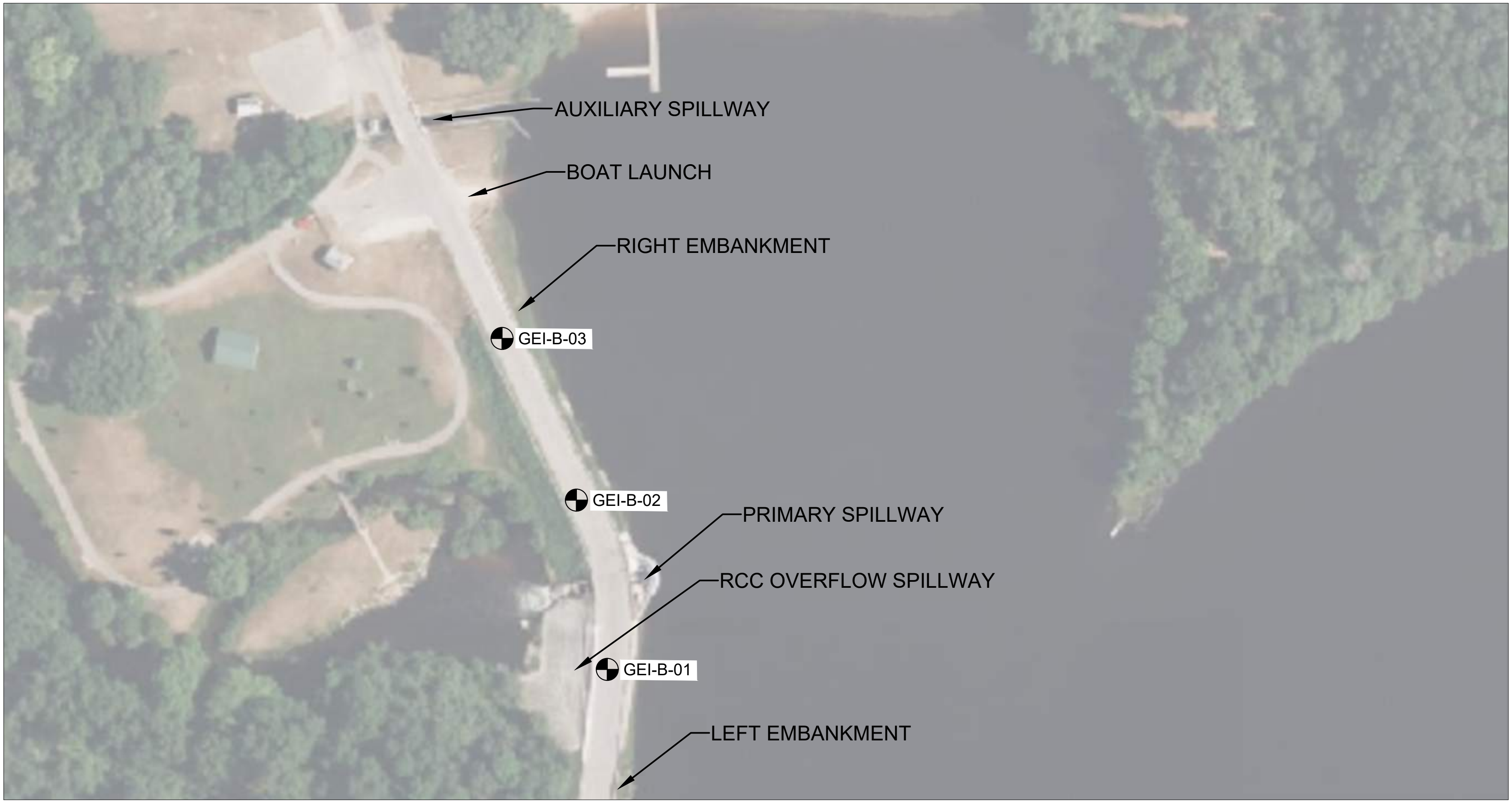
(USACE, 2014) U.S. Army Corps of Engineers (USACE) Engineering Manual ER 1110-1-1807. "Drilling in Earth Embankment Dams and Levees." December 2014.

# Figures

## Figure 1 – Site Location Diagram

## EXPLORATORY BORING SUMMARY TABLE

| ID | TYPE | LOCATION | GROUND SURFACE ELEV | DEPTH |
|---|---|---|---|---|
| GEI-B-01 | SPT SOIL BORING | CREST OF EMBANKMENT | 847.2 | 40 |
| GEI-B-02 | SPT SOIL BORING | CREST OF EMBANKMENT | 848.9 | 50 |
| GEI-B-03 | SPT SOIL BORING | CREST OF EMBANKMENT | 848.9 | 40 |



***LEGEND:***

⊕ PROPOSED SPT SOIL BORINGS

***SOURCE:***

1. PLAN BASED ON MAP PREPARED BY GOOGLE EARTH

FEASIBILITY STUDY
WHITECLOUD DAM
WHITECLOUD, MICHIGAN

CITY OF WHITECLOUD
WHITECLOUD, MICHIGAN

GEI Consultants

BORING LOCATION MAP

Project 2302435

JANUARY 2025

Fig. 1

LEMANSKI, MATT B:\Working\CITY OF WHITE CLOUD\2302435 White Cloud Dam Disposition Feasibility Study\ENG\GDR\FIG 1_gdr.dwg - 1/31/2025

# Appendix A

## Soil Boring Logs

### A.1 – Soil Boring Logs

### A.2 – General Soil Classification Procedures

# A.1 – Soil Boring Logs

CLIENT:
**City of White Cloud**

PROJECT NAME:
**White Cloud Dam**

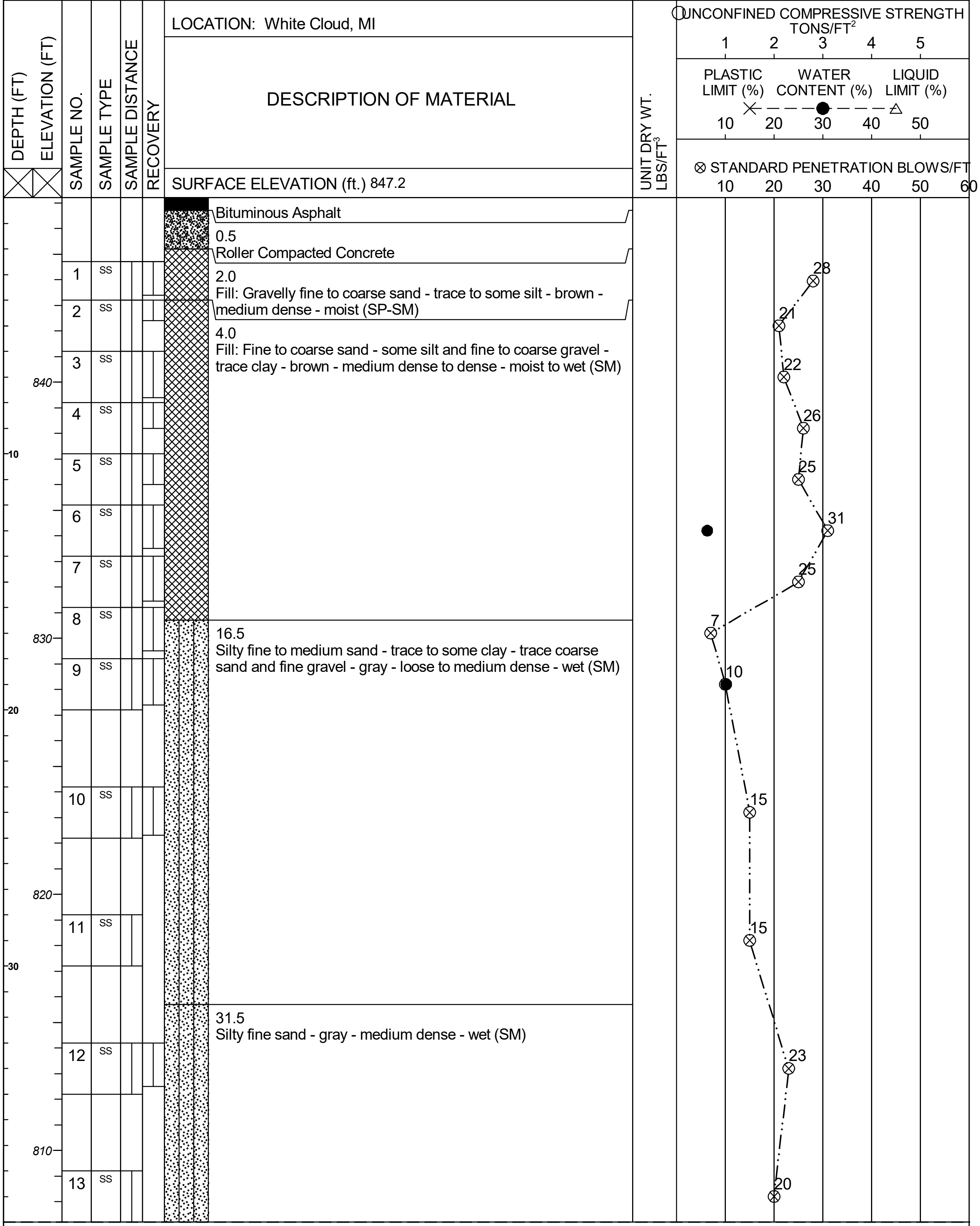
LOG OF BORING NUMBER **B-01**

ARCHITECT-ENGINEER
**GEI Consultants of Michigan, P.C.**

LOCATION: White Cloud, MI

DESCRIPTION OF MATERIAL

SURFACE ELEVATION (ft.) 847.2



Bituminous Asphalt

0.5
Roller Compacted Concrete

2.0
Fill: Gravelly fine to coarse sand - trace to some silt - brown - medium dense - moist (SP-SM)

4.0
Fill: Fine to coarse sand - some silt and fine to coarse gravel - trace clay - brown - medium dense to dense - moist to wet (SM)

16.5
Silty fine to medium sand - trace to some clay - trace coarse sand and fine gravel - gray - loose to medium dense - wet (SM)

31.5
Silty fine sand - gray - medium dense - wet (SM)

The stratification lines represent the approximate boundary lines between soil types: in situ, the transition may be gradual.

| | | | |
|---|---|---|---|
| WATER LEVEL: 5.0' WS | BORING STARTED<br>**10/23/2024** | GEI OFFICE<br>**Marquette, MI** | |
| | BORING COMPLETED<br>**10/23/2024** | ENTERED BY<br>**LJE** | APPROVED BY<br>**CRA** |
| NORTHING EASTING | RIG/FOREMAN<br>**Acker Renegade / WP (Pearson)** | GEI PROJECT NO.<br>**2302435** | PAGE NO. **1** OF **2** |

GEI Consultants

CLIENT:
**City of White Cloud**

PROJECT NAME:
**White Cloud Dam**

LOG OF BORING NUMBER **B-01**

ARCHITECT-ENGINEER
**GEI Consultants of Michigan, P.C.**

LOCATION: White Cloud, MI

| DEPTH (FT) | ELEVATION (FT) | SAMPLE NO. | SAMPLE TYPE | SAMPLE DISTANCE | RECOVERY | DESCRIPTION OF MATERIAL<br>SURFACE ELEVATION (ft.) 847.2 | UNIT DRY WT. LBS/FT³ | ○ UNCONFINED COMPRESSIVE STRENGTH TONS/FT² 1 2 3 4 5<br>PLASTIC LIMIT (%) ✕ WATER CONTENT (%) ● LIQUID LIMIT (%) △ 10 20 30 40 50<br>⊗ STANDARD PENETRATION BLOWS/FT 10 20 30 40 50 60 |
|---|---|---|---|---|---|---|---|---|
|  |  |  |  |  |  | 40.0<br>End of Boring<br><br>Boring advanced to 38.0 feet with hollow-stem auger.<br>Standard Penetration Tests performed with safety hammer.<br>Boring backfilled with cement bentonite grout. |  |  |
|  | 800 |  |  |  |  |  |  |  |
| 50 |  |  |  |  |  |  |  |  |
|  | 790 |  |  |  |  |  |  |  |
| 60 |  |  |  |  |  |  |  |  |
|  | 780 |  |  |  |  |  |  |  |
| 70 |  |  |  |  |  |  |  |  |
|  | 770 |  |  |  |  |  |  |  |

The stratification lines represent the approximate boundary lines between soil types: in situ, the transition may be gradual.

WATER LEVEL: 5.0' WS

BORING STARTED
**10/23/2024**

BORING COMPLETED
**10/23/2024**

GEI OFFICE
**Marquette, MI**

ENTERED BY
**LJE**

APPROVED BY
**CRA**

NORTHING

EASTING

RIG/FOREMAN
**Acker Renegade / WP (Pearson)**

GEI PROJECT NO.
**2302435**

PAGE NO. **2** OF **2**

GEI Consultants

CLIENT: **City of White Cloud**

PROJECT NAME: **White Cloud Dam**

LOG OF BORING NUMBER **B-02**

ARCHITECT-ENGINEER **GEI Consultants of Michigan, P.C.**

LOCATION: White Cloud, MI

DESCRIPTION OF MATERIAL

SURFACE ELEVATION (ft.) 849.0

DEPTH (FT) | ELEVATION (FT) | SAMPLE NO. | SAMPLE TYPE | SAMPLE DISTANCE | RECOVERY | UNIT DRY WT. LBS/FT³

| SAMPLE NO. | SAMPLE TYPE |
|---|---|
| 1 | SS |
| 2 | SS |
| 3 | SS |
| 4 | SS |
| 5 | SS |
| 6 | SS |
| 7 | SS |
| 8 | SS |
| 9 | SS |
| 10 | SS |
| 11 | SS |
| 12 | SS |
| 13 | SS |
| 14 | SS |

3 Inches of Bituminous Asphalt over Gravel Base

0.5
Fill: Fine to coarse sand - trace to some silt and fine to coarse gravel - brown - dense to medium dense - moist (SP-SM)

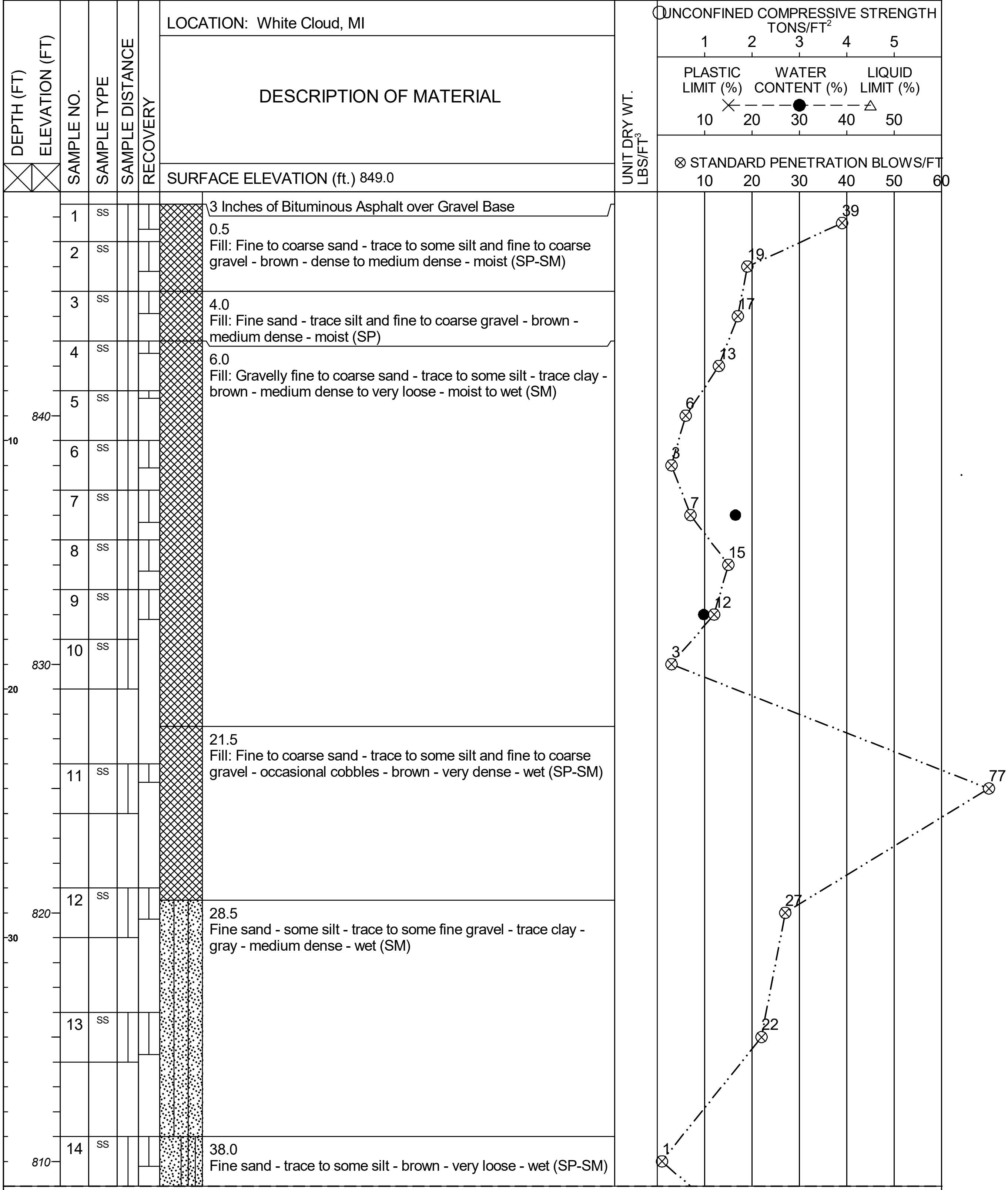
4.0
Fill: Fine sand - trace silt and fine to coarse gravel - brown - medium dense - moist (SP)

6.0
Fill: Gravelly fine to coarse sand - trace to some silt - trace clay - brown - medium dense to very loose - moist to wet (SM)

21.5
Fill: Fine to coarse sand - trace to some silt and fine to coarse gravel - occasional cobbles - brown - very dense - wet (SP-SM)

28.5
Fine sand - some silt - trace to some fine gravel - trace clay - gray - medium dense - wet (SM)

38.0
Fine sand - trace to some silt - brown - very loose - wet (SP-SM)



The stratification lines represent the approximate boundary lines between soil types: in situ, the transition may be gradual.

WATER LEVEL: 11.0' WS

BORING STARTED **10/23/2024**

BORING COMPLETED **10/23/2024**

GEI OFFICE **Marquette, MI**

ENTERED BY **LJE**

APPROVED BY **CRA**

NORTHING

EASTING

RIG/FOREMAN **Acker Renegade / WP (Pearson)**

GEI PROJECT NO. **2302435**

GEI Consultants

CLIENT: **City of White Cloud**

PROJECT NAME: **White Cloud Dam**

LOG OF BORING NUMBER **B-02**

ARCHITECT-ENGINEER **GEI Consultants of Michigan, P.C.**

LOCATION: White Cloud, MI

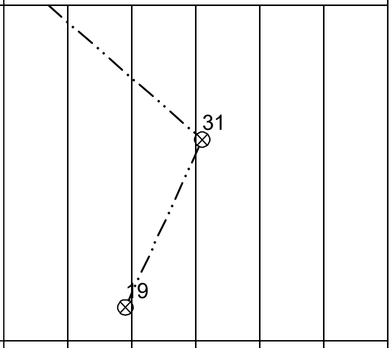
SURFACE ELEVATION (ft.) 849.0

UNCONFINED COMPRESSIVE STRENGTH TONS/FT$^2$: 1 2 3 4 5

PLASTIC LIMIT (%) — WATER CONTENT (%) — LIQUID LIMIT (%): 10 20 30 40 50

⊗ STANDARD PENETRATION BLOWS/FT: 10 20 30 40 50 60

| DEPTH (FT) | ELEVATION (FT) | SAMPLE NO. | SAMPLE TYPE | DESCRIPTION OF MATERIAL | UNIT DRY WT. LBS/FT$^3$ | STANDARD PENETRATION BLOWS/FT |
|---|---|---|---|---|---|---|
| 41.5 | | | | Fine sand - trace silt - gray - dense to medium dense - wet (SP) | | |
| | | 15 | SS | | | 31 |
| | 800 | 16 | SS | | | 19 |
| 50 | | | | 50.0<br>End of Boring<br>Boring advanced to 48.0 feet with hollow-stem auger.<br>Standard Penetration Tests performed with safety hammer.<br>Boring backfilled cement bentonite grout. | | |
| | 790 | | | | | |
| 60 | | | | | | |
| | 780 | | | | | |
| 70 | | | | | | |
| | 770 | | | | | |



The stratification lines represent the approximate boundary lines between soil types: in situ, the transition may be gradual.

WATER LEVEL: 11.0' WS

BORING STARTED **10/23/2024**

BORING COMPLETED **10/23/2024**

GEI OFFICE **Marquette, MI**

ENTERED BY **LJE**

APPROVED BY **CRA**

NORTHING

EASTING

RIG/FOREMAN **Acker Renegade / WP (Pearson)**

GEI PROJECT NO. **2302435**

GEI Consultants

CLIENT: **City of White Cloud**

PROJECT NAME: **White Cloud Dam**

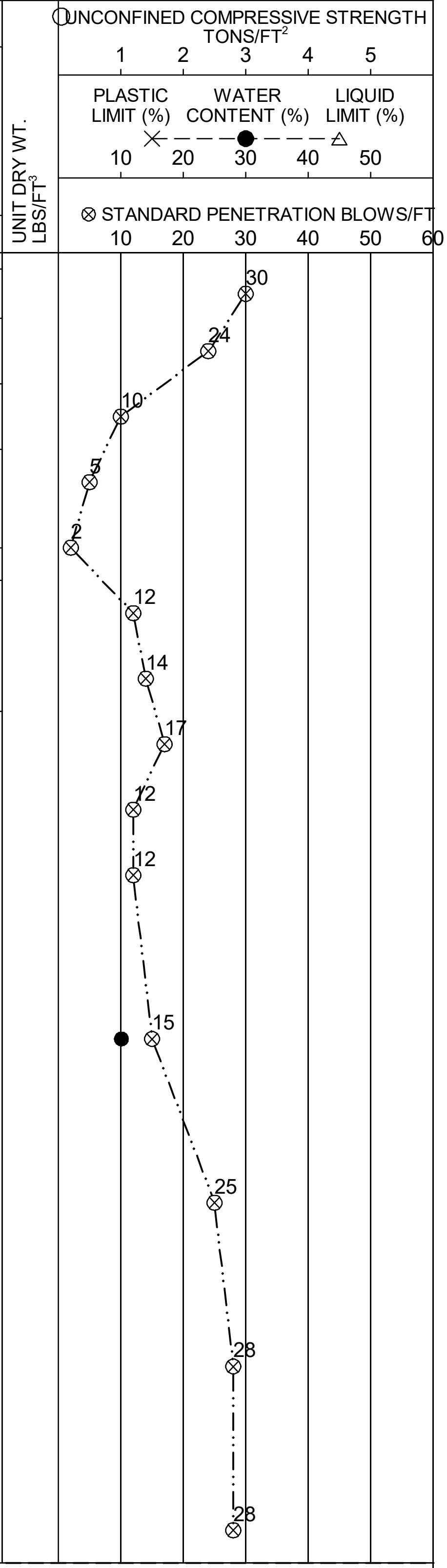
LOG OF BORING NUMBER **B-03**

ARCHITECT-ENGINEER **GEI Consultants of Michigan, P.C.**

LOCATION: White Cloud, MI

SURFACE ELEVATION (ft.) 848.9

| DEPTH (FT) | ELEVATION (FT) | SAMPLE NO. | SAMPLE TYPE | DESCRIPTION OF MATERIAL |
|---|---|---|---|---|
| | | 1 | SS | 3 Inches of Bituminous Asphalt over Gravel Base |
| 0.5 | | 2 | SS | Fill: Gravelly fine to coarse sand - trace to some silt - brown - medium dense - moist (SP-SM) |
| 2.0 | | 3 | SS | Fill: Fine sand - trace to some fine to coarse gravel - trace silt - brown - medium dense - moist (SP) |
| 4.0 | | 4 | SS | Fill: Fine to medium sand - trace to some silt and fine to coarse gravel - brown and dark brown - medium dense - moist (SP-SM) |
| 6.0 | 840 | 5 | SS | Fill: Fine sand - trace silt - brown - loose to very loose - moist to wet (SP) |
| 9.0 | | 6 | SS | Silt - some organic silt - brown - very loose - wet (ML) |
| 10.0 | | 7 | SS | Fine to medium sand - some silt - light brown - medium dense - wet (SM) |
| 14.0 | | 8 | SS | Silty fine to medium sand - trace clay, coarse sand, and fine gravel - gray - medium dense to dense - wet (SM) |
| | | 9 | SS | |
| 20 | 830 | 10 | SS | |
| | | 11 | SS | |
| 30 | 820 | 12 | SS | |
| | | 13 | SS | |
| | 810 | 14 | SS | |



The stratification lines represent the approximate boundary lines between soil types: in situ, the transition may be gradual.

| | | | |
|---|---|---|---|
| WATER LEVEL: 9.0' WS | BORING STARTED **10/23/2024** | GEI OFFICE **Marquette, MI** | |
| | BORING COMPLETED **10/23/2024** | ENTERED BY **LJE** | APPROVED BY **CRA** |
| NORTHING EASTING | RIG/FOREMAN **Acker Renegade / WP (Pearson)** | GEI PROJECT NO. **2302435** | PAGE NO. **1** OF **2** |

GEI Consultants

CLIENT:
**City of White Cloud**

PROJECT NAME:
**White Cloud Dam**

LOG OF BORING NUMBER **B-03**

ARCHITECT-ENGINEER
**GEI Consultants of Michigan, P.C.**

LOCATION: White Cloud, MI

DESCRIPTION OF MATERIAL

SURFACE ELEVATION (ft.) 848.9

DEPTH (FT) | ELEVATION (FT) | SAMPLE NO. | SAMPLE TYPE | SAMPLE DISTANCE | RECOVERY | UNIT DRY WT. LBS/FT³

○ UNCONFINED COMPRESSIVE STRENGTH TONS/FT²
1 2 3 4 5

PLASTIC LIMIT (%) ✕ — — ● — — △ WATER CONTENT (%) LIQUID LIMIT (%)
10 20 30 40 50

⊗ STANDARD PENETRATION BLOWS/FT
10 20 30 40 50 60

40.0
End of Boring

Boring advanced to 38.0 feet with hollow-stem auger.
Standard Penetration Tests performed with safety hammer.
Boring backfilled cement bentonite grout.

| DEPTH (FT) | ELEVATION (FT) |
|---|---|
| | 800 |
| 50 | |
| | 790 |
| 60 | |
| | 780 |
| 70 | |
| | 770 |

The stratification lines represent the approximate boundary lines between soil types: in situ, the transition may be gradual.

| | | | |
|---|---|---|---|
| WATER LEVEL: 9.0' WS | BORING STARTED<br>10/23/2024 | GEI OFFICE<br>Marquette, MI | |
| | BORING COMPLETED<br>10/23/2024 | ENTERED BY<br>LJE | APPROVED BY<br>CRA |
| NORTHING EASTING | RIG/FOREMAN<br>Acker Renegade / WP (Pearson) | GEI PROJECT NO.<br>2302435 | PAGE NO. 2 OF 2 |

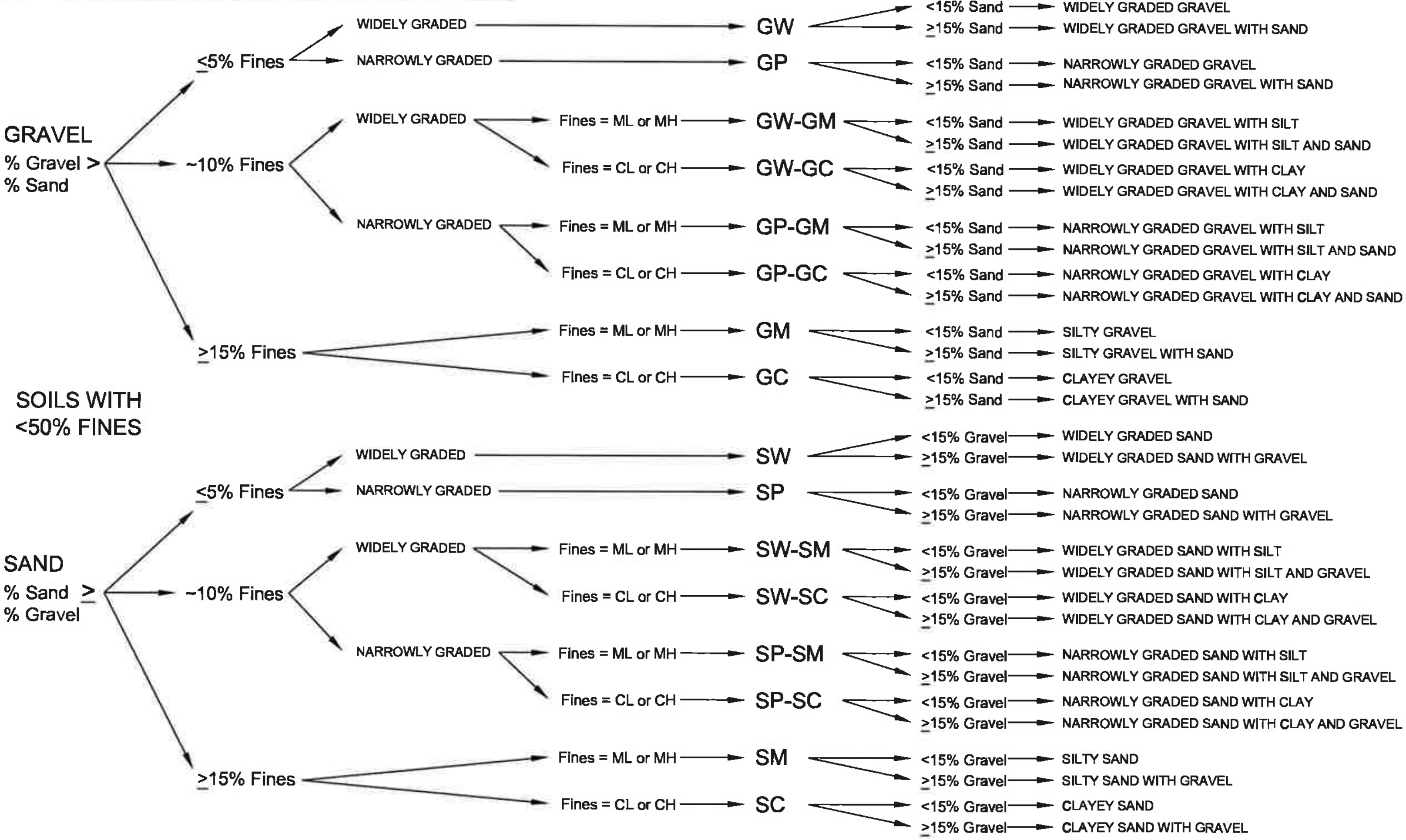
# A.2 – General Soil Classification Procedures

GEI Consultants

# COARSE-GRAINED SOILS
## VISUAL-MANUAL DESCRIPTIONS

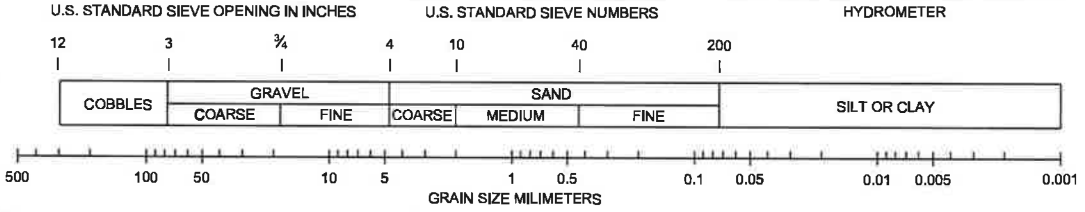
**TYPICAL SOIL COLORS**

- 🟨 YELLOW
- 🟩 OLIVE
- 🟫 LIGHT BROWN
- 🟫 BROWN
- 🟥 REDDISH BROWN
- 🟥 RED
- ⬜ GRAY
- ⬛ BLACK



SOILS WITH <50% FINES

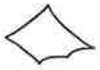
| | | | | GROUP SYMBOL | | GROUP NAME |
|---|---|---|---|---|---|---|
| GRAVEL<br>% Gravel > % Sand | ≤5% Fines | WIDELY GRADED | | GW | <15% Sand | WIDELY GRADED GRAVEL |
| | | | | | ≥15% Sand | WIDELY GRADED GRAVEL WITH SAND |
| | | NARROWLY GRADED | | GP | <15% Sand | NARROWLY GRADED GRAVEL |
| | | | | | ≥15% Sand | NARROWLY GRADED GRAVEL WITH SAND |
| | ~10% Fines | WIDELY GRADED | Fines = ML or MH | GW-GM | <15% Sand | WIDELY GRADED GRAVEL WITH SILT |
| | | | | | ≥15% Sand | WIDELY GRADED GRAVEL WITH SILT AND SAND |
| | | | Fines = CL or CH | GW-GC | <15% Sand | WIDELY GRADED GRAVEL WITH CLAY |
| | | | | | ≥15% Sand | WIDELY GRADED GRAVEL WITH CLAY AND SAND |
| | | NARROWLY GRADED | Fines = ML or MH | GP-GM | <15% Sand | NARROWLY GRADED GRAVEL WITH SILT |
| | | | | | ≥15% Sand | NARROWLY GRADED GRAVEL WITH SILT AND SAND |
| | | | Fines = CL or CH | GP-GC | <15% Sand | NARROWLY GRADED GRAVEL WITH CLAY |
| | | | | | ≥15% Sand | NARROWLY GRADED GRAVEL WITH CLAY AND SAND |
| | ≥15% Fines | | Fines = ML or MH | GM | <15% Sand | SILTY GRAVEL |
| | | | | | ≥15% Sand | SILTY GRAVEL WITH SAND |
| | | | Fines = CL or CH | GC | <15% Sand | CLAYEY GRAVEL |
| | | | | | ≥15% Sand | CLAYEY GRAVEL WITH SAND |
| SAND<br>% Sand ≥ % Gravel | ≤5% Fines | WIDELY GRADED | | SW | <15% Gravel | WIDELY GRADED SAND |
| | | | | | ≥15% Gravel | WIDELY GRADED SAND WITH GRAVEL |
| | | NARROWLY GRADED | | SP | <15% Gravel | NARROWLY GRADED SAND |
| | | | | | ≥15% Gravel | NARROWLY GRADED SAND WITH GRAVEL |
| | ~10% Fines | WIDELY GRADED | Fines = ML or MH | SW-SM | <15% Gravel | WIDELY GRADED SAND WITH SILT |
| | | | | | ≥15% Gravel | WIDELY GRADED SAND WITH SILT AND GRAVEL |
| | | | Fines = CL or CH | SW-SC | <15% Gravel | WIDELY GRADED SAND WITH CLAY |
| | | | | | ≥15% Gravel | WIDELY GRADED SAND WITH CLAY AND GRAVEL |
| | | NARROWLY GRADED | Fines = ML or MH | SP-SM | <15% Gravel | NARROWLY GRADED SAND WITH SILT |
| | | | | | ≥15% Gravel | NARROWLY GRADED SAND WITH SILT AND GRAVEL |
| | | | Fines = CL or CH | SP-SC | <15% Gravel | NARROWLY GRADED SAND WITH CLAY |
| | | | | | ≥15% Gravel | NARROWLY GRADED SAND WITH CLAY AND GRAVEL |
| | ≥15% Fines | | Fines = ML or MH | SM | <15% Gravel | SILTY SAND |
| | | | | | ≥15% Gravel | SILTY SAND WITH GRAVEL |
| | | | Fines = CL or CH | SC | <15% Gravel | CLAYEY SAND |
| | | | | | ≥15% Gravel | CLAYEY SAND WITH GRAVEL |




ROUNDED


SUBROUNDED


SUBANGULAR

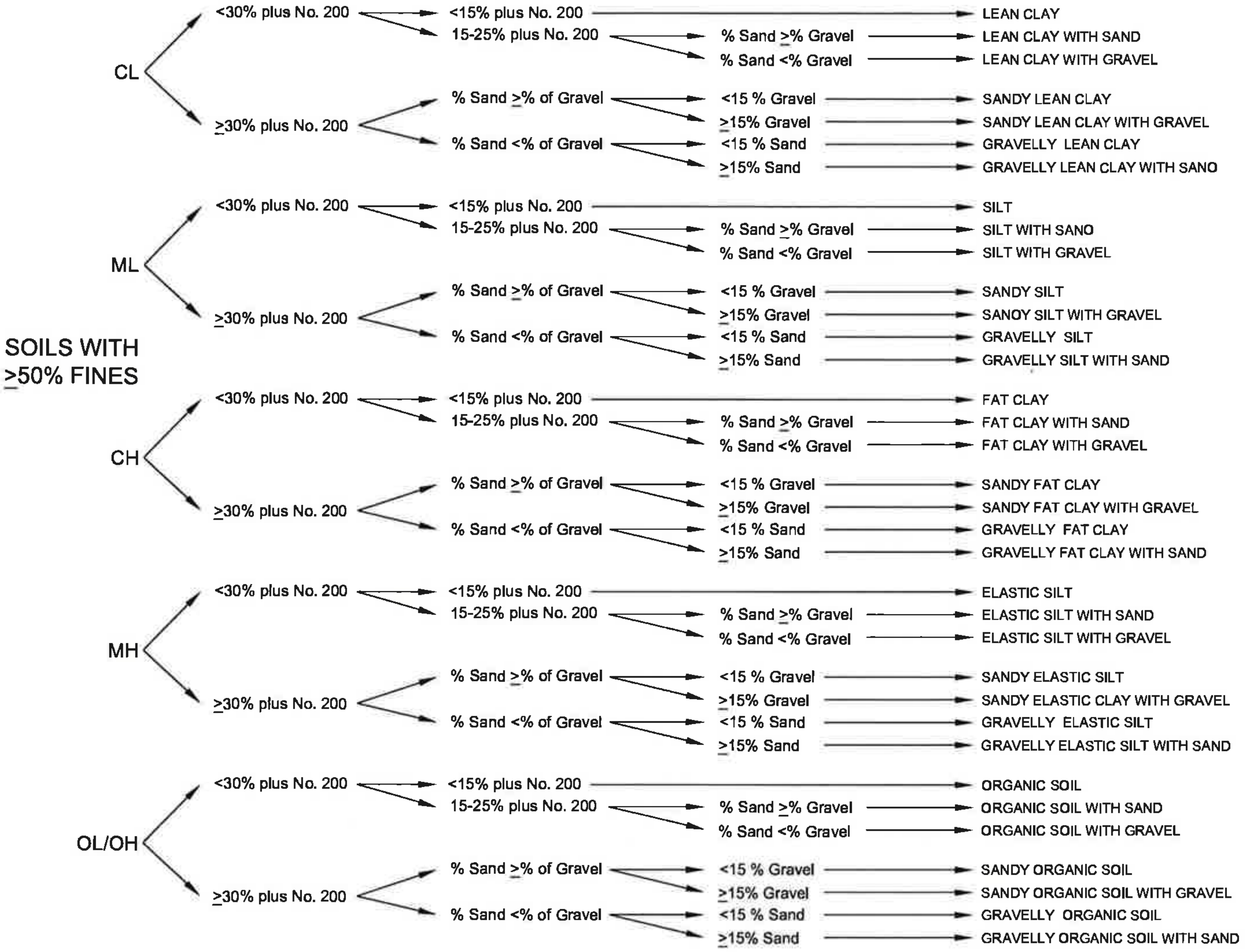

ANGULAR

1. GROUP NAME and (SYMBOL)
2. Structure , if any. (stratified layer thicknesses, lenses, varves, gradational changes)
3. Describe sand, gravel and fines components, with percentages, in order of predominance. Include max gravel size. For test pits give percent cobbles and boulders, by volume, and include max size.
4. Color
5. Sheen, odor, roots, ash, brick, cementation, reaction with HCL, etc.
6. "Fill," local name or geologic name, if known

GEI Consultants

# FINE-GRAINED SOILS

## VISUAL-MANUAL DESCRIPTIONS



## ID OF INORGANIC FINE SOILS FROM MANUAL TESTS

| Symbol | Name | Dry Strength | Dilatancy | Toughness* |
|---|---|---|---|---|
| ML | Silt | None to low | Slow to rapid | Low or thread cannot be formed |
| CL | Lean Clay | Medium to high | None to slow | Medium |
| MH | Elastic Silt | Low to medium | None to slow | Low to medium |
| CH | Fat Clay | High to very high | None | High |

## CRITERIA FOR DESCRIBING PLASTICITY

| Description | Criteria |
|---|---|
| Nonplastic ML | A 1/8-in. (3 -mm) thread cannot be rolled at any water content |
| Low Plasticity ML, MH | The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit * |
| Medium Plasticity MH, CL | The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit |
| High Plasticity CH | It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit |

1. GROUP NAME and (SYMBOL)
2. Describe fines, sand, and gravel components, in order of predominance. Include plasticity of fines. Include percentages of sand and gravel.
3. Color
4. Sheen, odor, roots, ash, brick, cementation, torvane and penetrometer results, etc.
5. "Fill," local name or geologic name, if known

### PEAT

Peat refers to a sample composed primarily of vegetable matter in varying stages of decomposition. The description should begin: PEAT (PT) and need not include percentages of sand, gravel or fines.

* Toughness refers to the strength of the thread near plastic limit. The lump refers to a lump of soil drier than the plastic, similar to dry strength.

# GENERAL NOTES

## Drilling and Sampling Symbols:

SS: Split-Spoon, 1 3/8-inch ID, 2-inch OD
Unless otherwise noted
ST: Shelby Tube
PA: Power Auger
DB: Diamond Bit
AS: Auger Sample
JS: Jar Sample
VS: Vane Shear
WOH: Weight of Hammer

OS: Osterburg Sampler
HSA: Hollow Stem Auger
WS: Wash Sample
FT: Fish Tail
RB: Rock Bit
BS: Bulk Sample
PMT: Pressuremeter Test
GS: Giddings Sampler

Standard Penetration Test (STP) Value: Blows per foot of a 140-pound hammer falling 30 inches on a 2-inch OD split-spoon sampler, except where otherwise noted.

## Water Level Measurement Symbols:

WL: Water Level
WS: While Sampling
WD: While Drilling
AB: After Boring

WCI: Wet Cave-in
DCI: Dry Cave-in
BCI: Before Casing Installation
BCR: Before Casing Removal
ACR: After Casing Removal

Water levels indicated on the boring logs are the levels measured in the boring at the time indicated. In permeable soils, the indicated elevations can be considered a reliable groundwater level. In impervious soils, the accurate determination of groundwater elevations may not be possible, even after several days of observations. In these cases, groundwater monitoring wells may need to be constructed and monitored for an extended period of time to determine the actual groundwater level.

## Gradation Description and Terminology:

Coarse-grained or granular soils are defined as having more than 50% of their dry weight retained on the No. 200 sieve. Coarse grained soils include boulders, cobbles, gravel, and/or sand. Fine-grained soils are defined as having less than 50% of their dry weight retained on the No. 200 sieve. Fine grained soils include clay or clayey silt (cohesive), and silt (non-cohesive). In addition to gradation, granular soils are further defined based on their relative in-place density. Fine-grained soils are further defined based of their strength or consistency and plasticity. Additional information is provided below.

| Major Component of Sample | Size Range | Other Components Present in Sample | Dry Weight, % |
|---|---|---|---|
| Boulders | Over 8 inches (200 mm) | Trace | 1 to 5 |
| Cobbles | 8 inches to 3 inches<br>(200 mm to 75 mm) | Trace to Some | 5 to 12 |
| Gravel | 3 inches to No. 4 sieve | Some | 12 to 34 |
| Sand | Nos. 4 to 200 sieves<br>(4.76 mm to 0.074 mm) | And | 34 to 50 |
| Silt | Passing No. 200 sieve<br>(0.074 mm to 0.005 mm) | | |
| Clay | Smaller than 0.005 mm | | |

| Consistency of Cohesive Soils | | Relative Density of Granular Soils | |
|---|---|---|---|
| Unconfined Compressive Strength, Qu, tsf | Consistency | N, blows per foot | Relative Density |
| <0.25 | Very Soft | 0 to 3 | Very Loose |
| 0.25 to 0.49 | Soft | 4 to 9 | Loose |
| 0.50 to 0.99 | Medium (firm) | 10 to 29 | Medium Dense |
| 1.0 to 1.99 | Stiff | 30 to 49 | Dense |
| 2.00 to 3.99 | Very Stiff | 50 – 80 | Very Dense |
| 4.00 to 8.00 | Hard | >80 | Extremely Dense |
| >8.00 | Very Hard | | |

# FIELD AND LABORATORY PROCEDURES

## Field Sampling Procedures

### Auger Sampling (AS)

In this procedure, soil samples are collected from cuttings off the auger flights as they are removed from the ground. Such samples provide a general indication of subsurface conditions; however, they do not provide undisturbed samples, nor do they provide samples from discrete depths.

### Split-Barrel Sampling (SS) – (ASTM Standard D-1586-99)

In the split-barrel sampling procedures, a 2-inch O.D. split-barrel sampler is driven into the soil a distance of 18 inches by means of a 140-pound hammer falling 30 inches. The value of the Standard Penetration Resistance is obtained by counting the number of blows of the hammer over the final 12 inches of driving. The value provides a qualitative indication of the in-place relative density of cohesionless soils. The indication is only qualitative, however, since many factors can significantly affect the Standard Penetration Resistance Value, and direct correlation of results obtained by drill crews using different rigs, frilling procedures, and hammer-rod-spoon assemblies should not be made. A portion of the recovered sample is place in a sample jar and returned to the laboratory for further analysis and testing.

### Shelby Tube Sampling Procedure (ST) - (ASTM D-1587-94)

In the Shelby tube sampling procedure, a thin-walled steel seamless tube with a sharp cutting edge is pushed hydraulically into the soil and a relatively undisturbed sample is obtained. This procedure is generally employed in cohesive soils. The tubes are identified, sealed, and carefully handled in the field to avoid excessive disturbance and are returned to the laboratory for extrusion and further analysis and testing.

### Giddings Sampler (GS)

This type of sampling device consists of 5-foot sections of thin-wall tubing, which are capable of retrieving continuous columns of soil in 5-foot maximum increments. Because of a continuous slot in the sampling tubes, the sampler allows field determination of stratification boundaries and containerization of soil samples from any sampling depth within the 5-foot interval.

# FIELD AND LABORATORY PROCEDURES

## Subsurface Exploration Field Procedures

### Hand-Auger Drilling (HA)

In this procedure, a sampling device is driven into the soil by repeated blows of a sledge hammer or a drop hammer. When the sampler is driven to the desired depth, the soil sample is retrieved. The hole is then advanced by manually turning the hand auger until the next sampling depth increment is reached. The hand auger drilling between sampling intervals also helps to clean and enlarge the borehole in preparation for obtaining the next sample.

### Power Auger Drilling (PA)

In this type of drilling procedures, continuous flight augers are used to advance the boreholes. They are turned and hydraulically advanced by a truck, trainer, or track-mounted unit as site accessibility dictates. In auger drilling, casing and drilling mud are not required to maintain open boreholes.

### Hollow-Stem Auger Drilling (HS)

In this drilling procedure, continuous flight augers (with open stems) are used to advance the boreholes. The open stem allows the sampling tool to be used without removing the augers from the borehole. Hollow-stem augers thus provide support to the sides of the borehole during the sampling operations.

### Rotary Drilling (RD)

In employing rotary drilling methods, various cutting bits are used to advance the boreholes. In this process, surface casing and/or drilling fluids are used to maintain open boreholes.

### Diamond Core Drilling (DB)

Diamond core drilling is used to sample cemented formations. In this procedure, a double tube (or triple tube) core barrel with a diamond bit cuts an annular space around a cylindrical prism of the material sampled. The sample is retrieved by a catcher just above the bit. Samples recovered by this procedure are placed in study containers in sequential order.

# FIELD AND LABORATORY PROCEDURES

## Laboratory Procedures

### Water Content (Wc)

The water content of a soil is the ratio of the weight of water in a given soil mass to the weight of the dry soil. Water content is generally expressed as a percentage.

### Hand Penetrometer (Qp)

In the hand penetrometer gtest, the unconfined compressive strength of a soil is determined to a maximum value of 4.5 tons per square foot (tsf) or 7.0 tsf, depending on the testing device utilized, by measuring the resistance of the soil sample to penetration by a small spring-calibrated cylinder. The hand penetrometer test has been carefully correlated with unconfined compressive strength tests and thereby provides a useful and a relative simple testing procedure in which soil strength can be quickly and easily estimated.

### Unconfined Compression Tests (Qu)

In the unconfined compression strength test, an undisturbed prism of soil is loaded axially until failure or until 20% strain has been reached, whichever comes first.

### Dry Density (γd)

The dry density is a measure of the amount of solids in a unit volume of soil. Use of this value is often made when measuring the degree of compaction of a soil.

### Classification of Samples

In conjunction with the sample testing program, all soil samples are examined in our laboratory and visually classified on the basis of their texture and plasticity in general accordance with the Unified Soil Classification System. The soil descriptions on the boring logs are derived from this system, as well as the component gradation terminology, consistency of cohesive soils, and relative density of granular soils, as described on a separate sheet entitled General Notes. The estimated groups symbols, included in parentheses following the soil descriptions on the boring logs, are in general conformance with the Unified Soil Classification System (USCS).

# FIELD AND LABORATORY PROCEDURES

## Standard Boring Log Procedures

In the process of obtaining and testing samples and preparing this report, standard procedures are followed regarding field logs, laboratory data sheets, and samples.

Field logs are prepared during performance of the drilling and sampling operations and are intended to essentially portray field occurrences, sampling locations, and procedures.

Samples obtained in the field are frequently subjected to additional testing an re-classification in the laboratory by experienced Geotechnical Engineers; and therefore, differences between the field logs and the final logs may exist. The engineer preparing the report reviews the field logs, laboratory test data, and classifications and then, using judgement and experience in interpreting this data, may make further changes. It is common practice in the geotechnical engineering profession not to include field logs and laboratory data sheets in engineering reports, because they do not represent the engineer's final opinions as to appropriate descriptions for conditions encountered in the exploration and testing work. Results of laboratory tests are generally shown on the boring logs or are described in the text of the report, as appropriate.

Samples taken in the field, some of which are later subjected to laboratory tests, are retained in our laboratory for 60 days and then discarded, unless special disposition is requested by our client. Samples retained over a long period of time, even though in sealed jars, are subject to moisture loss, which changes the apparent strength of cohesive soil, generally increasing the strength from what was originally encountered in the field. Since they are then no longer representative of the moisture conditions initially encountered, observers of these samples need to recognize this factor.

# Appendix B

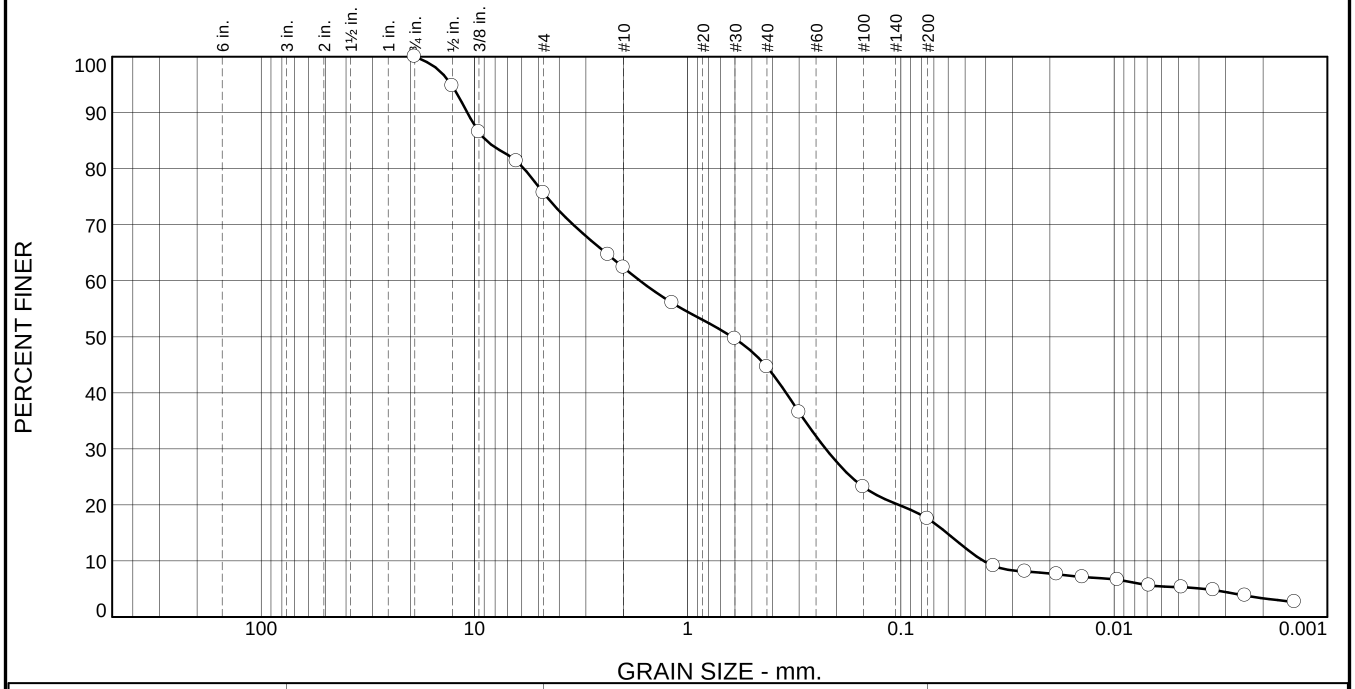
## Laboratory Test Results

# B.1 – Summary of Laboratory Testing

| Boring | Sample | Average Elevation (ft) | USCS | Moisture Content (%) | Grain Size Analysis | | | | | Atterberg Limits | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Gravel (%) | Sand (%) | Fines | | | | | |
| | | | | | | | Total (%) | Silt (%) | Clay (%) | Liquid Limit | Plastic Limit | Plasticity Index |
| B-01 | S-6 | 834.2 | SM | 6.3 | 24.3 | 58.2 | 17.5 | 14.2 | 3.3 | - | - | - |
| B-01 | S-9 | 828.2 | SM | 10.2 | 3.5 | 59.5 | 37.0 | 30.3 | 6.7 | - | - | - |
| B-02 | S-7 | 836 | SM | 16.5 | 30.9 | 56.1 | 13.0 | 8.7 | 4.3 | - | - | - |
| B-02 | S-13 | 815 | SM | 9.8 | 8.5 | 62.5 | 29.0 | 24.1 | 4.9 | - | - | - |
| B-03 | S-11 | 824.9 | SM | 10.1 | 2.8 | 60.3 | 36.9 | 32.7 | 4.2 | - | - | - |

# B.2 – Combined Gradation

# Particle Size Distribution Report



| % +3" | % Gravel |  | % Sand |  |  | % Fines |  |
|---|---|---|---|---|---|---|---|
|  | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 24.3 | 13.3 | 17.8 | 27.1 | 14.2 | 3.3 |

**Test Results (ASTM D422 & ASTM D1140)**

| Opening Size | Percent Finer | Spec.* (Percent) | Pass? (X=Fail) |
|---|---|---|---|
| 3/4" | 100.0 |  |  |
| 1/2" | 94.8 |  |  |
| 3/8" | 86.5 |  |  |
| 1/4" | 81.4 |  |  |
| #4 | 75.7 |  |  |
| #8 | 64.6 |  |  |
| #10 | 62.4 |  |  |
| #16 | 56.0 |  |  |
| #30 | 49.6 |  |  |
| #40 | 44.6 |  |  |
| #50 | 36.5 |  |  |
| #100 | 23.2 |  |  |
| #200 | 17.5 |  |  |
| 0.0368 mm. | 9.1 |  |  |
| 0.0261 mm. | 8.1 |  |  |
| 0.0185 mm. | 7.6 |  |  |
| 0.0141 mm. | 7.1 |  |  |
| 0.0096 mm. | 6.6 |  |  |
| 0.0068 mm. | 5.6 |  |  |
| 0.0048 mm. | 5.3 |  |  |
| 0.0034 mm. | 4.8 |  |  |
| 0.0024 mm. | 3.8 |  |  |
| 0.0014 mm. | 2.7 |  |  |

* (no specification provided)

## Material Description

Fine to coarse sand- some silt and fine gravel- trace clay- brown

### Atterberg Limits (ASTM D 4318)

**PL=** NP **LL=** NV **PI=** NP

### Classification

**USCS (D 2487)=** SM **AASHTO (M 145)=** A-1-b

### Coefficients

|  |  |  |
|---|---|---|
| $D_{90}$= 10.7938 | $D_{85}$= 8.7813 | $D_{60}$= 1.6721 |
| $D_{50}$= 0.6199 | $D_{30}$= 0.2253 | $D_{15}$= 0.0608 |
| $D_{10}$= 0.0410 | $C_u$= 40.82 | $C_c$= 0.74 |

### Remarks

Moisture Content: 6.3%

**Date Received:** 10/23/2024 **Date Tested:** 12/8/2024

**Tested By:** Kevin Rautiola

**Checked By:** Chris Abraham, PE

**Title:** QA Manager

**Location:** B-01

**Sample Number:** S-6 **Depth:** 12 to 14 Feet

**Date Sampled:** 10/23/2024

**GEI Consultants of Michigan, P.C.**

**Marquette, Michigan**

**Client:** City of White Cloud

**Project:** White Cloud Dam Disposition Feasibility Study

**Project No:** 2302435

## GRAIN SIZE DISTRIBUTION TEST DATA

**12/9/2024**

**Client:** City of White Cloud
**Project:** White Cloud Dam Disposition Feasibility Study
**Project Number:** 2302435
**Location:** B-01
**Depth:** 12 to 14 Feet **Sample Number:** S-6
**Material Description:** Fine to coarse sand- some silt and fine gravel- trace clay- brown
**Sample Date:** 10/23/2024
**Date Received:** 10/23/2024 **PL:** NP **LL:** NV **PI:** NP
**USCS Classification:** SM **AASHTO Classification:** A-1-b
**Grain Size Test Method:** ASTM D422
**#200 Wash Method:** ASTM D1140
**Testing Remarks:** Moisture Content: 6.3%
**Tested By:** Kevin Rautiola **Test Date:** 12/8/2024
**Checked By:** Chris Abraham, PE **Title:** QA Manager

### Sieve Test Data

**Post #200 Wash Test Weights (grams):** **Dry Sample and Tare =** 570.40
**Tare Wt. =** 346.50
**Minus #200 from wash =** 16.0%

| Dry Sample and Tare (grams) | Tare (grams) | Sieve Opening Size | Weight Retained (grams) | Sieve Weight (grams) | Percent Finer |
|---|---|---|---|---|---|
| 613.20 | 346.50 | 1-1/2" | | | |
| | | 1" | | | |
| | | 3/4" | 0.00 | 0.00 | 100.0 |
| | | 1/2" | 13.90 | 0.00 | 94.8 |
| | | 3/8" | 22.00 | 0.00 | 86.5 |
| | | 1/4" | 13.80 | 0.00 | 81.4 |
| | | #4 | 15.10 | 0.00 | 75.7 |
| | | #8 | 29.50 | 0.00 | 64.6 |
| | | #10 | 6.10 | 0.00 | 62.4 |
| | | #16 | 16.90 | 0.00 | 56.0 |
| | | #30 | 17.00 | 0.00 | 49.6 |
| | | #40 | 13.40 | 0.00 | 44.6 |
| | | #50 | 21.60 | 0.00 | 36.5 |
| | | #100 | 35.50 | 0.00 | 23.2 |
| | | #200 | 15.10 | 0.00 | 17.5 |

## Hydrometer Test Data

**Hydrometer test uses material passing** #10

**Percent passing #10 based upon complete sample =** 62.4

**Weight of hydrometer sample =**67.0

**Hygroscopic moisture correction:**

**Moist weight and tare =** 630.10

**Dry weight and tare =** 613.20

**Tare weight =** 346.50

**Hygroscopic moisture =** 6.3%

**Table of composite correction values:**

| | | | | | | |
|---|---|---|---|---|---|---|
| **Temp., deg. C:** | 23.1 | 22.0 | 21.0 | 20.0 | 19.0 | 18.2 |
| **Comp. corr.:** | -2.1 | -2.4 | -2.7 | -3.0 | -3.3 | -3.8 |

**Meniscus correction only =** 0.5

**Specific gravity of solids =** 2.65

**Hydrometer type =** 152H

**Hydrometer effective depth equation: L =** 16.294964 - 0.164 **x Rm**

| Elapsed Time (min.) | Temp. (deg. C.) | Actual Reading | Corrected Reading | K | Rm | Eff. Depth | Diameter (mm.) | Percent Finer |
|---|---|---|---|---|---|---|---|---|
| 2.00 | 19.0 | 12.5 | 9.2 | 0.0138 | 13.0 | 14.2 | 0.0368 | 9.1 |
| 4.00 | 19.0 | 11.5 | 8.2 | 0.0138 | 12.0 | 14.3 | 0.0261 | 8.1 |
| 8.00 | 19.0 | 11.0 | 7.7 | 0.0138 | 11.5 | 14.4 | 0.0185 | 7.6 |
| 14.00 | 19.0 | 10.5 | 7.2 | 0.0138 | 11.0 | 14.5 | 0.0141 | 7.1 |
| 30.00 | 19.0 | 10.0 | 6.7 | 0.0138 | 10.5 | 14.6 | 0.0096 | 6.6 |
| 60.00 | 19.0 | 9.0 | 5.7 | 0.0138 | 9.5 | 14.7 | 0.0068 | 5.6 |
| 120.00 | 19.5 | 8.5 | 5.3 | 0.0137 | 9.0 | 14.8 | 0.0048 | 5.3 |
| 240.00 | 19.5 | 8.0 | 4.8 | 0.0137 | 8.5 | 14.9 | 0.0034 | 4.8 |
| 480.00 | 19.5 | 7.0 | 3.9 | 0.0137 | 7.5 | 15.1 | 0.0024 | 3.8 |
| 1440.00 | 19.0 | 6.0 | 2.7 | 0.0138 | 6.5 | 15.2 | 0.0014 | 2.7 |

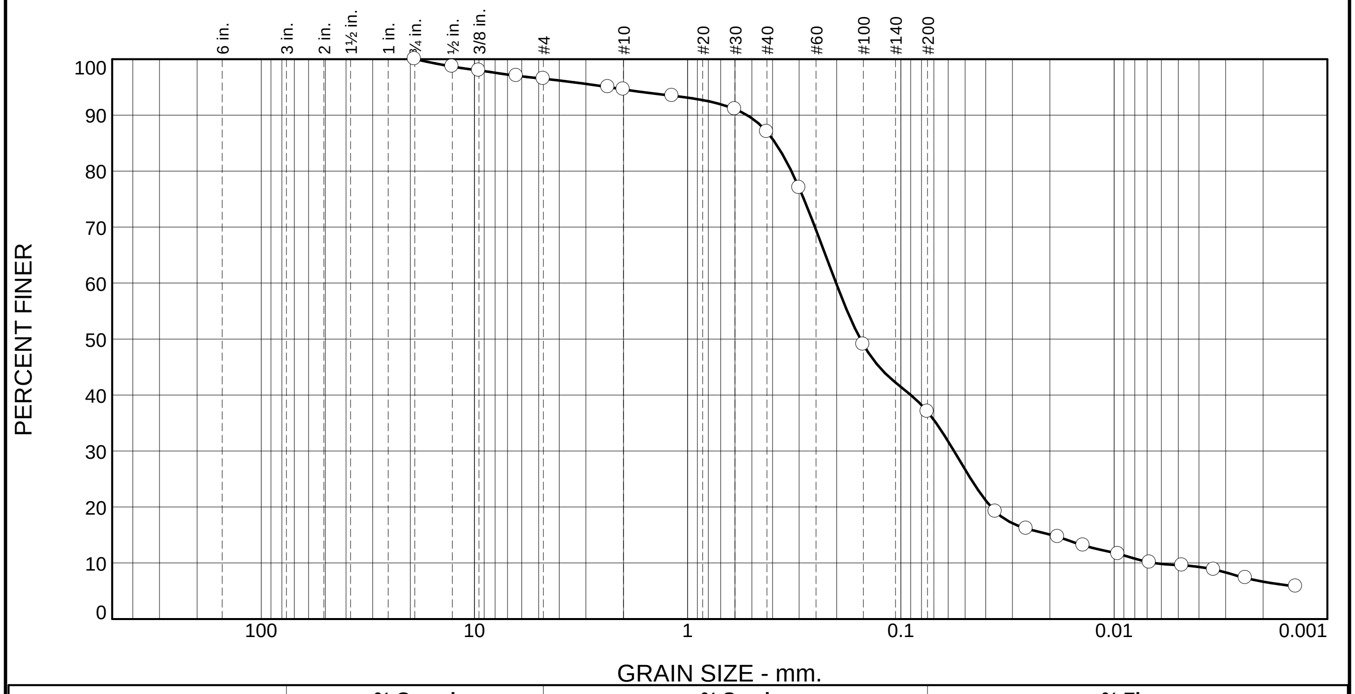
## Fractional Components

| Cobbles | Gravel | | | Sand | | | | Fines | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Total | Coarse | Medium | Fine | Total | Silt | Clay | Total |
| 0.0 | 0.0 | 24.3 | 24.3 | 13.3 | 17.8 | 27.1 | 58.2 | 14.2 | 3.3 | 17.5 |

| $D_{5}$ | $D_{10}$ | $D_{15}$ | $D_{20}$ | $D_{30}$ | $D_{40}$ | $D_{50}$ | $D_{60}$ | $D_{80}$ | $D_{85}$ | $D_{90}$ | $D_{95}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0038 | 0.0410 | 0.0608 | 0.1022 | 0.2253 | 0.3457 | 0.6199 | 1.6721 | 5.8491 | 8.7813 | 10.7938 | 12.8093 |

| Fineness Modulus | $C_{u}$ | $C_{c}$ |
|---|---|---|
| 3.08 | 40.82 | 0.74 |

# Particle Size Distribution Report



| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 3.5 | 1.9 | 7.6 | 50.0 | 30.3 | 6.7 |

**Test Results (ASTM D422 & ASTM D1140)**

| Opening Size | Percent Finer | Spec.* (Percent) | Pass? (X=Fail) |
|---|---|---|---|
| 3/4" | 100.0 | | |
| 1/2" | 98.7 | | |
| 3/8" | 98.0 | | |
| 1/4" | 97.0 | | |
| #4 | 96.5 | | |
| #8 | 95.0 | | |
| #10 | 94.6 | | |
| #16 | 93.5 | | |
| #30 | 91.1 | | |
| #40 | 87.0 | | |
| #50 | 77.0 | | |
| #100 | 49.0 | | |
| #200 | 37.0 | | |
| 0.0360 mm. | 19.2 | | |
| 0.0258 mm. | 16.2 | | |
| 0.0183 mm. | 14.7 | | |
| 0.0139 mm. | 13.1 | | |
| 0.0096 mm. | 11.6 | | |
| 0.0068 mm. | 10.1 | | |
| 0.0048 mm. | 9.6 | | |
| 0.0034 mm. | 8.8 | | |
| 0.0024 mm. | 7.3 | | |
| 0.0014 mm. | 5.8 | | |

* (no specification provided)

## Material Description

Silty fine to medium sand- trace to some clay- trace coarse sand and fine gravel- gray

### Atterberg Limits (ASTM D 4318)

**PL=** NP **LL=** NV **PI=** NP

### Classification

**USCS (D 2487)=** SM **AASHTO (M 145)=** A-4(0)

### Coefficients

$D_{90}$= 0.5262 $D_{85}$= 0.3875 $D_{60}$= 0.2014
$D_{50}$= 0.1549 $D_{30}$= 0.0563 $D_{15}$= 0.0197
$D_{10}$= 0.0065 $C_u$= 30.95 $C_c$= 2.42

### Remarks

Moisture Content: 10.2%

**Date Received:** 10/23/2024 **Date Tested:** 12/8/2024
**Tested By:** Kevin Rautiola
**Checked By:** Chris Abraham, PE
**Title:** QA Manager

**Location:** B-01
**Sample Number:** S-9 **Depth:** 16 to 20 Feet
**Date Sampled:** 10/23/2024

**GEI Consultants of Michigan, P.C.**

**Marquette, Michigan**

**Client:** City of White Cloud
**Project:** White Cloud Dam Disposition Feasibility Study
**Project No:** 2302435

# GRAIN SIZE DISTRIBUTION TEST DATA

**12/9/2024**

**Client:** City of White Cloud
**Project:** White Cloud Dam Disposition Feasibility Study
**Project Number:** 2302435
**Location:** B-01
**Depth:** 16 to 20 Feet **Sample Number:** S-9
**Material Description:** Silty fine to medium sand- trace to some clay- trace coarse sand and fine gravel- gray
**Sample Date:** 10/23/2024
**Date Received:** 10/23/2024 **PL:** NP **LL:** NV **PI:** NP
**USCS Classification:** SM **AASHTO Classification:** A-4(0)
**Grain Size Test Method:** ASTM D422
**#200 Wash Method:** ASTM D1140
**Testing Remarks:** Moisture Content: 10.2%
**Tested By:** Kevin Rautiola **Test Date:** 12/8/2024
**Checked By:** Chris Abraham, PE **Title:** QA Manager

## Sieve Test Data

**Post #200 Wash Test Weights (grams):** **Dry Sample and Tare =** 520.10
**Tare Wt. =** 351.50
**Minus #200 from wash =** 35.8%

| Dry Sample and Tare (grams) | Tare (grams) | Sieve Opening Size | Weight Retained (grams) | Sieve Weight (grams) | Percent Finer |
|---|---|---|---|---|---|
| 614.20 | 351.50 | 1-1/2" | | | |
| | | 1" | | | |
| | | 3/4" | 0.00 | 0.00 | 100.0 |
| | | 1/2" | 3.50 | 0.00 | 98.7 |
| | | 3/8" | 1.80 | 0.00 | 98.0 |
| | | 1/4" | 2.50 | 0.00 | 97.0 |
| | | #4 | 1.40 | 0.00 | 96.5 |
| | | #8 | 3.90 | 0.00 | 95.0 |
| | | #10 | 1.10 | 0.00 | 94.6 |
| | | #16 | 3.00 | 0.00 | 93.5 |
| | | #30 | 6.30 | 0.00 | 91.1 |
| | | #40 | 10.60 | 0.00 | 87.0 |
| | | #50 | 26.30 | 0.00 | 77.0 |
| | | #100 | 73.50 | 0.00 | 49.0 |
| | | #200 | 31.50 | 0.00 | 37.0 |

## Hydrometer Test Data

**Hydrometer test uses material passing** #10

**Percent passing #10 based upon complete sample =** 94.6

**Weight of hydrometer sample =**69.0

**Hygroscopic moisture correction:**

**Moist weight and tare =** 641.10
**Dry weight and tare =** 614.20
**Tare weight =** 351.50
**Hygroscopic moisture =** 10.2%

**Table of composite correction values:**

| | | | | | | |
|---|---|---|---|---|---|---|
| **Temp., deg. C:** | 23.1 | 22.0 | 21.0 | 20.0 | 19.0 | 18.2 |
| **Comp. corr.:** | -2.1 | -2.4 | -2.7 | -3.0 | -3.3 | -3.8 |

**Meniscus correction only =** 0.5

**Specific gravity of solids =** 2.65

**Hydrometer type =** 152H

**Hydrometer effective depth equation: L =** 16.294964 **-** 0.164 **x Rm**

| Elapsed Time (min.) | Temp. (deg. C.) | Actual Reading | Corrected Reading | K | Rm | Eff. Depth | Diameter (mm.) | Percent Finer |
|---|---|---|---|---|---|---|---|---|
| 2.00 | 19.0 | 16.0 | 12.7 | 0.0138 | 16.5 | 13.6 | 0.0360 | 19.2 |
| 4.00 | 19.0 | 14.0 | 10.7 | 0.0138 | 14.5 | 13.9 | 0.0258 | 16.2 |
| 8.00 | 19.0 | 13.0 | 9.7 | 0.0138 | 13.5 | 14.1 | 0.0183 | 14.7 |
| 14.00 | 19.0 | 12.0 | 8.7 | 0.0138 | 12.5 | 14.2 | 0.0139 | 13.1 |
| 30.00 | 19.0 | 11.0 | 7.7 | 0.0138 | 11.5 | 14.4 | 0.0096 | 11.6 |
| 60.00 | 19.0 | 10.0 | 6.7 | 0.0138 | 10.5 | 14.6 | 0.0068 | 10.1 |
| 120.00 | 19.5 | 9.5 | 6.3 | 0.0137 | 10.0 | 14.7 | 0.0048 | 9.6 |
| 240.00 | 19.5 | 9.0 | 5.8 | 0.0137 | 9.5 | 14.7 | 0.0034 | 8.8 |
| 480.00 | 19.5 | 8.0 | 4.8 | 0.0137 | 8.5 | 14.9 | 0.0024 | 7.3 |
| 1440.00 | 19.5 | 7.0 | 3.9 | 0.0137 | 7.5 | 15.1 | 0.0014 | 5.8 |

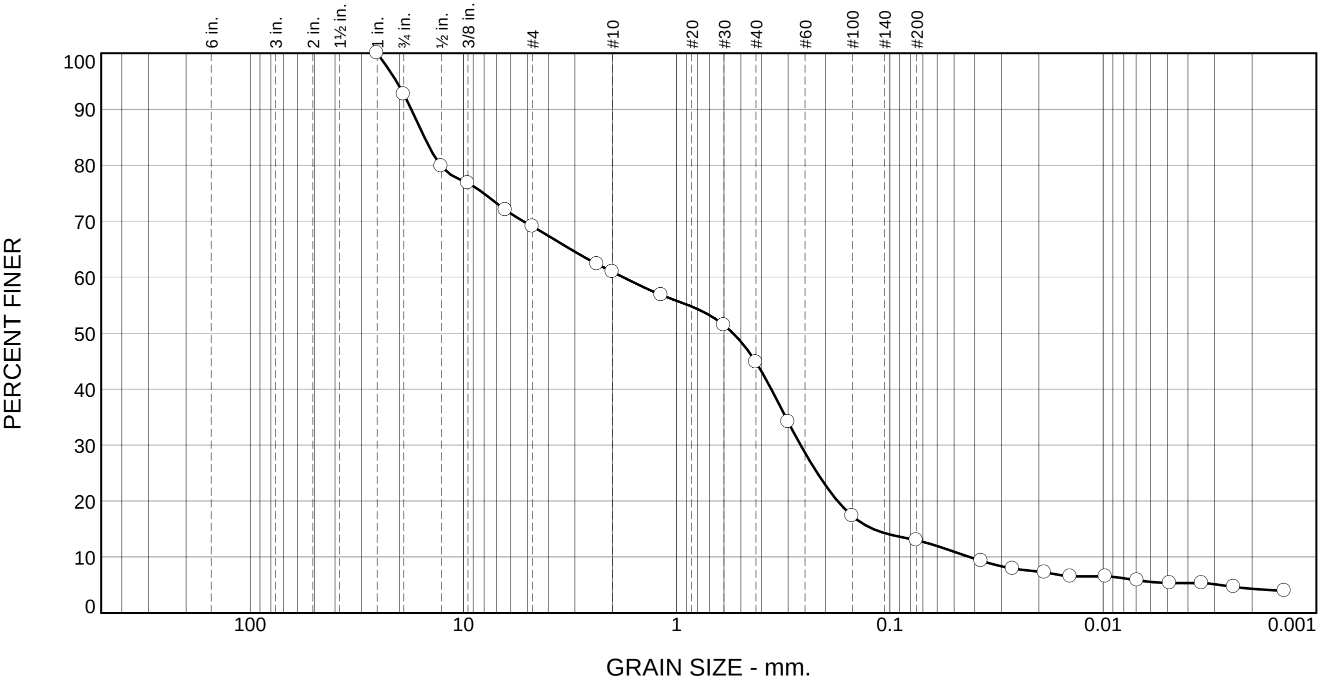
## Fractional Components

| Cobbles | Gravel | | | Sand | | | | Fines | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Total | Coarse | Medium | Fine | Total | Silt | Clay | Total |
| 0.0 | 0.0 | 3.5 | 3.5 | 1.9 | 7.6 | 50.0 | 59.5 | 30.3 | 6.7 | 37.0 |

| $D_{5}$ | $D_{10}$ | $D_{15}$ | $D_{20}$ | $D_{30}$ | $D_{40}$ | $D_{50}$ | $D_{60}$ | $D_{80}$ | $D_{85}$ | $D_{90}$ | $D_{95}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0065 | 0.0197 | 0.0377 | 0.0563 | 0.0898 | 0.1549 | 0.2014 | 0.3263 | 0.3875 | 0.5262 | 2.3475 |

| Fineness Modulus | $C_u$ | $C_c$ |
|---|---|---|
| 1.00 | 30.95 | 2.42 |

# Particle Size Distribution Report



| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 7.3 | 23.6 | 8.2 | 16.1 | 31.8 | 8.7 | 4.3 |

| Test Results (ASTM C136 & ASTM C117) | | | |
|---|---|---|---|
| **Opening Size** | **Percent Finer** | **Spec.* (Percent)** | **Pass? (X=Fail)** |
| 1" | 100.0 | | |
| 3/4" | 92.7 | | |
| 1/2" | 79.8 | | |
| 3/8" | 76.8 | | |
| 1/4" | 72.0 | | |
| #4 | 69.1 | | |
| #8 | 62.3 | | |
| #10 | 60.9 | | |
| #16 | 56.8 | | |
| #30 | 51.4 | | |
| #40 | 44.8 | | |
| #50 | 34.1 | | |
| #100 | 17.3 | | |
| #200 | 13.0 | | |
| 0.0373 mm. | 9.3 | | |
| 0.0265 mm. | 7.9 | | |
| 0.0188 mm. | 7.2 | | |
| 0.0143 mm. | 6.5 | | |
| 0.0097 mm. | 6.5 | | |
| 0.0069 mm. | 5.8 | | |
| 0.0049 mm. | 5.4 | | |
| 0.0034 mm. | 5.4 | | |
| 0.0024 mm. | 4.7 | | |
| 0.0014 mm. | 4.0 | | |

* (no specification provided)

### Material Description

Gravelly fine to coarse sand- trace to some silt- trace clay- brown

### Atterberg Limits (ASTM D 4318)

**PL=** NP **LL=** NV **PI=** NP

### Classification

**USCS (D 2487)=** SM **AASHTO (M 145)=** A-1-b

### Coefficients

| | | | | | |
|---|---|---|---|---|---|
| $D_{90}$= | 17.5883 | $D_{85}$= | 15.2600 | $D_{60}$= | 1.7855 |
| $D_{50}$= | 0.5451 | $D_{30}$= | 0.2622 | $D_{15}$= | 0.1198 |
| $D_{10}$= | 0.0423 | $C_u$= | 42.24 | $C_c$= | 0.91 |

### Remarks

Moisture Content: 16.5%

**Date Received:** 10/23/2024 **Date Tested:** 12/8/2024

**Tested By:** Kevin Rautiola

**Checked By:** Chris Abraham, PE

**Title:** QA Manager

**Location:** B-02

**Sample Number:** S-7 **Depth:** 12 to 14 Feet

**Date Sampled:** 10/23/2024

**GEI Consultants of Michigan, P.C.**

**Marquette, Michigan**

**Client:** City of White Cloud

**Project:** White Cloud Dam Disposition Feasibility Study

**Project No:** 2302435

## GRAIN SIZE DISTRIBUTION TEST DATA

**12/9/2024**

**Client:** City of White Cloud
**Project:** White Cloud Dam Disposition Feasibility Study
**Project Number:** 2302435
**Location:** B-02
**Depth:** 12 to 14 Feet **Sample Number:** S-7
**Material Description:** Gravelly fine to coarse sand- trace to some silt- trace clay- brown
**Sample Date:** 10/23/2024
**Date Received:** 10/23/2024 **PL:** NP **LL:** NV **PI:** NP
**USCS Classification:** SM **AASHTO Classification:** A-1-b
**Grain Size Test Method:** ASTM C136
**#200 Wash Method:** ASTM C117
**Testing Remarks:** Moisture Content: 16.5%
**Tested By:** Kevin Rautiola **Test Date:** 12/8/2024
**Checked By:** Chris Abraham, PE **Title:** QA Manager

### Sieve Test Data

**Post #200 Wash Test Weights (grams):** **Dry Sample and Tare =** 543.20
**Tare Wt. =** 348.60
**Minus #200 from wash =** 11.3%

| Dry Sample and Tare (grams) | Tare (grams) | Sieve Opening Size | Weight Retained (grams) | Sieve Weight (grams) | Percent Finer |
|---|---|---|---|---|---|
| 568.00 | 348.60 | 1-1/2" | | | |
| | | 1" | 0.00 | 0.00 | 100.0 |
| | | 3/4" | 16.10 | 0.00 | 92.7 |
| | | 1/2" | 28.20 | 0.00 | 79.8 |
| | | 3/8" | 6.60 | 0.00 | 76.8 |
| | | 1/4" | 10.60 | 0.00 | 72.0 |
| | | #4 | 6.40 | 0.00 | 69.1 |
| | | #8 | 14.80 | 0.00 | 62.3 |
| | | #10 | 3.00 | 0.00 | 60.9 |
| | | #16 | 9.10 | 0.00 | 56.8 |
| | | #30 | 11.80 | 0.00 | 51.4 |
| | | #40 | 14.50 | 0.00 | 44.8 |
| | | #50 | 23.40 | 0.00 | 34.1 |
| | | #100 | 36.90 | 0.00 | 17.3 |
| | | #200 | 9.50 | 0.00 | 13.0 |

## Hydrometer Test Data

**Hydrometer test uses material passing** #10

**Percent passing #10 based upon complete sample =** 60.9

**Weight of hydrometer sample =**51.0

**Hygroscopic moisture correction:**

| | |
|---|---|
| **Moist weight and tare =** | 604.10 |
| **Dry weight and tare =** | 568.00 |
| **Tare weight =** | 348.60 |
| **Hygroscopic moisture =** | 16.5% |

**Table of composite correction values:**

| | | | | | | |
|---|---|---|---|---|---|---|
| **Temp., deg. C:** | 23.1 | 22.0 | 21.0 | 20.0 | 19.0 | 18.2 |
| **Comp. corr.:** | -2.1 | -2.4 | -2.7 | -3.0 | -3.3 | -3.8 |

**Meniscus correction only =** 0.5

**Specific gravity of solids =** 2.65

**Hydrometer type =** 152H

**Hydrometer effective depth equation: L =** 16.294964 - 0.164 **x Rm**

| Elapsed Time (min.) | Temp. (deg. C.) | Actual Reading | Corrected Reading | K | Rm | Eff. Depth | Diameter (mm.) | Percent Finer |
|---|---|---|---|---|---|---|---|---|
| 2.00 | 19.0 | 10.0 | 6.7 | 0.0138 | 10.5 | 14.6 | 0.0373 | 9.3 |
| 4.00 | 19.0 | 9.0 | 5.7 | 0.0138 | 9.5 | 14.7 | 0.0265 | 7.9 |
| 8.00 | 19.0 | 8.5 | 5.2 | 0.0138 | 9.0 | 14.8 | 0.0188 | 7.2 |
| 14.00 | 19.0 | 8.0 | 4.7 | 0.0138 | 8.5 | 14.9 | 0.0143 | 6.5 |
| 30.00 | 19.0 | 8.0 | 4.7 | 0.0138 | 8.5 | 14.9 | 0.0097 | 6.5 |
| 60.00 | 19.0 | 7.5 | 4.2 | 0.0138 | 8.0 | 15.0 | 0.0069 | 5.8 |
| 120.00 | 19.5 | 7.0 | 3.9 | 0.0137 | 7.5 | 15.1 | 0.0049 | 5.4 |
| 240.00 | 19.5 | 7.0 | 3.9 | 0.0137 | 7.5 | 15.1 | 0.0034 | 5.4 |
| 480.00 | 19.5 | 6.5 | 3.4 | 0.0137 | 7.0 | 15.1 | 0.0024 | 4.7 |
| 1440.00 | 19.5 | 6.0 | 2.9 | 0.0137 | 6.5 | 15.2 | 0.0014 | 4.0 |

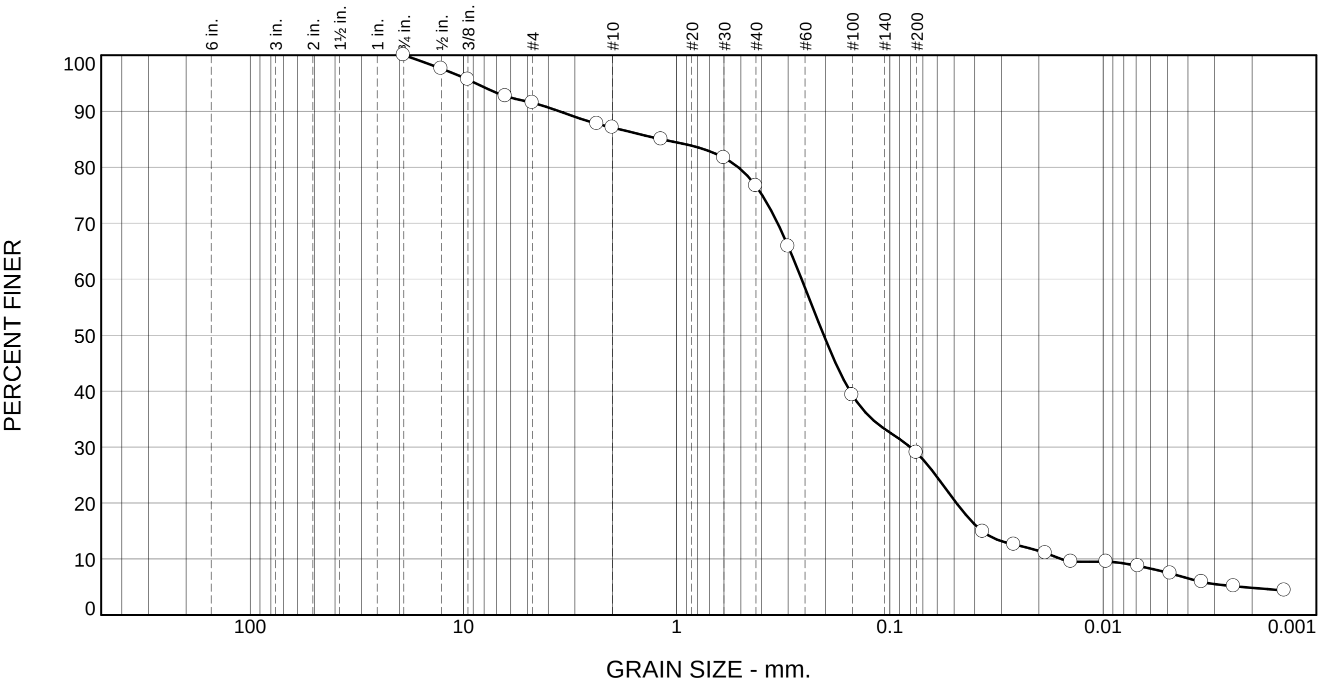
## Fractional Components

| Cobbles | Gravel | | | Sand | | | | Fines | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Total | Coarse | Medium | Fine | Total | Silt | Clay | Total |
| 0.0 | 7.3 | 23.6 | 30.9 | 8.2 | 16.1 | 31.8 | 56.1 | 8.7 | 4.3 | 13.0 |

| $D_{5}$ | $D_{10}$ | $D_{15}$ | $D_{20}$ | $D_{30}$ | $D_{40}$ | $D_{50}$ | $D_{60}$ | $D_{80}$ | $D_{85}$ | $D_{90}$ | $D_{95}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0028 | 0.0423 | 0.1198 | 0.1759 | 0.2622 | 0.3603 | 0.5451 | 1.7855 | 12.8191 | 15.2600 | 17.5883 | 20.6412 |

| Fineness Modulus | $C_u$ | $C_c$ |
|---|---|---|
| 3.40 | 42.24 | 0.91 |

# Particle Size Distribution Report



| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 8.5 | 4.4 | 10.5 | 47.6 | 24.1 | 4.9 |

| Test Results (ASTM C136 & ASTM C117) | | | |
|---|---|---|---|
| **Opening Size** | **Percent Finer** | **Spec.* (Percent)** | **Pass? (X=Fail)** |
| 3/4" | 100.0 | | |
| 1/2" | 97.6 | | |
| 3/8" | 95.6 | | |
| 1/4" | 92.7 | | |
| #4 | 91.5 | | |
| #8 | 87.7 | | |
| #10 | 87.1 | | |
| #16 | 85.0 | | |
| #30 | 81.7 | | |
| #40 | 76.6 | | |
| #50 | 65.9 | | |
| #100 | 39.3 | | |
| #200 | 29.0 | | |
| 0.0367 mm. | 14.9 | | |
| 0.0261 mm. | 12.6 | | |
| 0.0186 mm. | 11.1 | | |
| 0.0141 mm. | 9.5 | | |
| 0.0097 mm. | 9.5 | | |
| 0.0068 mm. | 8.7 | | |
| 0.0048 mm. | 7.4 | | |
| 0.0034 mm. | 5.9 | | |
| 0.0024 mm. | 5.1 | | |
| 0.0014 mm. | 4.4 | | |

* (no specification provided)

### Material Description

Fine to medium sand- some silt- trace to some fine gravel- trace clay- gray

### Atterberg Limits (ASTM D 4318)

**PL=** NP **LL=** NV **PI=** NP

### Classification

**USCS (D 2487)=** SM **AASHTO (M 145)=** A-2-4(0)

### Coefficients

| | | |
|---|---|---|
| $D_{90}$= 3.5667 | $D_{85}$= 1.1878 | $D_{60}$= 0.2595 |
| $D_{50}$= 0.2044 | $D_{30}$= 0.0803 | $D_{15}$= 0.0370 |
| $D_{10}$= 0.0157 | $C_u$= 16.51 | $C_c$= 1.58 |

### Remarks

Moisture Content: 9.8%

**Date Received:** 10/23/2024 **Date Tested:** 12/8/2024
**Tested By:** Kevin Rautiola
**Checked By:** Chris Abraham, PE
**Title:** QA Manager

**Location:** B-02
**Sample Number:** S-13 **Depth:** 33 to 35 Feet
**Date Sampled:** 10/23/2024

**GEI Consultants of Michigan, P.C.**

**Marquette, Michigan**

**Client:** City of White Cloud
**Project:** White Cloud Dam Disposition Feasibility Study

**Project No:** 2302435

## GRAIN SIZE DISTRIBUTION TEST DATA

**12/9/2024**

**Client:** City of White Cloud
**Project:** White Cloud Dam Disposition Feasibility Study
**Project Number:** 2302435
**Location:** B-02
**Depth:** 33 to 35 Feet **Sample Number:** S-13
**Material Description:** Fine to medium sand- some silt- trace to some fine gravel- trace clay- gray
**Sample Date:** 10/23/2024
**Date Received:** 10/23/2024 **PL:** NP **LL:** NV **PI:** NP
**USCS Classification:** SM **AASHTO Classification:** A-2-4(0)
**Grain Size Test Method:** ASTM C136
**#200 Wash Method:** ASTM C117
**Testing Remarks:** Moisture Content: 9.8%
**Tested By:** Kevin Rautiola **Test Date:** 12/8/2024
**Checked By:** Chris Abraham, PE **Title:** QA Manager

### Sieve Test Data

**Post #200 Wash Test Weights (grams):** **Dry Sample and Tare =** 552.90
**Tare Wt. =** 348.70
**Minus #200 from wash =** 27.8%

| Dry Sample and Tare (grams) | Tare (grams) | Sieve Opening Size | Weight Retained (grams) | Sieve Weight (grams) | Percent Finer |
|---|---|---|---|---|---|
| 631.60 | 348.70 | 1-1/2" | | | |
| | | 1" | | | |
| | | 3/4" | 0.00 | 0.00 | 100.0 |
| | | 1/2" | 6.80 | 0.00 | 97.6 |
| | | 3/8" | 5.60 | 0.00 | 95.6 |
| | | 1/4" | 8.30 | 0.00 | 92.7 |
| | | #4 | 3.40 | 0.00 | 91.5 |
| | | #8 | 10.60 | 0.00 | 87.7 |
| | | #10 | 1.90 | 0.00 | 87.1 |
| | | #16 | 5.90 | 0.00 | 85.0 |
| | | #30 | 9.40 | 0.00 | 81.7 |
| | | #40 | 14.20 | 0.00 | 76.6 |
| | | #50 | 30.50 | 0.00 | 65.9 |
| | | #100 | 75.20 | 0.00 | 39.3 |
| | | #200 | 29.10 | 0.00 | 29.0 |

## Hydrometer Test Data

**Hydrometer test uses material passing** #10

**Percent passing #10 based upon complete sample =** 87.1

**Weight of hydrometer sample =**62.3

**Hygroscopic moisture correction:**

**Moist weight and tare =** 659.40

**Dry weight and tare =** 631.60

**Tare weight =** 348.70

**Hygroscopic moisture =** 9.8%

**Table of composite correction values:**

| | | | | | | |
|---|---|---|---|---|---|---|
| **Temp., deg. C:** | 23.1 | 22.0 | 21.0 | 20.0 | 19.0 | 18.2 |
| **Comp. corr.:** | -2.1 | -2.4 | -2.7 | -3.0 | -3.3 | -3.8 |

**Meniscus correction only =** 0.5

**Specific gravity of solids =** 2.65

**Hydrometer type =** 152H

**Hydrometer effective depth equation: L =** 16.294964 **-** 0.164 **x Rm**

| Elapsed Time (min.) | Temp. (deg. C.) | Actual Reading | Corrected Reading | K | Rm | Eff. Depth | Diameter (mm.) | Percent Finer |
|---|---|---|---|---|---|---|---|---|
| 2.00 | 19.0 | 13.0 | 9.7 | 0.0138 | 13.5 | 14.1 | 0.0367 | 14.9 |
| 4.00 | 19.0 | 11.5 | 8.2 | 0.0138 | 12.0 | 14.3 | 0.0261 | 12.6 |
| 8.00 | 19.0 | 10.5 | 7.2 | 0.0138 | 11.0 | 14.5 | 0.0186 | 11.1 |
| 14.00 | 19.0 | 9.5 | 6.2 | 0.0138 | 10.0 | 14.7 | 0.0141 | 9.5 |
| 30.00 | 19.0 | 9.5 | 6.2 | 0.0138 | 10.0 | 14.7 | 0.0097 | 9.5 |
| 60.00 | 19.0 | 9.0 | 5.7 | 0.0138 | 9.5 | 14.7 | 0.0068 | 8.7 |
| 120.00 | 19.5 | 8.0 | 4.8 | 0.0137 | 8.5 | 14.9 | 0.0048 | 7.4 |
| 240.00 | 19.5 | 7.0 | 3.9 | 0.0137 | 7.5 | 15.1 | 0.0034 | 5.9 |
| 480.00 | 19.5 | 6.5 | 3.4 | 0.0137 | 7.0 | 15.1 | 0.0024 | 5.1 |
| 1440.00 | 19.5 | 6.0 | 2.9 | 0.0137 | 6.5 | 15.2 | 0.0014 | 4.4 |

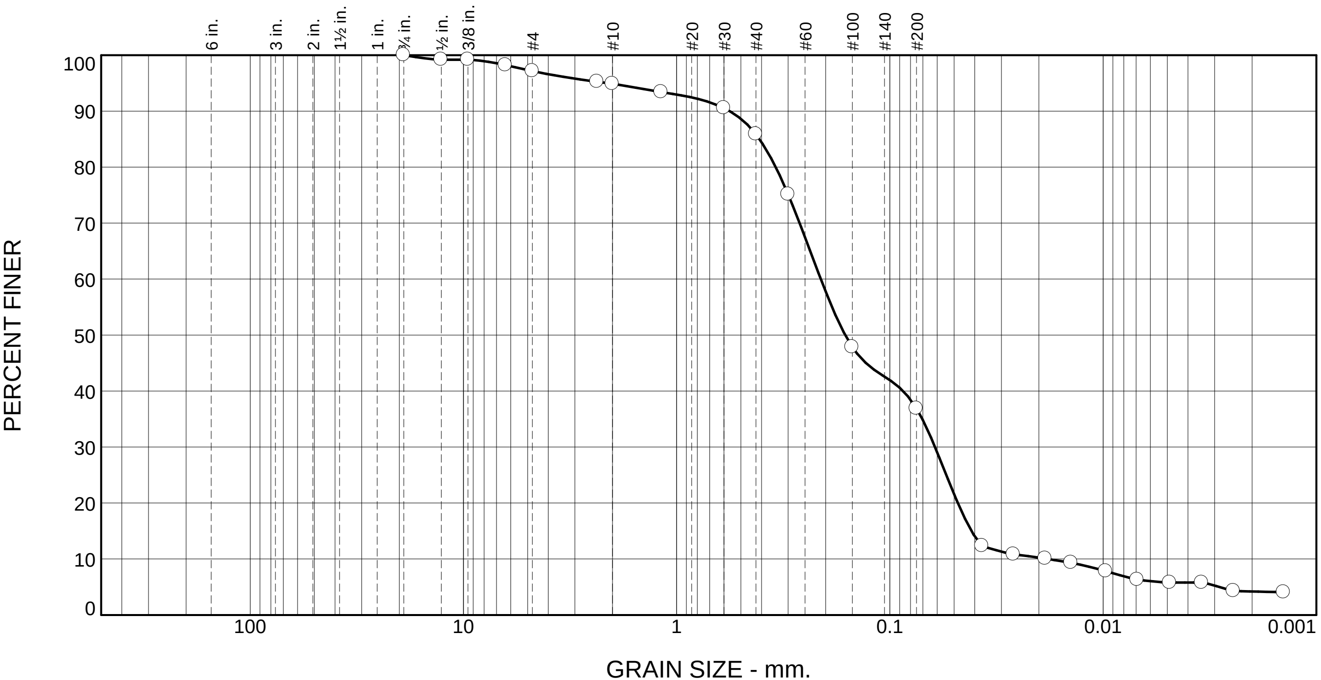
## Fractional Components

| Cobbles | Gravel | | | Sand | | | | Fines | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Total | Coarse | Medium | Fine | Total | Silt | Clay | Total |
| 0.0 | 0.0 | 8.5 | 8.5 | 4.4 | 10.5 | 47.6 | 62.5 | 24.1 | 4.9 | 29.0 |

| $D_{5}$ | $D_{10}$ | $D_{15}$ | $D_{20}$ | $D_{30}$ | $D_{40}$ | $D_{50}$ | $D_{60}$ | $D_{80}$ | $D_{85}$ | $D_{90}$ | $D_{95}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0022 | 0.0157 | 0.0370 | 0.0487 | 0.0803 | 0.1541 | 0.2044 | 0.2595 | 0.5155 | 1.1878 | 3.5667 | 8.7972 |

| Fineness Modulus | $C_u$ | $C_c$ |
|---|---|---|
| 1.53 | 16.51 | 1.58 |

# Particle Size Distribution Report



| % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| 0.0 | 0.0 | 2.8 | 2.3 | 9.0 | 49.0 | 32.7 | 4.2 |

**Test Results (ASTM C136 & ASTM C117)**

| Opening Size | Percent Finer | Spec.* (Percent) | Pass? (X=Fail) |
|---|---|---|---|
| 3/4" | 100.0 | | |
| 1/2" | 99.2 | | |
| 3/8" | 99.2 | | |
| 1/4" | 98.2 | | |
| #4 | 97.2 | | |
| #8 | 95.2 | | |
| #10 | 94.9 | | |
| #16 | 93.4 | | |
| #30 | 90.5 | | |
| #40 | 85.9 | | |
| #50 | 75.1 | | |
| #100 | 47.9 | | |
| #200 | 36.9 | | |
| 0.0370 mm. | 12.3 | | |
| 0.0263 mm. | 10.8 | | |
| 0.0186 mm. | 10.1 | | |
| 0.0141 mm. | 9.3 | | |
| 0.0097 mm. | 7.8 | | |
| 0.0069 mm. | 6.3 | | |
| 0.0049 mm. | 5.8 | | |
| 0.0034 mm. | 5.8 | | |
| 0.0024 mm. | 4.3 | | |
| 0.0014 mm. | 4.1 | | |

* (no specification provided)

### Material Description

Silty fine to medium sand- trace clay, coarse sand, and fine gravel- gray

### Atterberg Limits (ASTM D 4318)

**PL=** NP **LL=** NV **PI=** NP

### Classification

**USCS (D 2487)=** SM **AASHTO (M 145)=** A-4(0)

### Coefficients

| | | | | | |
|---|---|---|---|---|---|
| $D_{90}$= | 0.5648 | $D_{85}$= | 0.4090 | $D_{60}$= | 0.2109 |
| $D_{50}$= | 0.1620 | $D_{30}$= | 0.0614 | $D_{15}$= | 0.0415 |
| $D_{10}$= | 0.0181 | $C_u$= | 11.64 | $C_c$= | 0.99 |

### Remarks

Moisture Content: 10.1%

**Date Received:** 10/23/2024 **Date Tested:** 12/8/2024

**Tested By:** Kevin Rautiola

**Checked By:** Chris Abraham, PE

**Title:** QA Manager

**Location:** B-03

**Sample Number:** S-11 **Depth:** 23 to 25 Feet

**Date Sampled:** 10/23/2024

**GEI Consultants of Michigan, P.C.**

**Marquette, Michigan**

**Client:** City of White Cloud

**Project:** White Cloud Dam Disposition Feasibility Study

**Project No:** 2302435

# GRAIN SIZE DISTRIBUTION TEST DATA

**12/9/2024**

**Client:** City of White Cloud
**Project:** White Cloud Dam Disposition Feasibility Study
**Project Number:** 2302435
**Location:** B-03
**Depth:** 23 to 25 Feet **Sample Number:** S-11
**Material Description:** Silty fine to medium sand- trace clay, coarse sand, and fine gravel- gray
**Sample Date:** 10/23/2024
**Date Received:** 10/23/2024 **PL:** NP **LL:** NV **PI:** NP
**USCS Classification:** SM **AASHTO Classification:** A-4(0)
**Grain Size Test Method:** ASTM C136
**#200 Wash Method:** ASTM C117
**Testing Remarks:** Moisture Content: 10.1%
**Tested By:** Kevin Rautiola **Test Date:** 12/8/2024
**Checked By:** Chris Abraham, PE **Title:** QA Manager

## Sieve Test Data

**Post #200 Wash Test Weights (grams):** **Dry Sample and Tare =** 555.00
**Tare Wt. =** 349.50
**Minus #200 from wash =** 35.7%

| Dry Sample and Tare (grams) | Tare (grams) | Sieve Opening Size | Weight Retained (grams) | Sieve Weight (grams) | Percent Finer |
|---|---|---|---|---|---|
| 669.20 | 349.50 | 1-1/2" | | | |
| | | 1" | | | |
| | | 3/4" | 0.00 | 0.00 | 100.0 |
| | | 1/2" | 2.60 | 0.00 | 99.2 |
| | | 3/8" | 0.00 | 0.00 | 99.2 |
| | | 1/4" | 3.10 | 0.00 | 98.2 |
| | | #4 | 3.40 | 0.00 | 97.2 |
| | | #8 | 6.10 | 0.00 | 95.2 |
| | | #10 | 1.20 | 0.00 | 94.9 |
| | | #16 | 4.70 | 0.00 | 93.4 |
| | | #30 | 9.20 | 0.00 | 90.5 |
| | | #40 | 14.90 | 0.00 | 85.9 |
| | | #50 | 34.30 | 0.00 | 75.1 |
| | | #100 | 87.20 | 0.00 | 47.9 |
| | | #200 | 35.10 | 0.00 | 36.9 |

## Hydrometer Test Data

**Hydrometer test uses material passing** #10

**Percent passing #10 based upon complete sample =** 94.9

**Weight of hydrometer sample =**69.4

**Hygroscopic moisture correction:**

**Moist weight and tare =** 701.40

**Dry weight and tare =** 669.20

**Tare weight =** 349.50

**Hygroscopic moisture =** 10.1%

**Table of composite correction values:**

| | | | | | | |
|---|---|---|---|---|---|---|
| **Temp., deg. C:** | 23.1 | 22.0 | 21.0 | 20.0 | 19.0 | 18.2 |
| **Comp. corr.:** | -2.1 | -2.4 | -2.7 | -3.0 | -3.3 | -3.8 |

**Meniscus correction only =** 0.5

**Specific gravity of solids =** 2.65

**Hydrometer type =** 152H

**Hydrometer effective depth equation: L =** 16.294964 **-** 0.164 **x Rm**

| Elapsed Time (min.) | Temp. (deg. C.) | Actual Reading | Corrected Reading | K | Rm | Eff. Depth | Diameter (mm.) | Percent Finer |
|---|---|---|---|---|---|---|---|---|
| 2.00 | 19.0 | 11.5 | 8.2 | 0.0138 | 12.0 | 14.3 | 0.0370 | 12.3 |
| 4.00 | 19.0 | 10.5 | 7.2 | 0.0138 | 11.0 | 14.5 | 0.0263 | 10.8 |
| 8.00 | 19.0 | 10.0 | 6.7 | 0.0138 | 10.5 | 14.6 | 0.0186 | 10.1 |
| 14.00 | 19.0 | 9.5 | 6.2 | 0.0138 | 10.0 | 14.7 | 0.0141 | 9.3 |
| 30.00 | 19.0 | 8.5 | 5.2 | 0.0138 | 9.0 | 14.8 | 0.0097 | 7.8 |
| 60.00 | 19.0 | 7.5 | 4.2 | 0.0138 | 8.0 | 15.0 | 0.0069 | 6.3 |
| 120.00 | 19.5 | 7.0 | 3.9 | 0.0137 | 7.5 | 15.1 | 0.0049 | 5.8 |
| 240.00 | 19.5 | 7.0 | 3.9 | 0.0137 | 7.5 | 15.1 | 0.0034 | 5.8 |
| 480.00 | 19.5 | 6.0 | 2.9 | 0.0137 | 6.5 | 15.2 | 0.0024 | 4.3 |
| 1440.00 | 19.0 | 6.0 | 2.7 | 0.0138 | 6.5 | 15.2 | 0.0014 | 4.1 |

## Fractional Components

| Cobbles | Gravel | | | Sand | | | | Fines | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Coarse | Fine | Total | Coarse | Medium | Fine | Total | Silt | Clay | Total |
| 0.0 | 0.0 | 2.8 | 2.8 | 2.3 | 9.0 | 49.0 | 60.3 | 32.7 | 4.2 | 36.9 |

| $D_{5}$ | $D_{10}$ | $D_{15}$ | $D_{20}$ | $D_{30}$ | $D_{40}$ | $D_{50}$ | $D_{60}$ | $D_{80}$ | $D_{85}$ | $D_{90}$ | $D_{95}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0028 | 0.0181 | 0.0415 | 0.0480 | 0.0614 | 0.0867 | 0.1620 | 0.2109 | 0.3432 | 0.4090 | 0.5648 | 2.1138 |

| Fineness Modulus | $C_{u}$ | $C_{c}$ |
|---|---|---|
| 1.02 | 11.64 | 0.99 |

# Appendix C

## Photographs

# White Cloud Dam Inspection Photos

**Date:** 10/21/2024-10/22/2024
**GEI Project No.:** 2302435
**Client:** City of White Cloud

# White Cloud Dam Inspection Photos

**Date:** 10/21/2024-10/22/2024
**GEI Project No.:** 2302435
**Client:** City of White Cloud

Photo No.  1 – Pearson Drilling Starting Boring B-01 by Coring through HMA and RCC

Photo No.  2 – Top of RCC Core from B-01

Photo No.  3 – Side/Section of RCC Core from B-01

Photo No.  4 – Degraded RCC Layer from B-01

# White Cloud Dam Inspection Photos

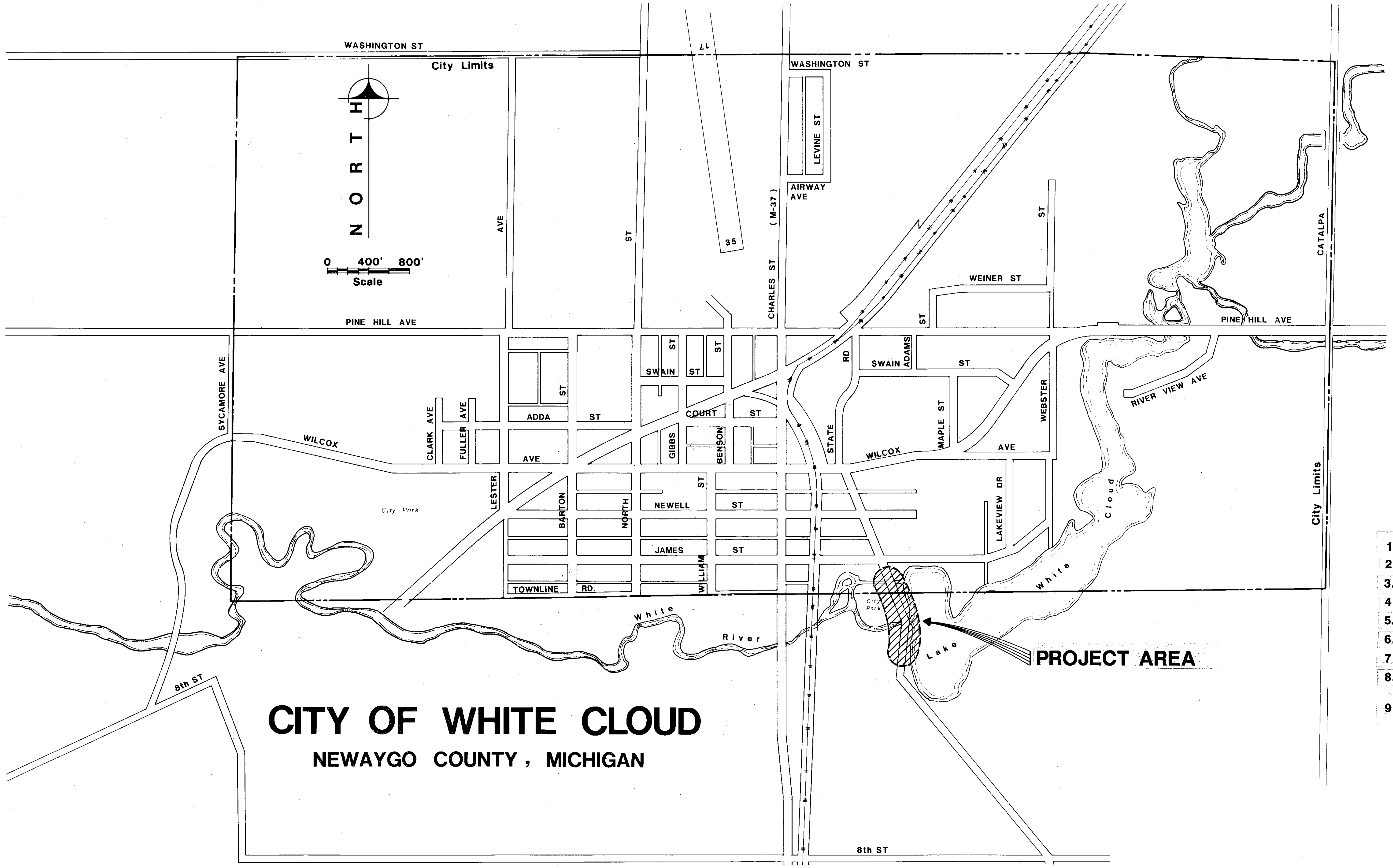
**Date:** 10/21/2024-10/22/2024
**GEI Project No.:** 2302435
**Client:** City of White Cloud

Photo No.  5 – Image Showing Thickness of HMA and RCC Layers

# Appendix D

## Historical Reference Documents

# WHITE CLOUD DAM RECONSTRUCTION



## CITY OF WHITE CLOUD

### NEWAYGO COUNTY, MICHIGAN

**SHEET INDEX**

1. Cover Sheet
2. Site Plan
3. Embankment Reconstruction
4. Embankment Cross Sections
5. Fencing and Wingwall Repair: Principal Spillway
6. Slide Gates: Principal Spillway
7. Catwalks: Principal Spillway
8. Slide Gate, Catwalk and Channel Wall Repairs: Auxiliary Spillway
9. Bar Grate and Retaining Wall Details

*NOTE: Revised December, 1989 to reflect D.N.R. review comments.*

OM

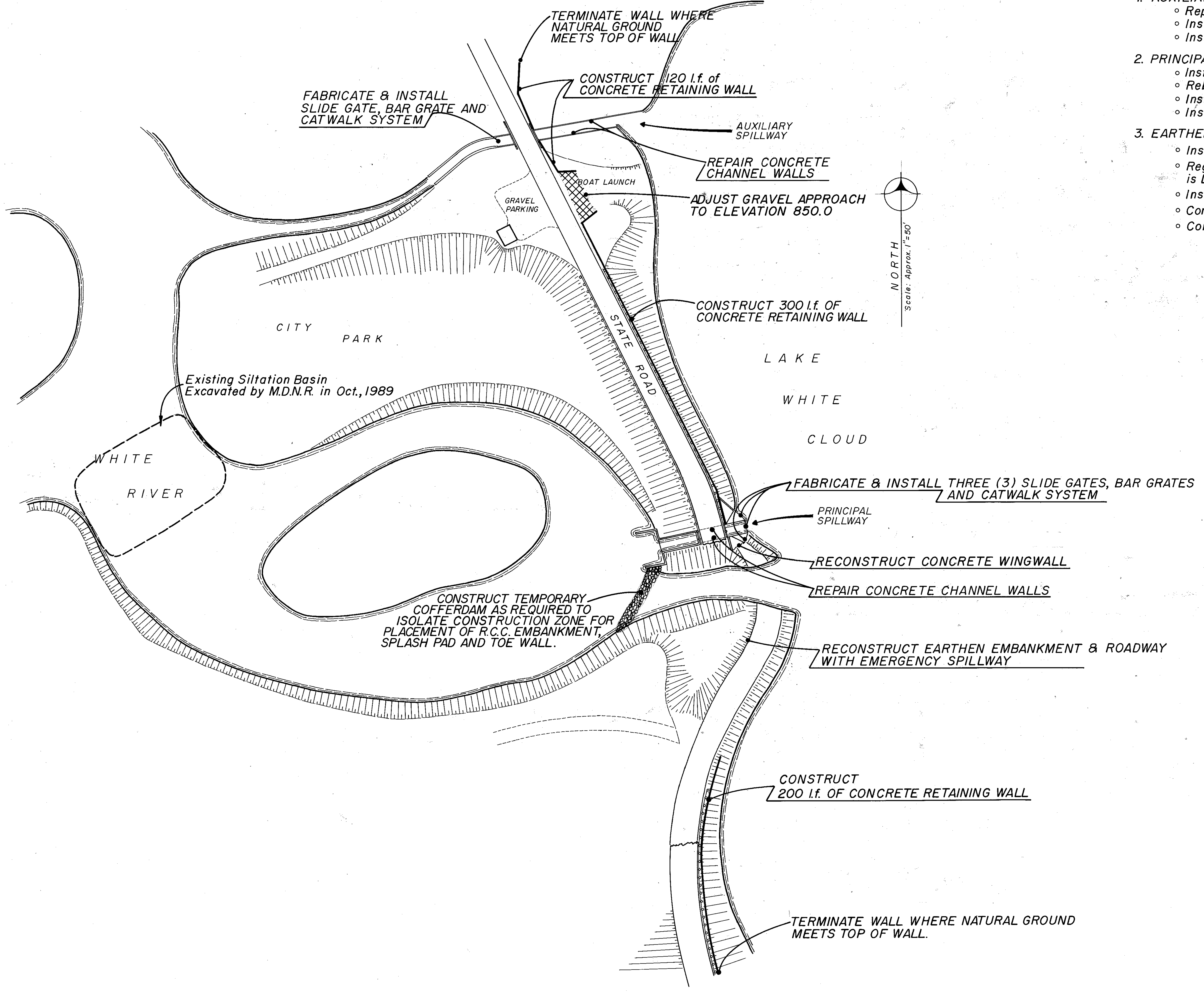
*olson, meyers & may, inc.*
*civil engineers*
*1550 e. beltline se*
*grand rapids, mi. 49506*
*616-957-4350*

project number 01355

sheet number 1 of 9

## CONSTRUCTION PRIORITY SCHEDULE

1. AUXILIARY SPILLWAY AREA
   - Repair Concrete Channel Walls
   - Install Slide Gate & Bar Grate
   - Install Catwalk
2. PRINCIPAL SPILLWAY AREA
   - Install Temporary Sheeting
   - Rebuild South Wingwall
   - Install Slide Gates & Bar Grates
   - Install Catwalk System
3. EARTHEN EMBANKMENT AREA
   - Install Temporary Culverts
   - Regulate Flow Through Culverts Until Flow is Diverted to Auxiliary Spillway
   - Install Cofferdam
   - Construct Emergency Spillway
   - Construct Retaining Walls



olson, meyers & may, inc.
civil engineers
1550 e. beltline se
grand rapids, mi. 49506
616-957-4350

sewer permit number
water permit number
approved by
name date
revised in accordance w/ construction records by date

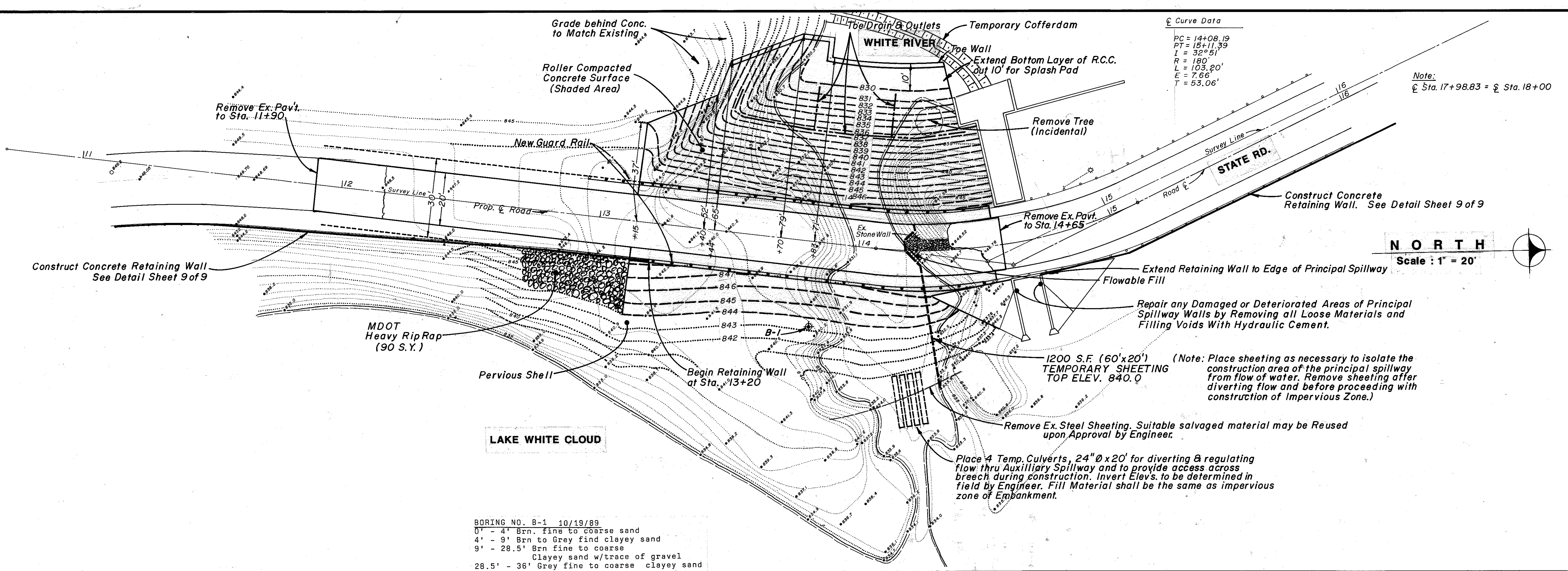
| revisions | by | date |
|---|---|---|
| Revised December, 1989 to reflect D.N.R. review comments | E.J. | 12-89 |

**CITY OF WHITE CLOUD**
**Newaygo County, Michigan**

# WHITE CLOUD DAM RECONSTRUCTION

drawn by E.J. date 8-87
checked by D.G.M. date 11-89
project number 01355
sheet number 2 of 9

SITE PLAN



(Note: Place sheeting as necessary to isolate the construction area of the principal spillway from flow of water. Remove sheeting after diverting flow and before proceeding with construction of Impervious Zone.)

℄ Curve Data

PC = 14+08.19
PT = 15+11.39
I = 32°51'
R = 180'
L = 103.20'
E = 7.66'
T = 53.06'

Note:
℄ Sta. 17+98.83 = ℄ Sta. 18+00

BORING NO. B-1 10/19/89
0' - 4' Brn. fine to coarse sand
4' - 9' Brn to Grey find clayey sand
9' - 28.5' Brn fine to coarse
Clayey sand w/trace of gravel
28.5' - 36' Grey fine to coarse clayey sand

**olson, meyers & may, inc.**
civil engineers
1550 e. beltline se
grand rapids, mi. 49506
616-957-4350

| | | |
|---|---|---|
| Revised December, 1989 to reflect DNR review comments. | E.J. | 12-89 |
| revisions | by | date |

sewer permit number
water permit number
approved by
name ________ date ________
revised in accordance w/ construction records by ______ date ______

**CITY OF WHITE CLOUD**
**Newaygo County, Michigan**

## WHITE CLOUD DAM RECONSTRUCTION

| drawn by | date |
|---|---|
| E.J. | 10-89 |

| checked by | date |
|---|---|
| D.G.M. | 11-89 |

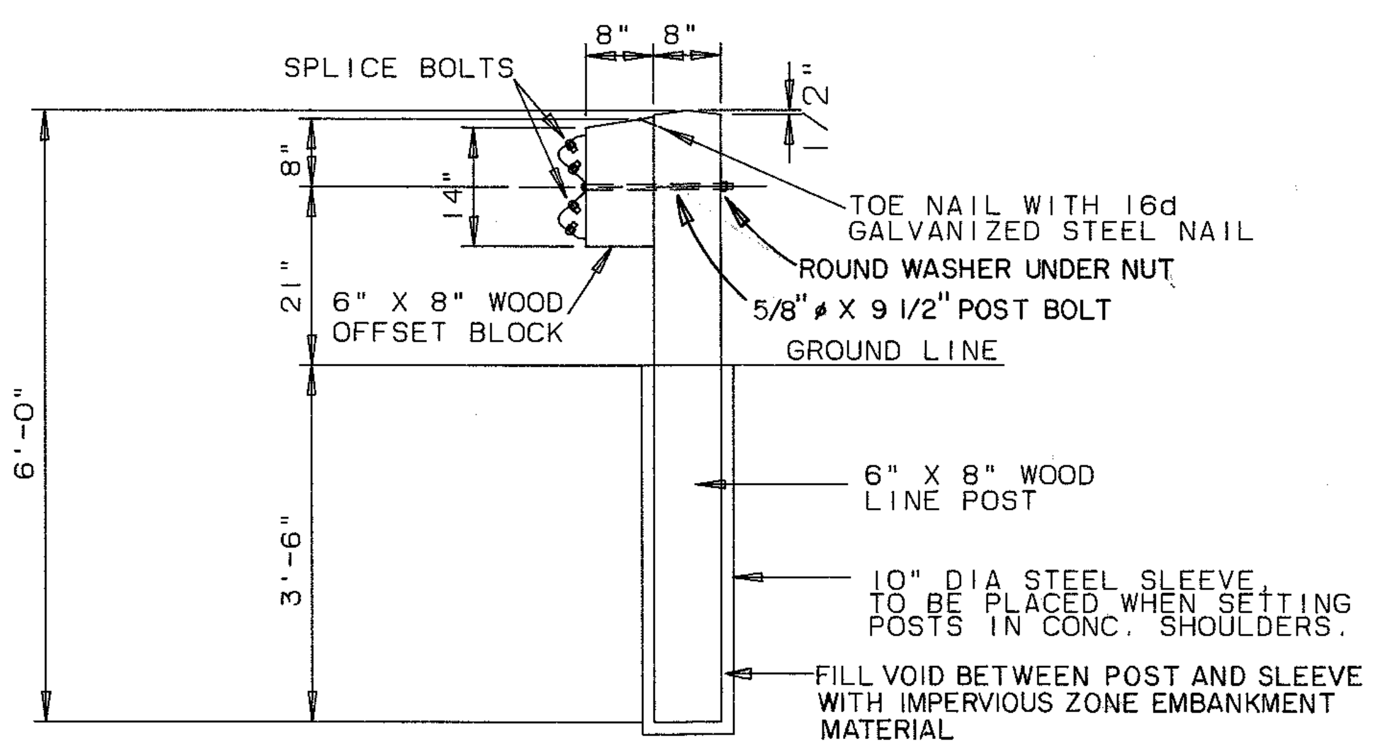
project number
01355

sheet number
3 of 9

Dead file

**EMBANKMENT RECONSTRUCTION**

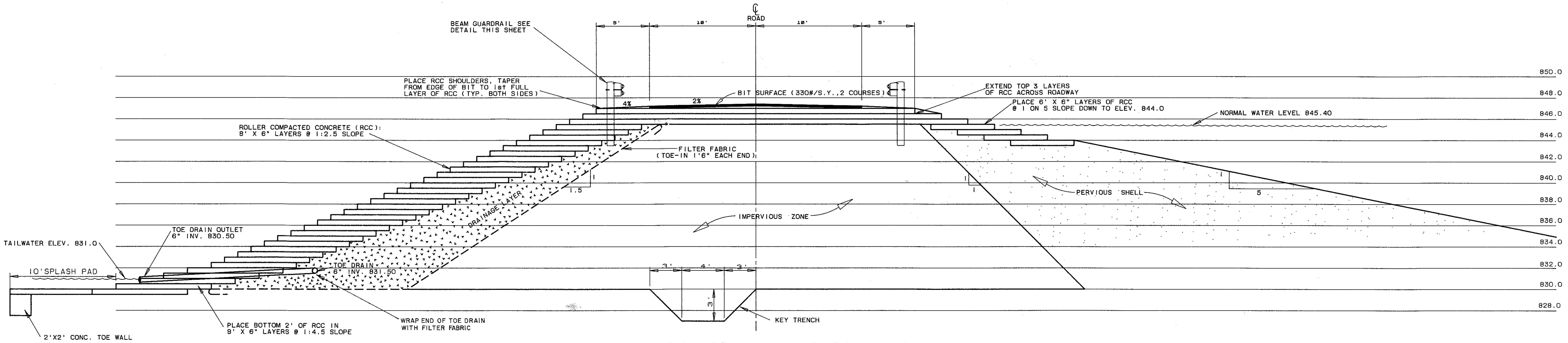
513881



## TYPICAL MDOT - TYPE B BEAM GUARDRAIL 8 POST

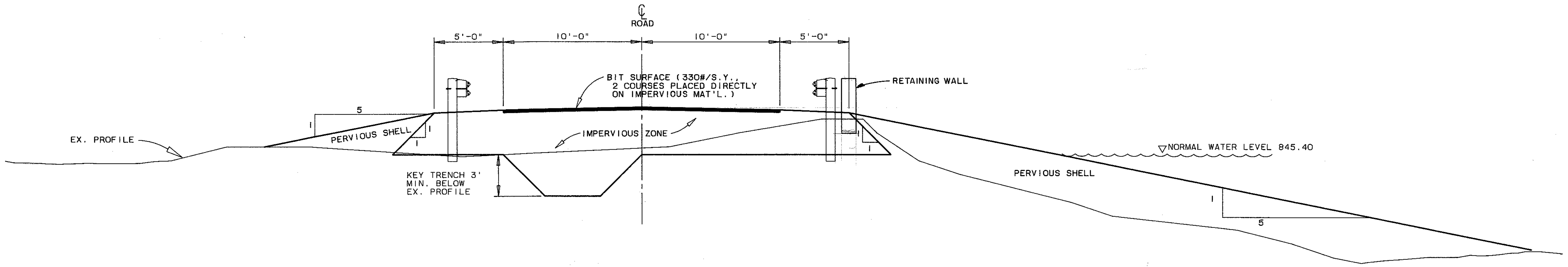
TYPICAL POST SPACING = 6'-3"

NO SCALE



## EMERGENCY SPILLWAY @ Sta. 14+00

TYPICAL SECTION THRU EMBANKMENT
SCALE: 1" = 5'



## TYPICAL SECTION THRU EMBANKMENT @ Sta. 13+00

OUTSIDE OF EMERGENCY SPILLWAY
SCALE 1" = 5'

olson meyers & may, inc.
civil engineers/surveyors
1550 e. beltline se
grand rapids, mi. 49506
616-957-4350

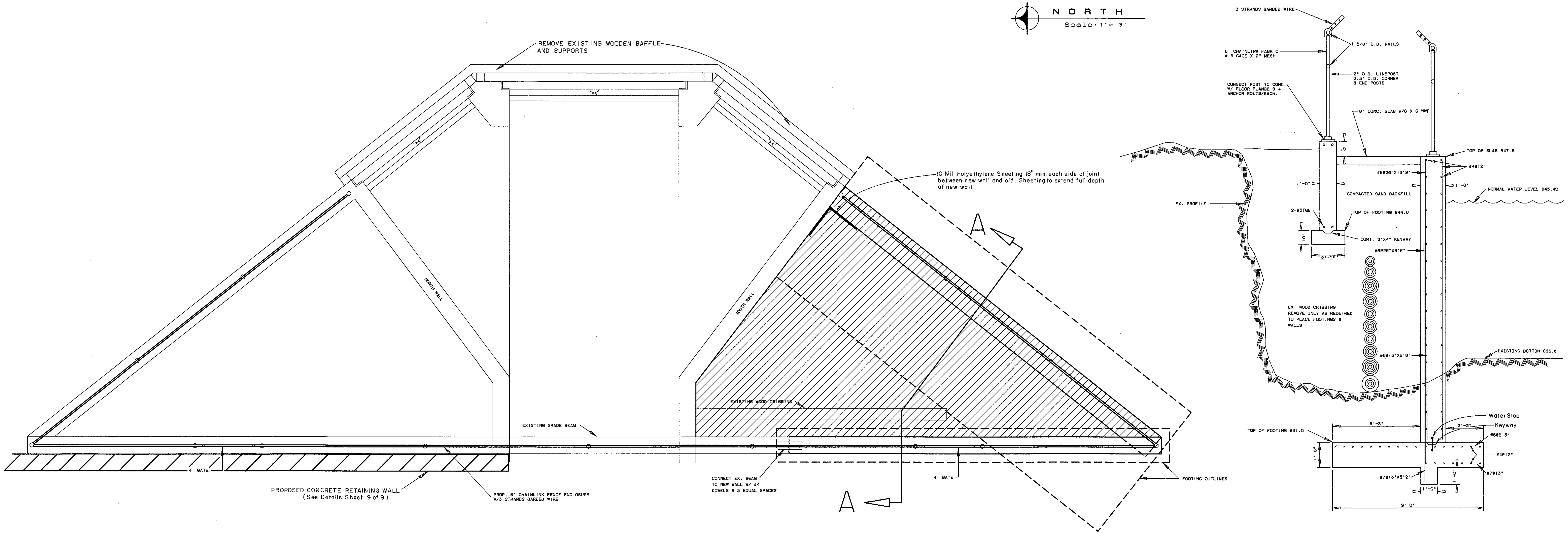
| | | | |
|---|---|---|---|
| Revised December 1989 to reflect DNR review comments. | E.J. | 12-89 | |
| revisions | by | date | |

sewer permit number
water permit number
approved by
name ____ date ____
revised in accordance by ____
w/construction records date ____

CITY OF WHITE CLOUD
Newaygo County, Michigan
**WHITE CLOUD DAM RECONSTRUCTION**

drawn by date
M C S 10/31/89
checked by date
D G M 10/31/89
project number
1355
sheet number
4 of 9
C A D

**EMBANKMENT CROSS-SECTIONS**



## SECTION A-A

Notes:

① Existing fence and handrails to be removed. Remnants imbedded in existing concrete shall be cut off at grade.

② Existing concrete surfaces shall be brushed and all loose material removed. Spalled or broken areas shall be repaired with an epoxy based grout to provide a surface adequate for mounting all accessories ( Gates, Catwalks, Fences, Etc. )

▨ NEW CONCRETE CONSTRUCTION

olson meyers & may, inc.
civil engineers/surveyors
1550 e. beltline se
grand rapids, mi. 49506
616-957-4350

sewer permit number
water permit number
approved by
name ____ date ____
revised in accordance by ____
w/construction records date ____

| revisions | by | date |
|---|---|---|
| Revised December, 1989 to reflect DNR review comments | E.J. | 12-89 |

CITY OF WHITE CLOUD
Newaygo County, Michigan

# WHITE CLOUD DAM RECONSTRUCTION

| drawn by | date |
|---|---|
| M C S | 11/06/89 |

| checked by | date |
|---|---|
| J L R | 11/06/89 |

project number
1355

sheet number
5 of 9

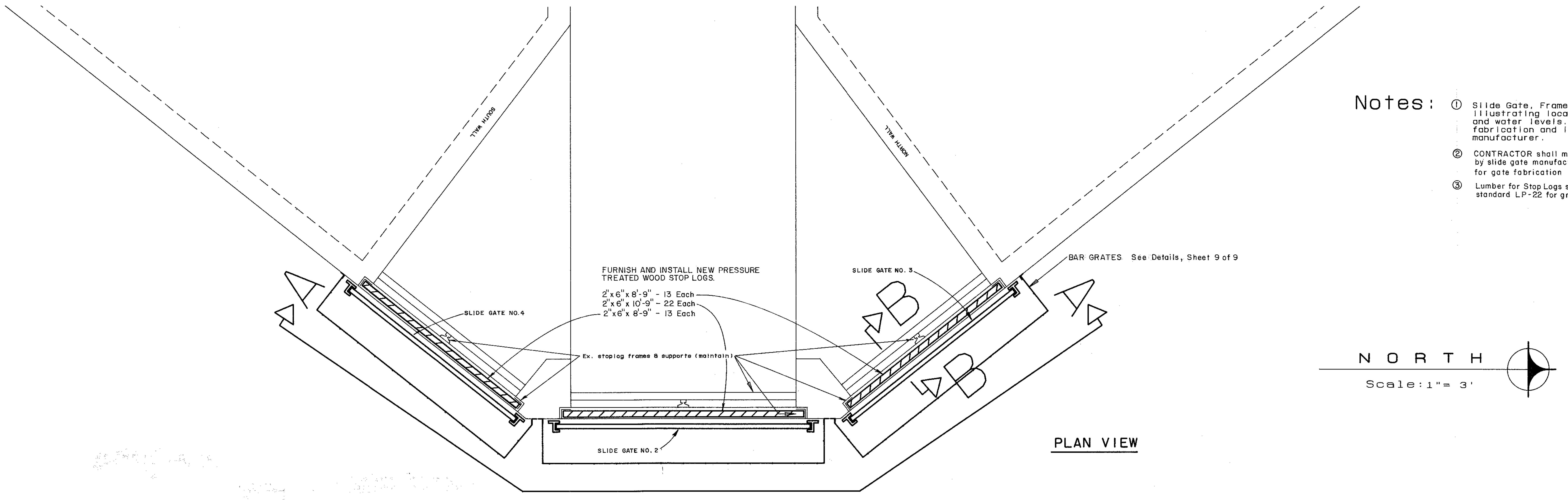
CAD

**FENCING AND WINGWALL REPAIR: PRINCIPAL SPILLWAY**

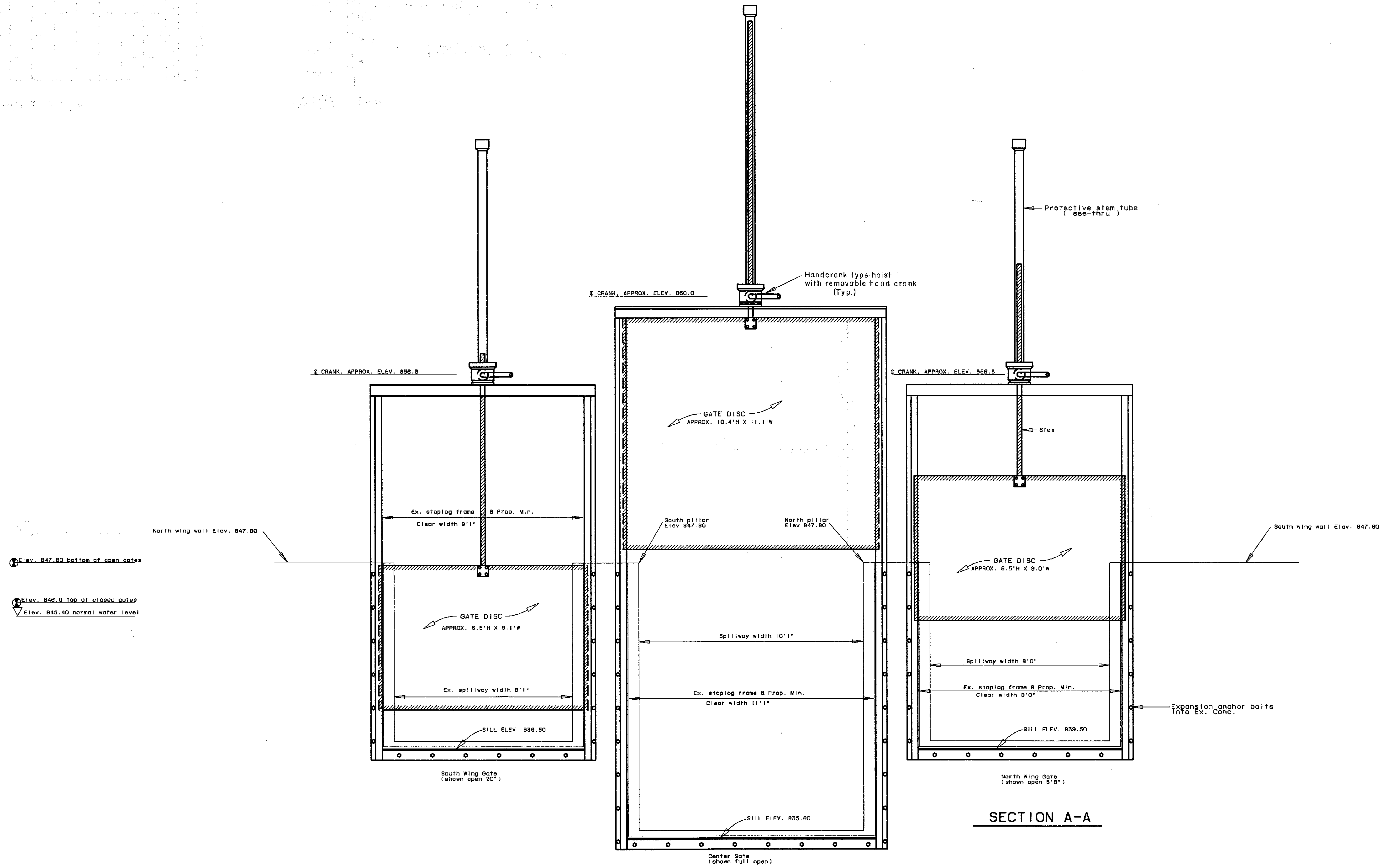
Notes: ① Slide Gate, Frame & Hoist shown for the purpose of illustrating location, required opening dimensions and water levels. Actual details of member sizes, fabrication and installation to be supplied by manufacturer.

② CONTRACTOR shall make field measurements as may be required by slide gate manufacturer to establish exact dimensions necessary for gate fabrication

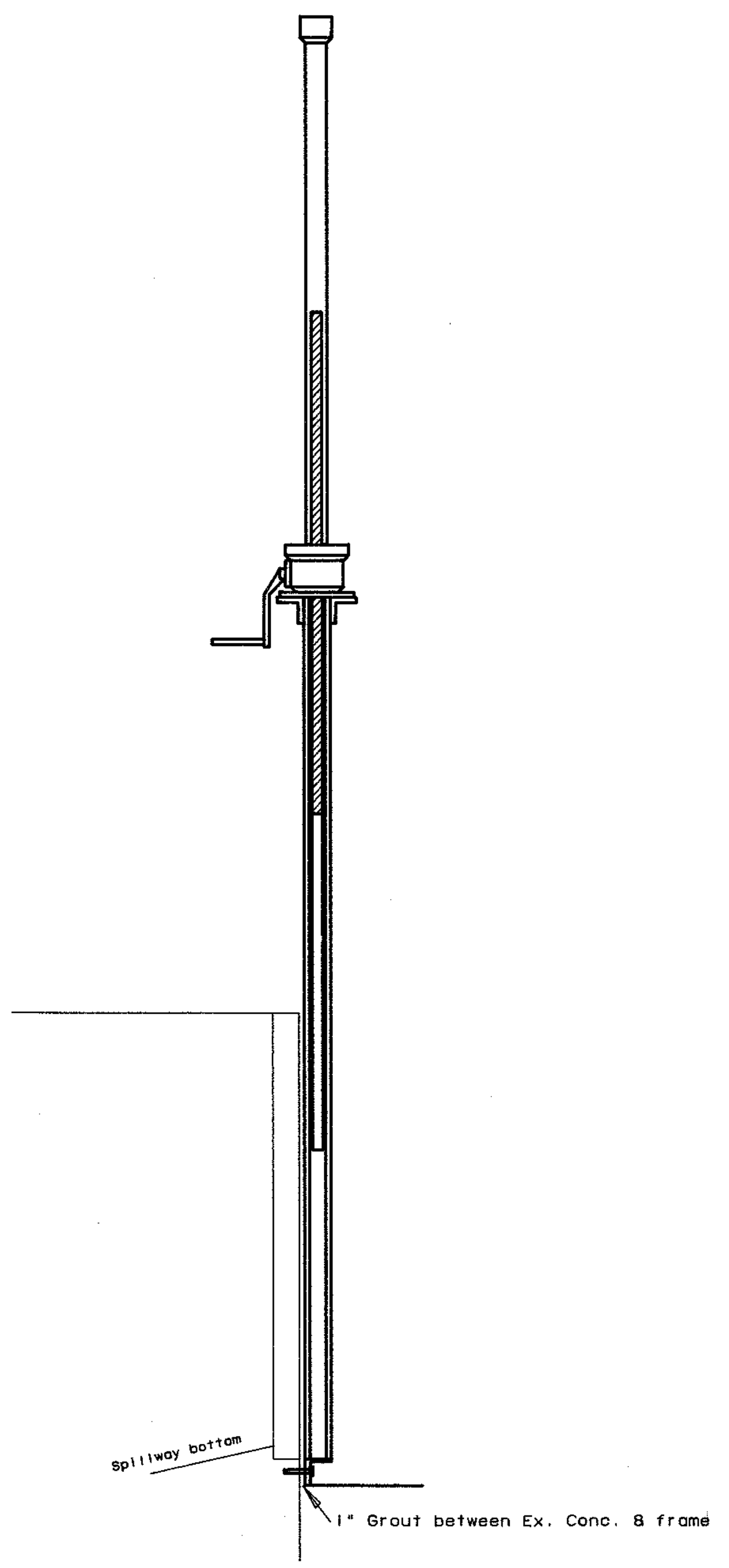
③ Lumber for Stop Logs shall be pressure treated to meet A.W.P.B. standard LP-22 for ground contact (0.40 #/cf Retention)



PLAN VIEW



SECTION A-A



SECTION B-B

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civil engineers/surveyors
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grand rapids, mi. 49506
616-957-4350

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| Revised December, 1989 to reflect DNR review comments | E.J. | 12-89 |
| Add Stop Logs | E.J. | 1-90 |

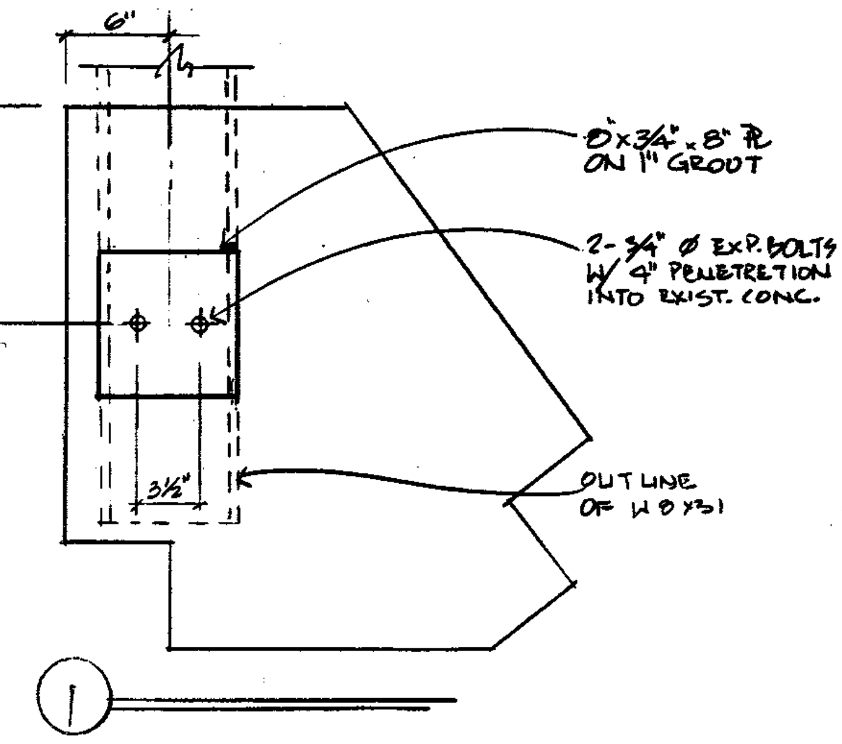
sewer permit number
water permit number
approved by
name date
revised in accordance by
w/construction records date

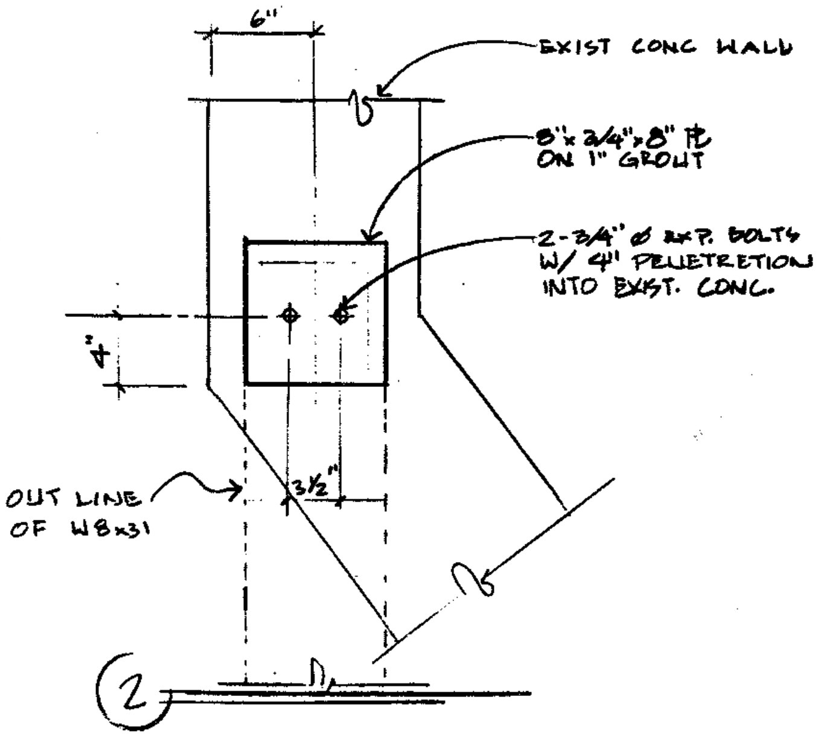
CITY OF WHITE CLOUD
Newaygo County, Michigan

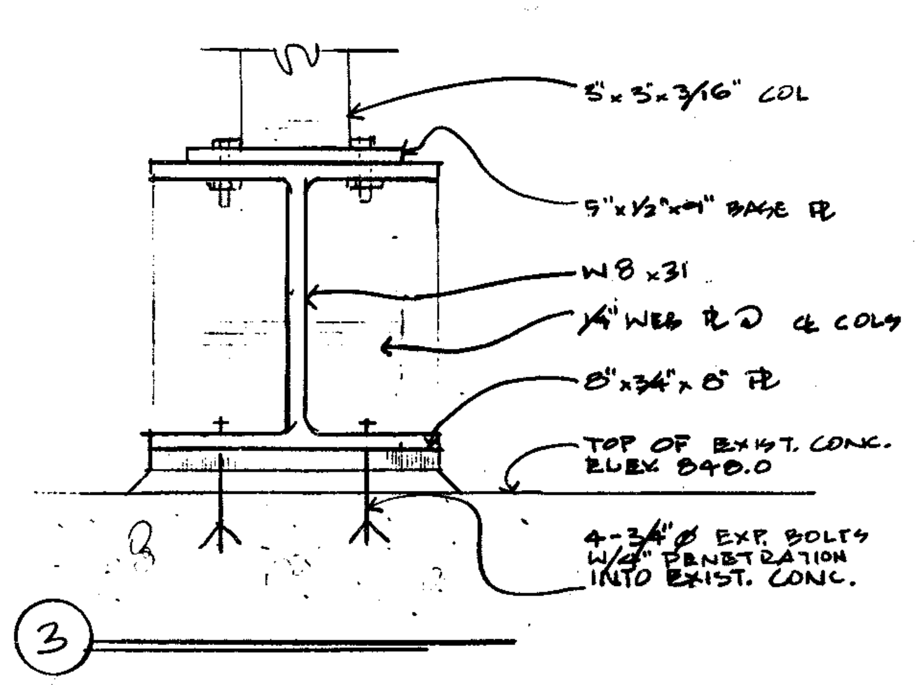
**WHITE CLOUD DAM RECONSTRUCTION**

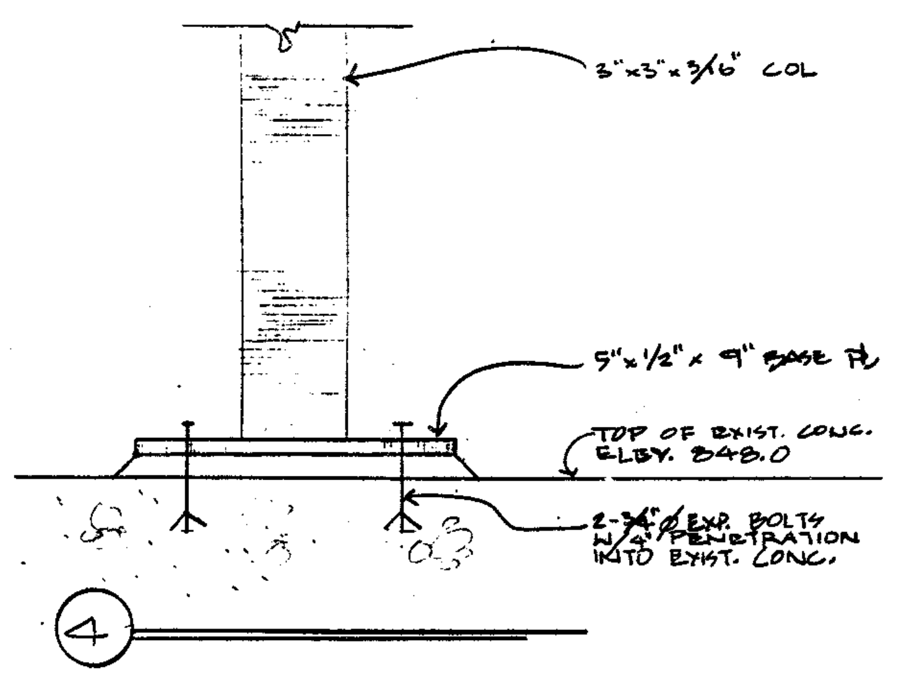
drawn by date
M C S 11/06/89
checked by date
D G M 11/06/89
project number
1355
sheet number
6 of 9
CAD

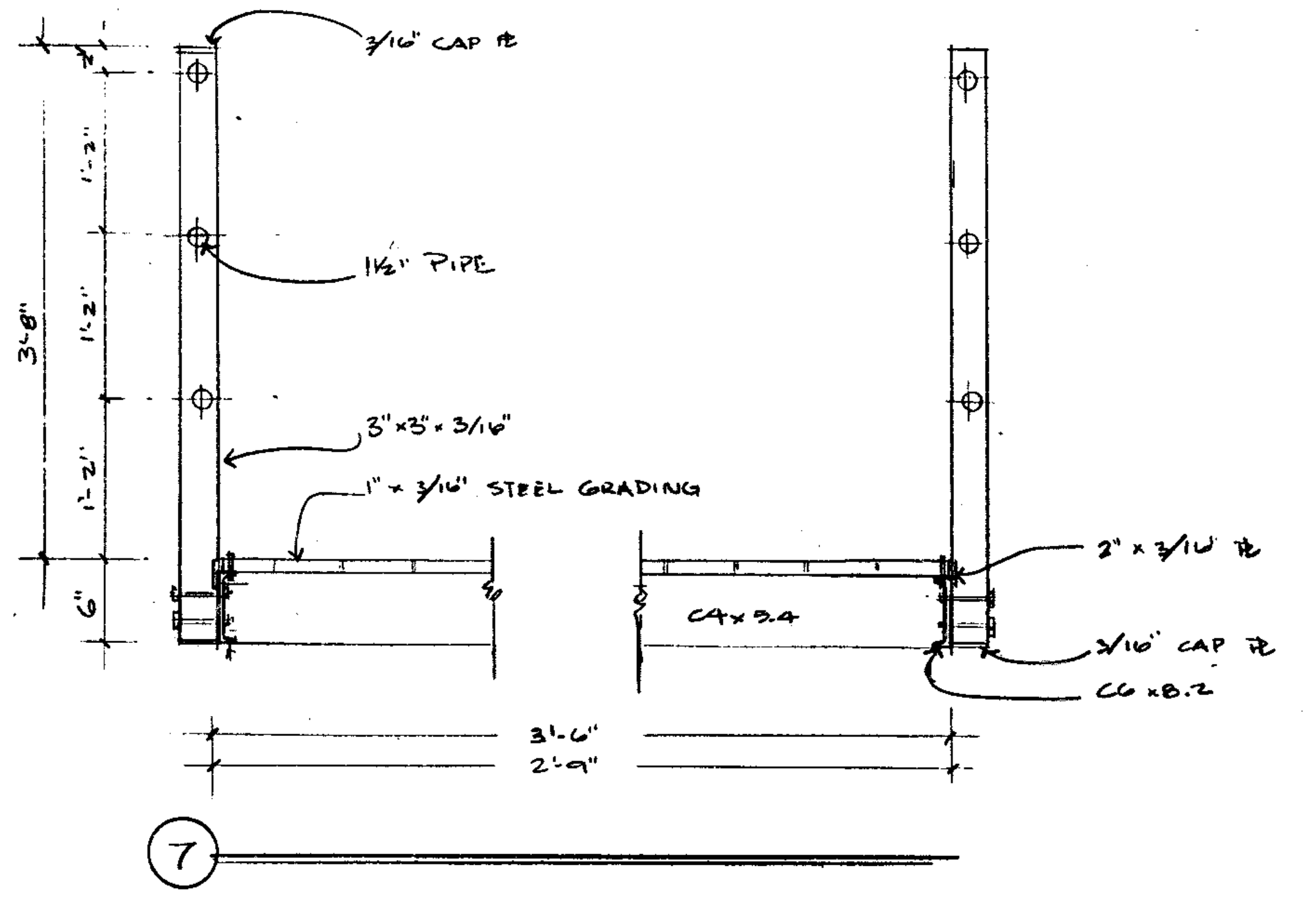
**SLIDE GATES: PRINCIPAL SPILLWAY**

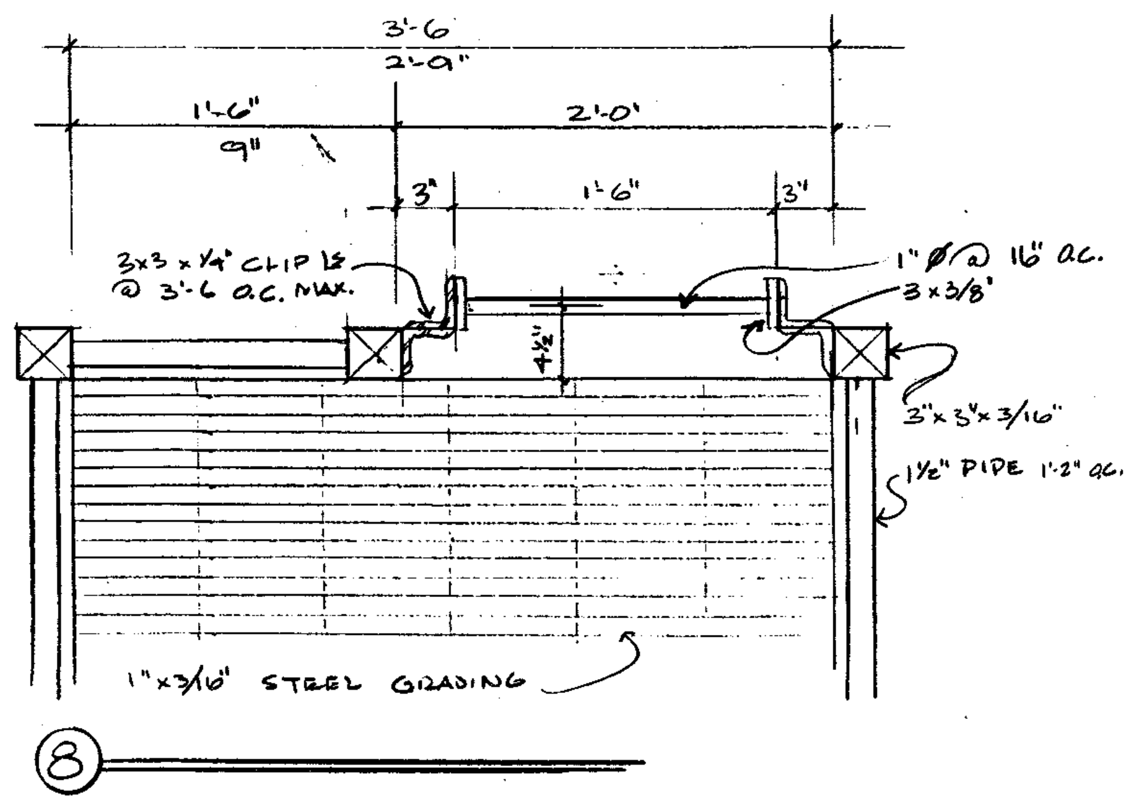


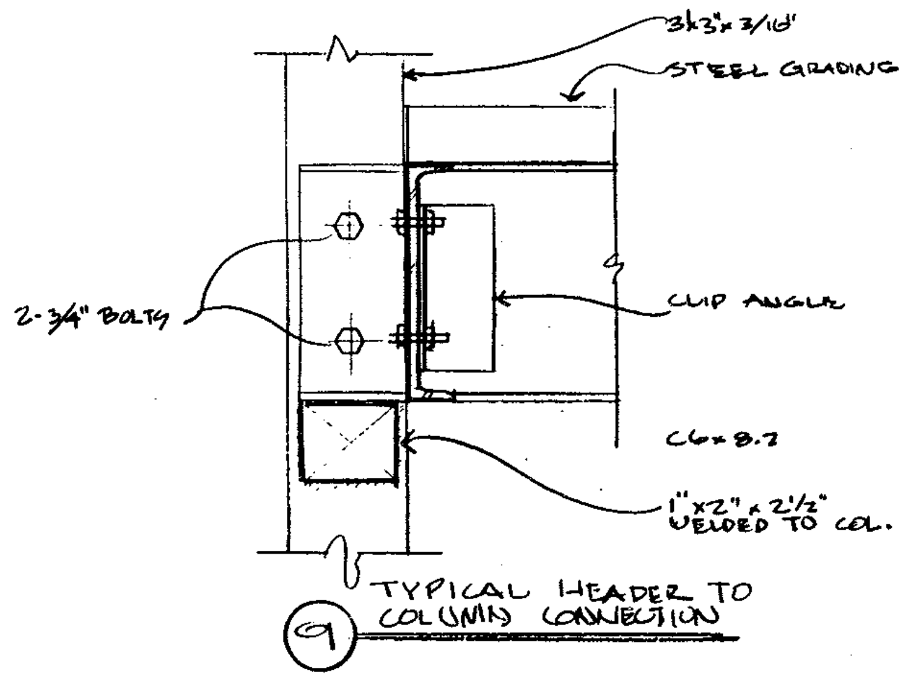




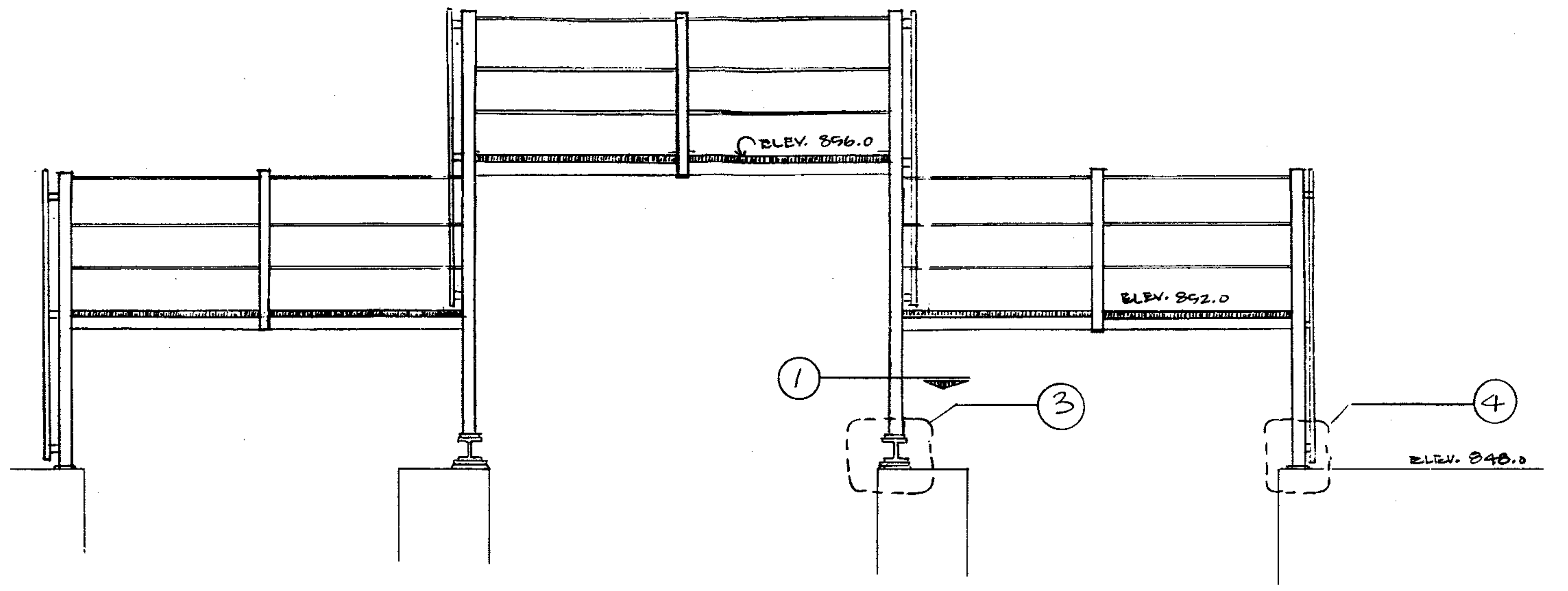




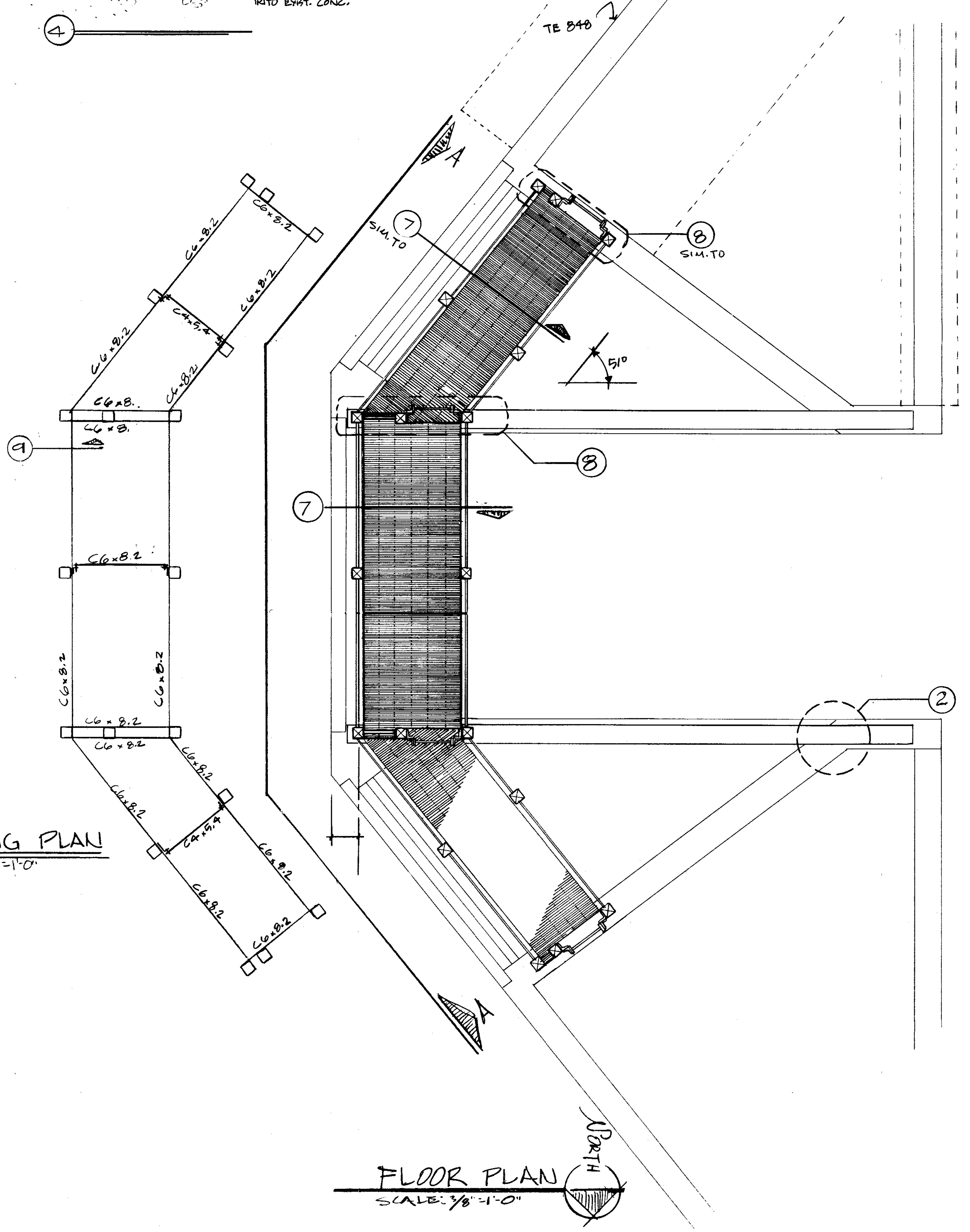




TYPICAL HEADER TO COLUMN CONNECTION



SECTION A



FRAMING PLAN
SCALE: 3/8"=1'-0"

FLOOR PLAN
SCALE: 3/8"=1'-0"

Newhof & Winer
Architects
Engineers
Planners
Grand Rapids, Michigan

olson, meyers & may, inc.
civil engineers
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water permit number
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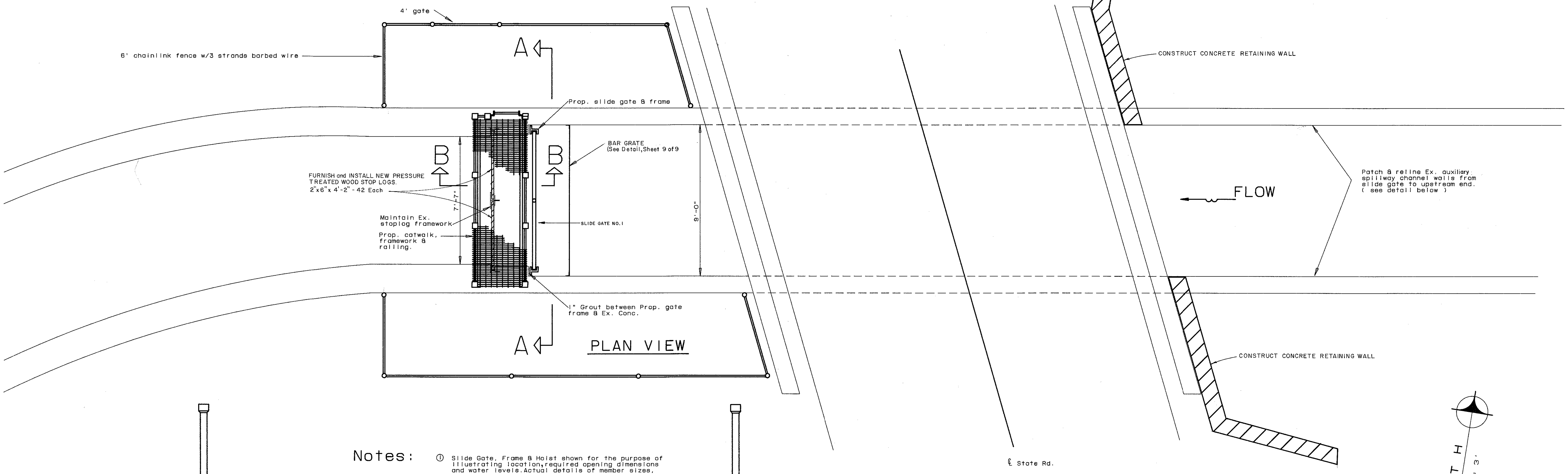
name
date
revised in accordance w/construction records
by
date

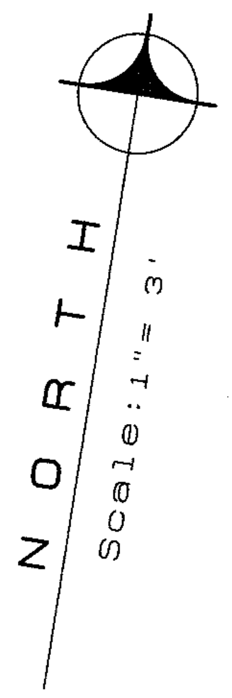
Revised December 1989 to reflect DNR review comments
E.J.
12-89
revisions
by
date

CITY OF WHITE CLOUD
Newaygo County, Michigan
WHITE CLOUD DAM
RECONSTRUCTION

drawn by N.& W. date 10-89
checked by D.G.M. date 10-89
project number 01355
sheet number 7 of 9

CATWALKS: PRINCIPAL SPILLWAY



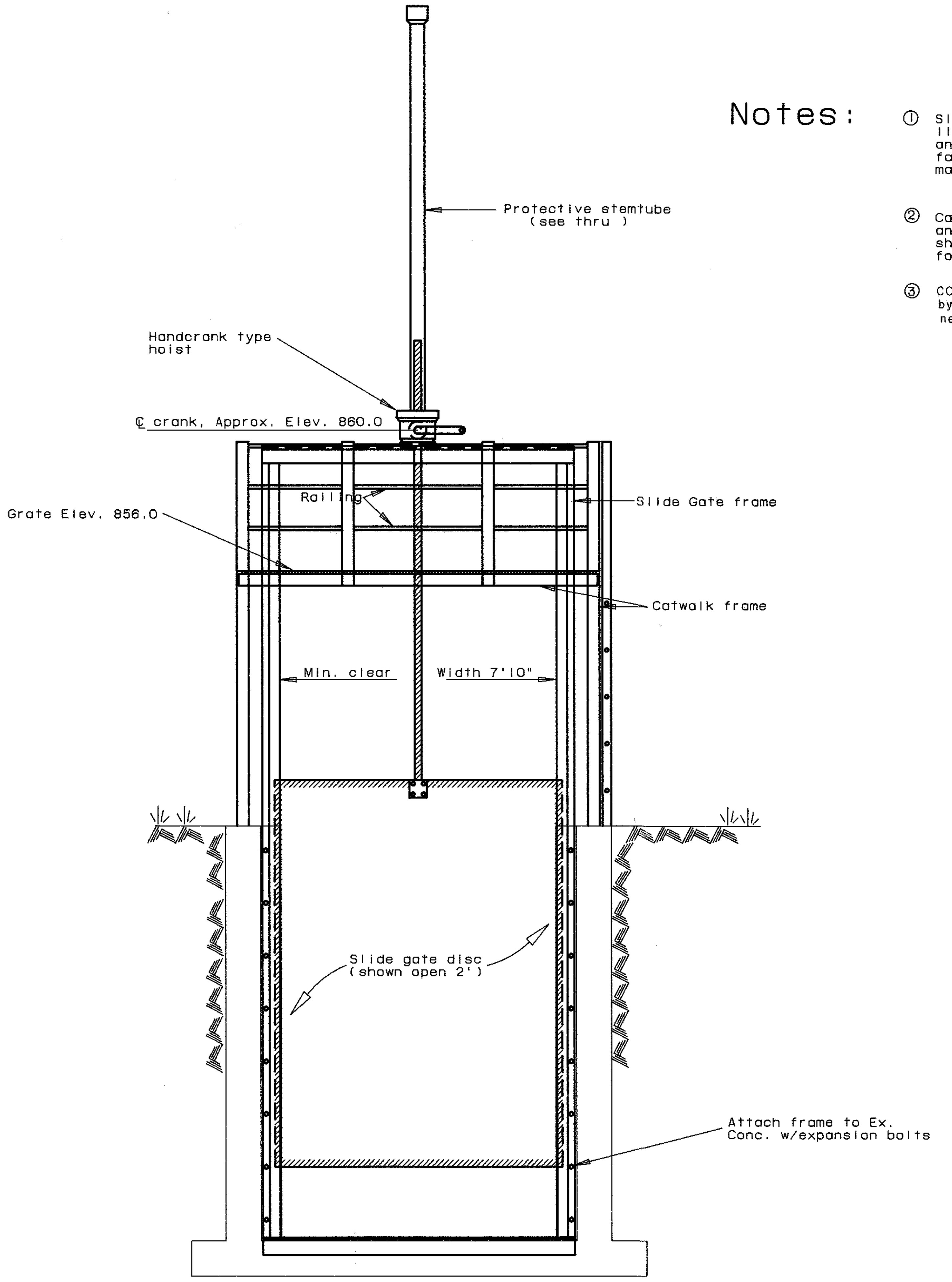


Notes:

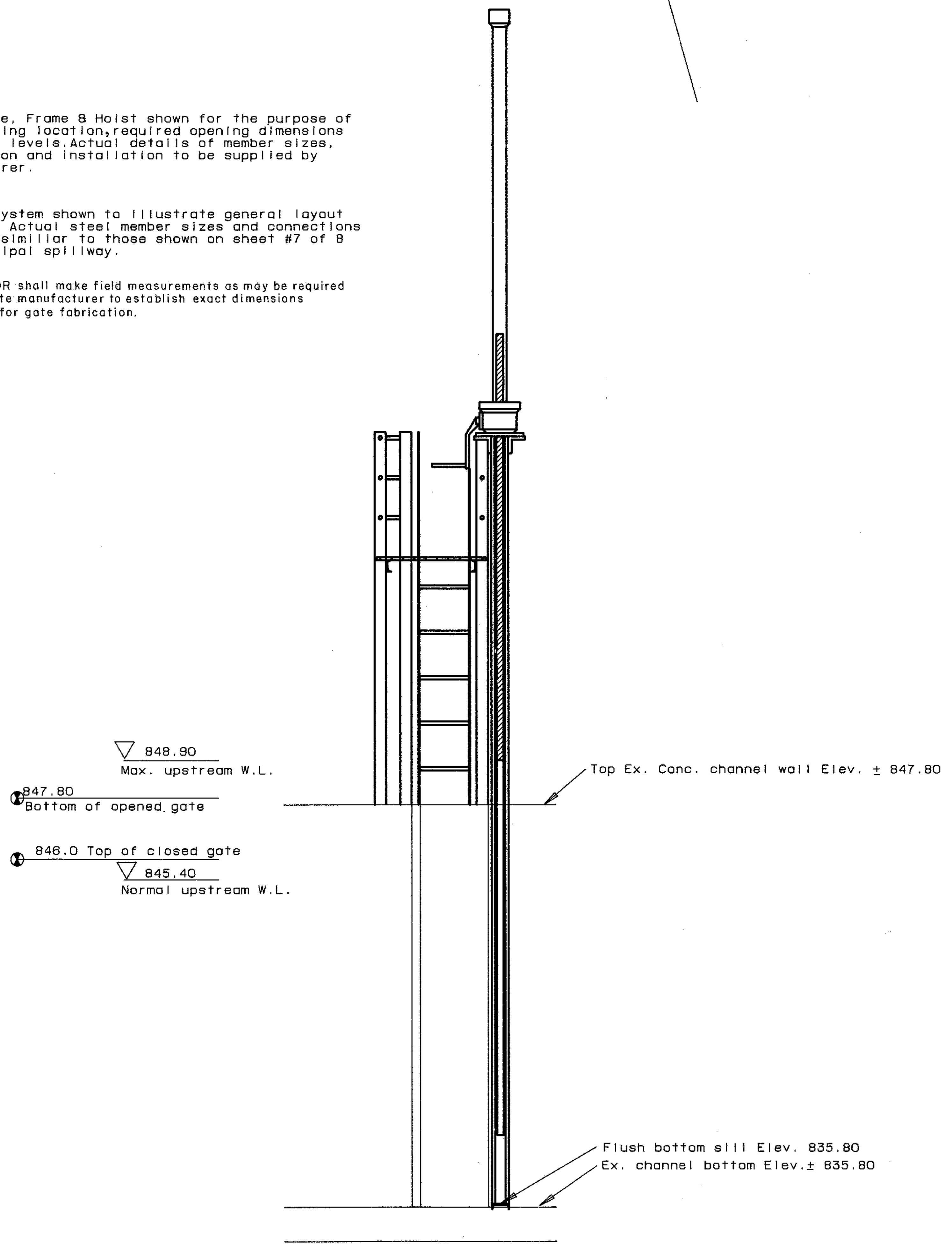
① Slide Gate, Frame & Hoist shown for the purpose of illustrating location,required opening dimensions and water levels.Actual details of member sizes, fabrication and installation to be supplied by manufacturer.

② Catwalk system shown to illustrate general layout and Elev. Actual steel member sizes and connections shall be similiar to those shown on sheet #7 of 8 for principal spillway.

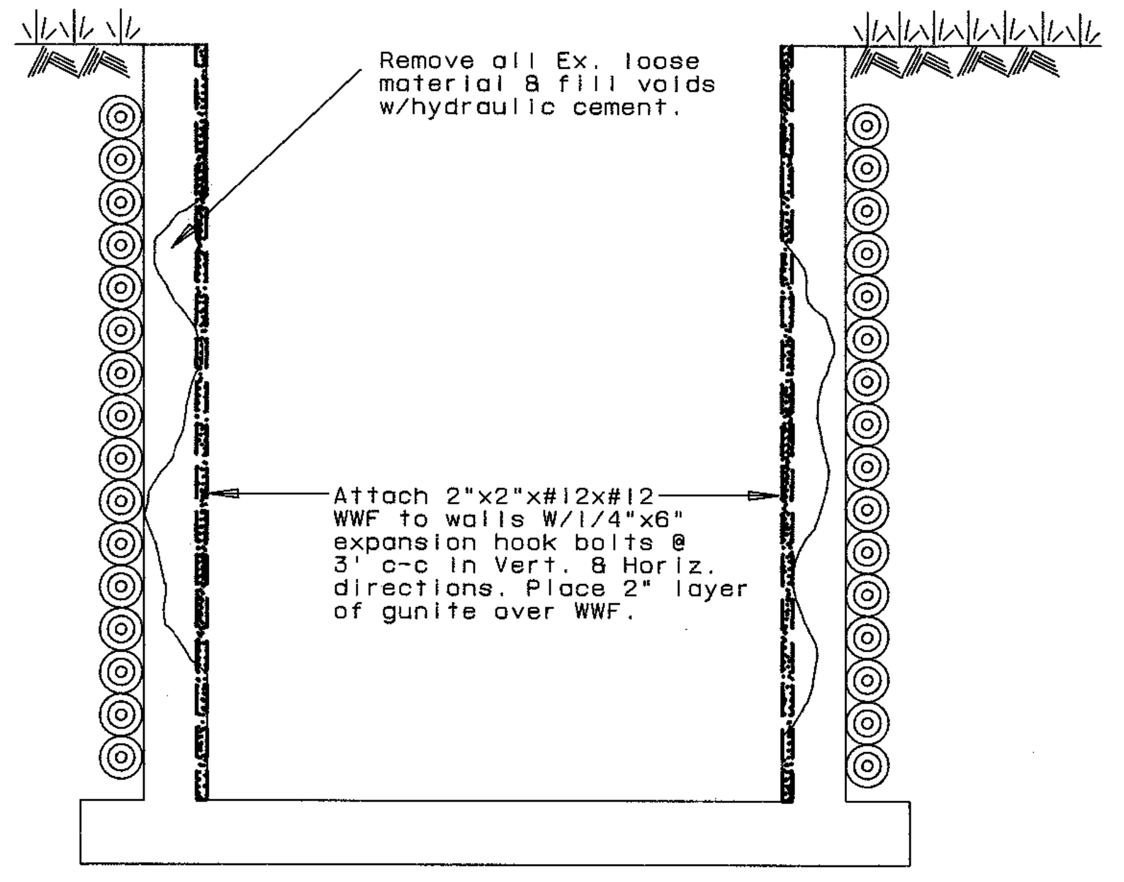
③ CONTRACTOR shall make field measurements as may be required by slide gate manufacturer to establish exact dimensions necessary for gate fabrication.



SECTION A-A



SECTION B-B



AUXILIARY SPILLWAY

Patching & Relining Details

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civil engineers/surveyors
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616-957-4350

| revisions | by | date |
|---|---|---|
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| Add Stop Logs | E.J. | 1-90 |

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water permit number
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name ______ date ______
revised in accordance by ______
w/construction records date ______

CITY OF WHITE CLOUD
Newaygo County, Michigan

**WHITE CLOUD DAM RECONSTRUCTION**

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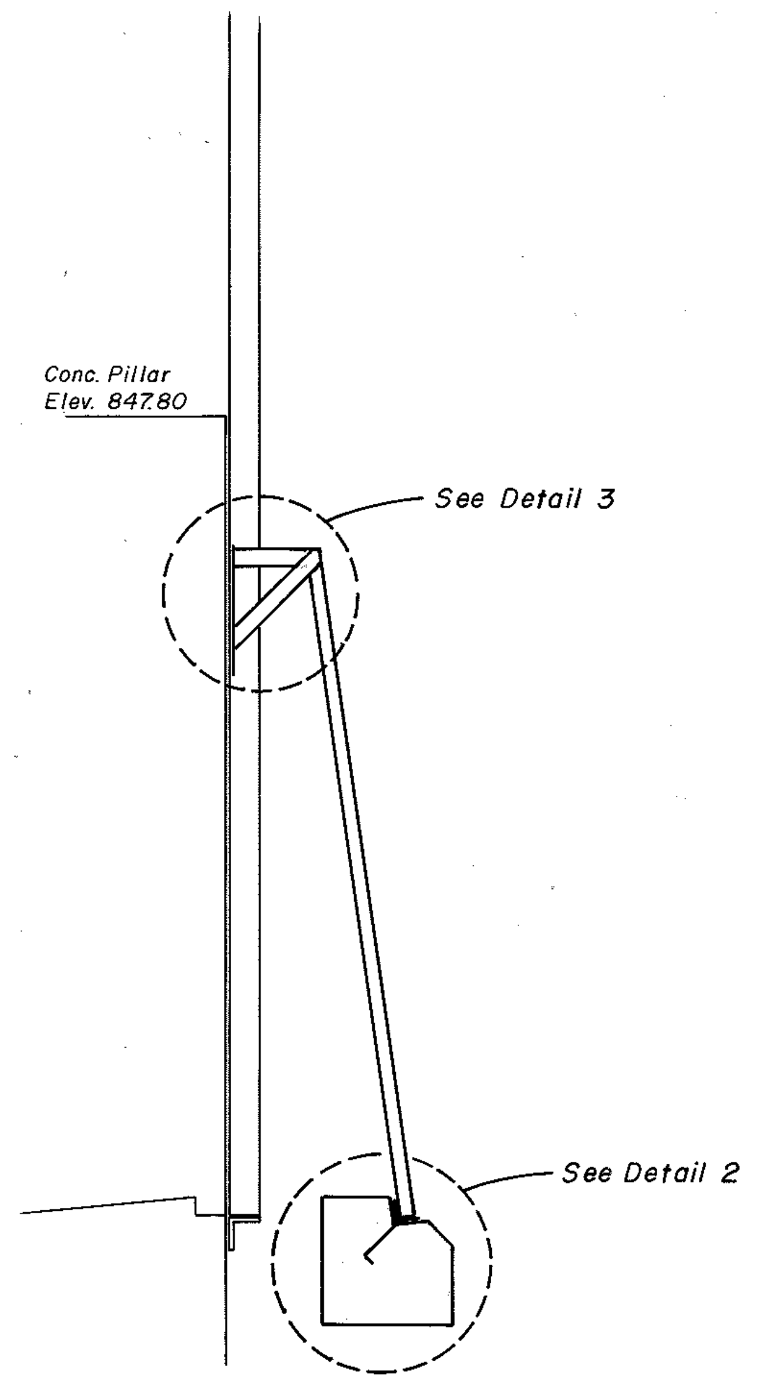
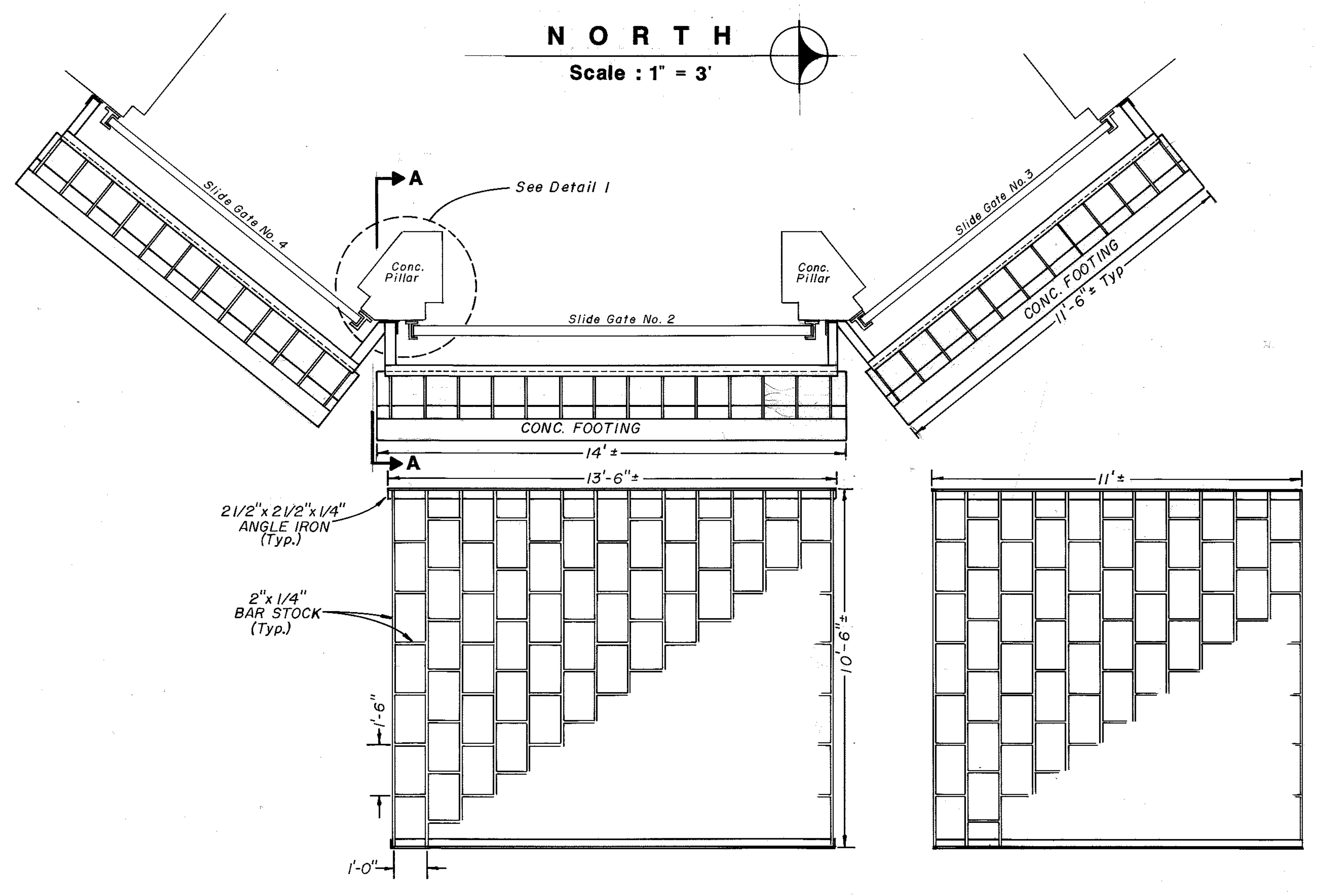
M C S 11/06/89
checked by date
D G M 11/06/89
project number
1355
sheet number
8 of 9
CAD

**SLIDEGATE,CATWALK AND CHANNEL WALL REPAIRS: AUXILIARY SPILLWAY**

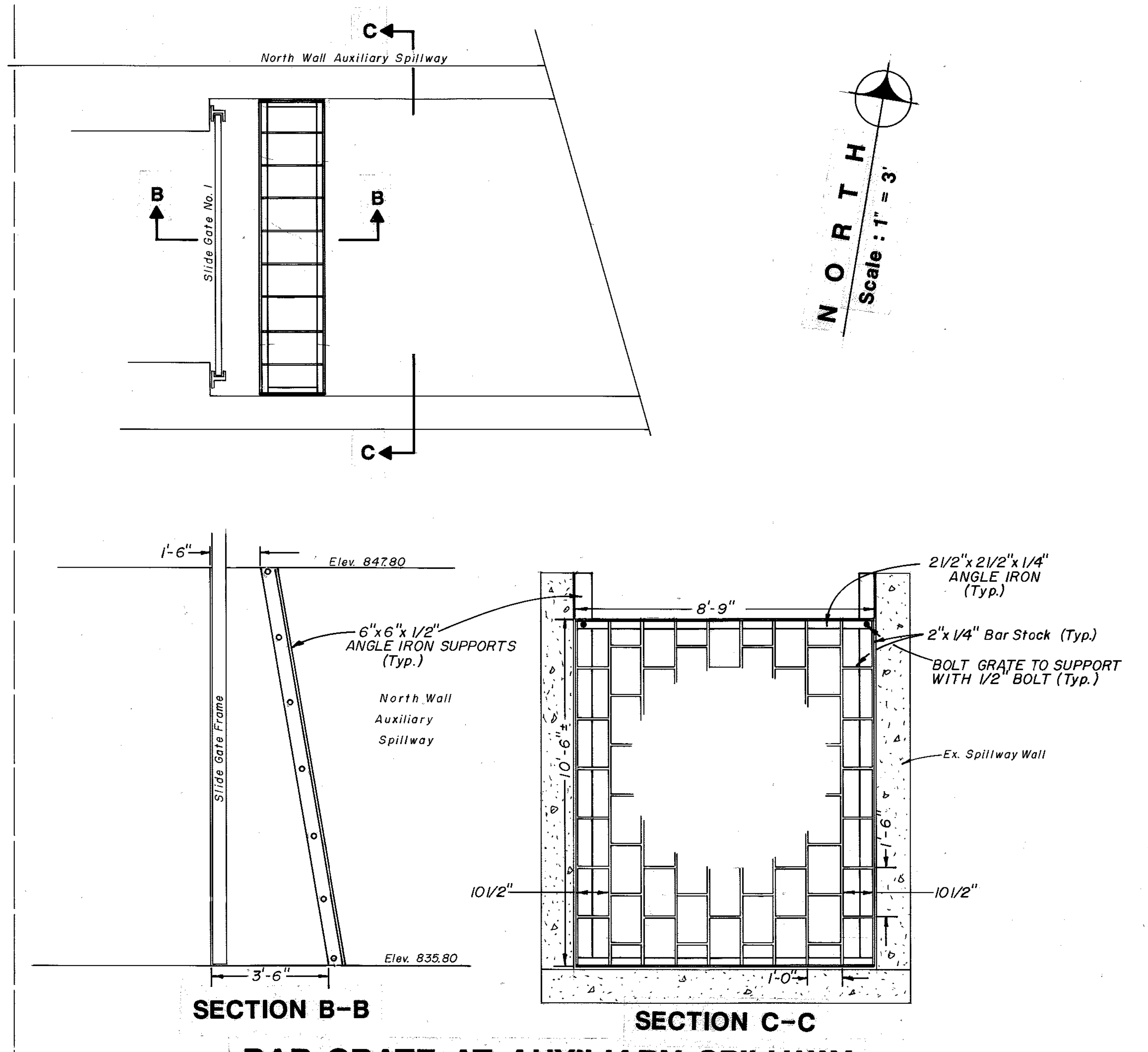


## SECTION A–A

Scale : 1" = 3'



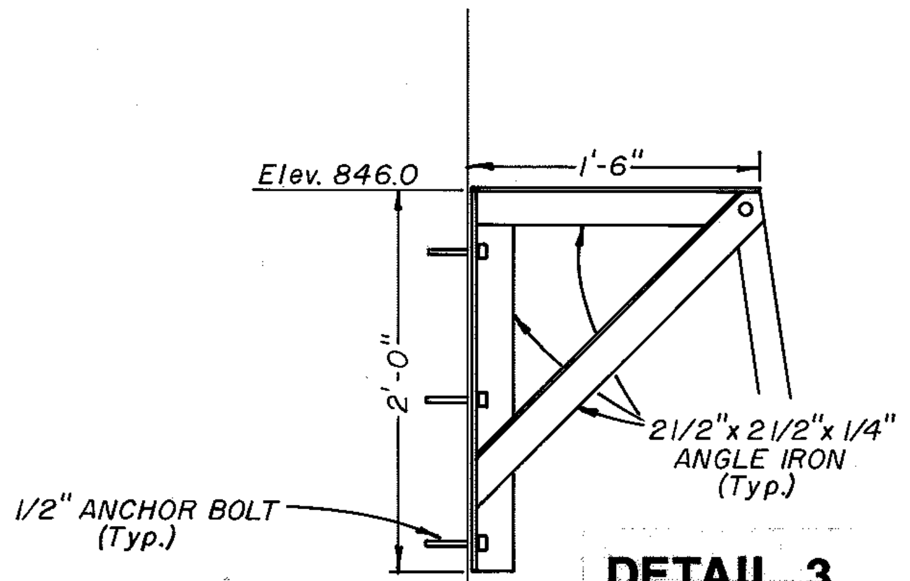
## BAR GRATES AT PRINCIPAL SPILLWAY



SECTION B–B

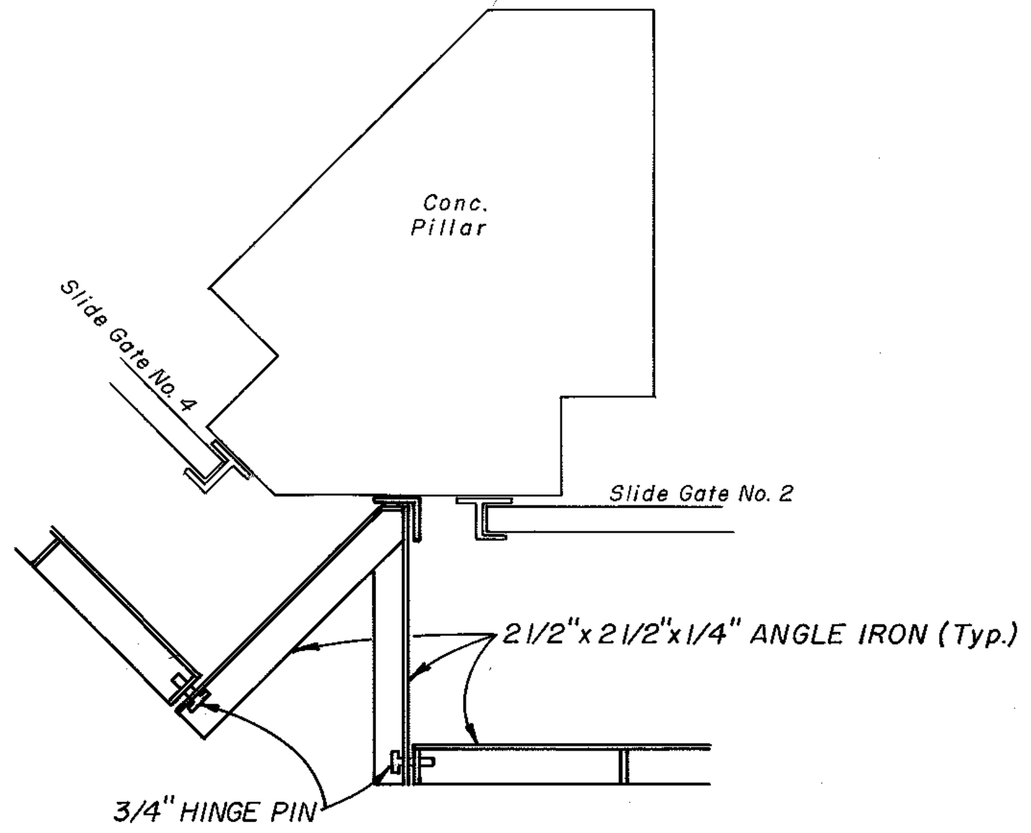
SECTION C–C

## BAR GRATE AT AUXILIARY SPILLWAY



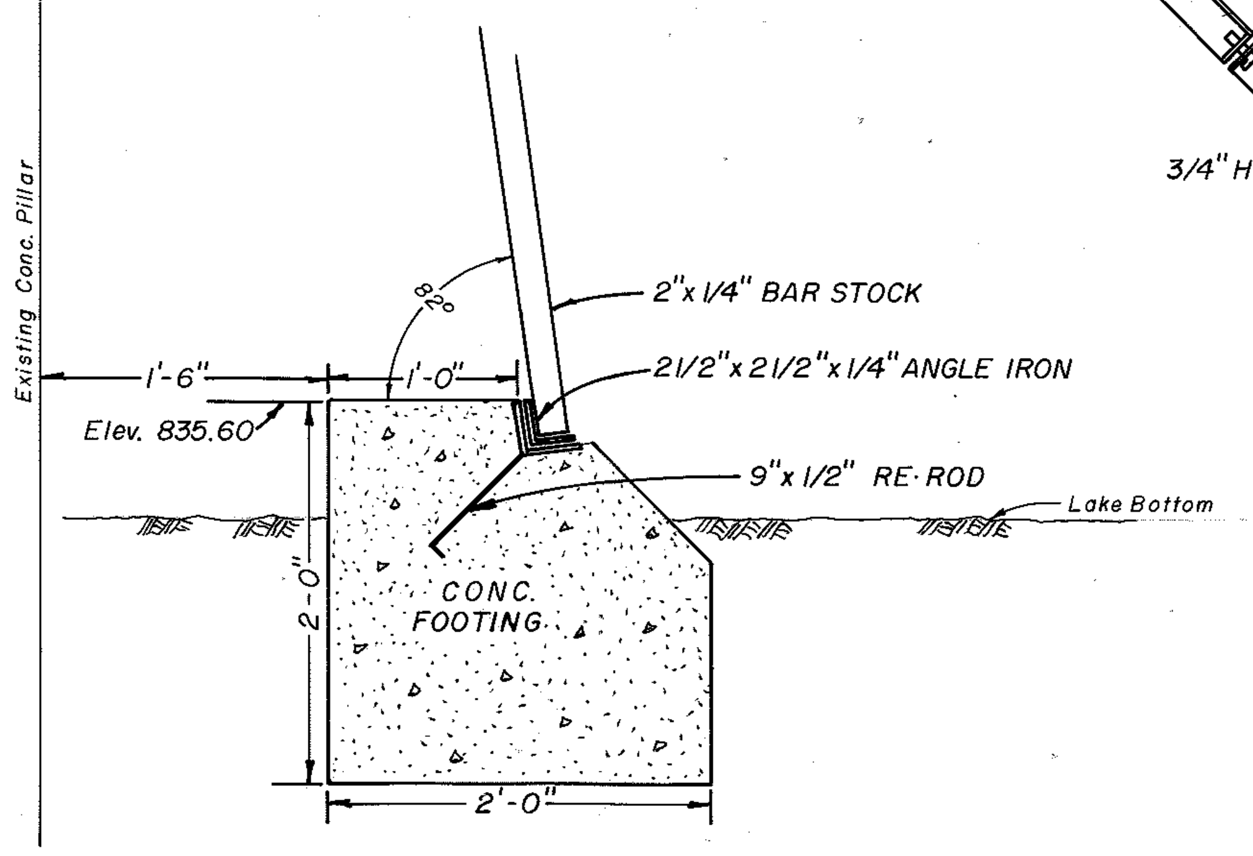
## DETAIL 3

Scale : 1" = 1'



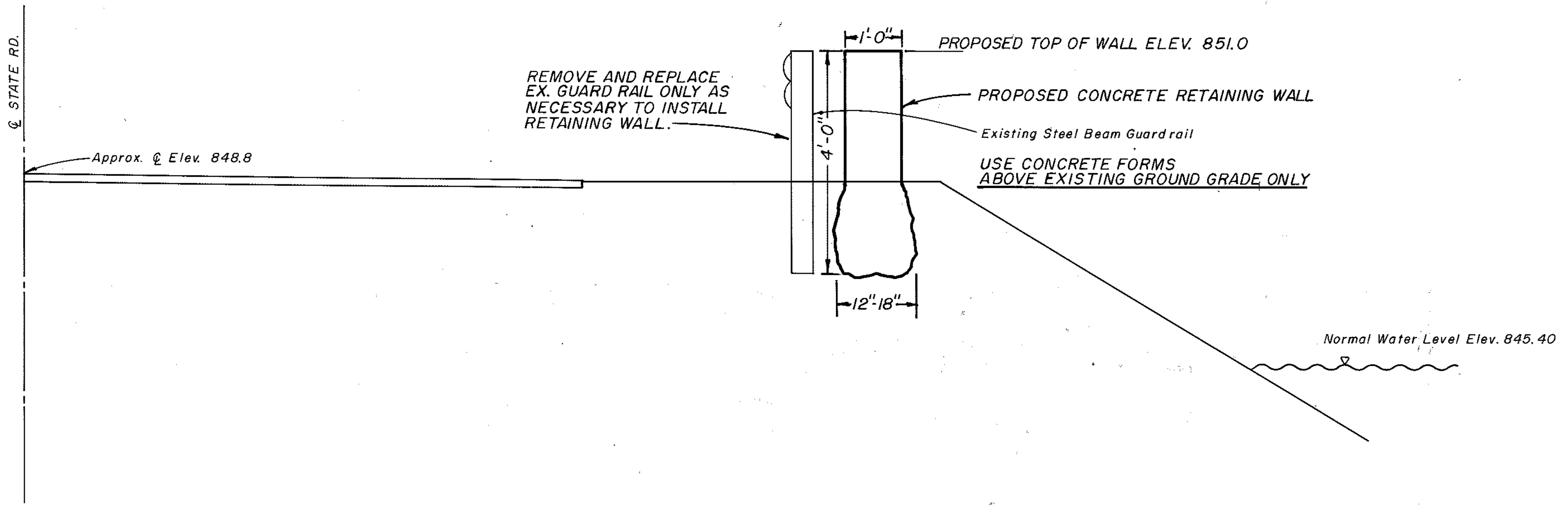
## DETAIL 1

Scale : 1" = 1'



## DETAIL 2

Scale : 1" = 1'



## PROPOSED RETAINING WALL DETAIL

Scale : 1" = 2' Vert. & Horiz.

olson, meyers & may, inc.
civil engineers
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grand rapids, mi. 49506
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sewer permit number
water permit number
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name date
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| revisions | by | date |
|---|---|---|
| Revised December, 1989 to reflect DNR review comments. | E.J. | 12-89 |

CITY OF WHITE CLOUD
Newaygo County, Michigan

WHITE CLOUD DAM
RECONSTRUCTION

| | |
|---|---|
| drawn by<br>E.J. | date<br>12-89 |
| checked by<br>L.A.M. | date<br>12-89 |
| project number<br>01355 | |
| sheet number<br>9 of 9 | |

DETAILS

GEI Consultants

# CALCULATION COVER PAGE

| | |
|---|---|
| **Page** | 1 of 11 |
| **Rev. No.** | 1 |
| **Client** | City of White Cloud |
| **Project** | White Cloud Dam Disposition Feasibility Study |
| **Project No.** | 2302435 |
| **Document No.** | N/A |

**Summary**

This section summarizes the preliminary geotechnical analyses for the earthen embankments. The right earthen embankment adjacent to the principal spillway structure was selected for evaluation as it is the steepest unarmored earthen section with visible seepage at the toe of the embankment. Key components of the geotechnical analysis include estimating material properties and completing embankment seepage and slope stability analysis.

**Record of Revisions**

| Rev. | Description | Code | Pages/Sections | Name | Date |
|---|---|---|---|---|---|
| 0 | Preliminary | P | All | M. Carden | 11/16/2023 |
| | | R | All | J. McDermott | 3/7/2024 |
| | | A | All | D. DeVaun | 3/7/2024 |
| 1 | Final – Feasibility Study | P | All | J. Roell | 1/16/2025 |
| | | R | All | M. Carden | 1/17/2025 |
| | | A | All | M. Carpenter | 1/24/2025 |

*Codes: P = Prepared; C = Checked; A = Approved*

| | | | | | | |
|---|---|---|---|---|---|---|
| GEI Consultants | **Client** | City of Whitecloud | | | **Page** | 2 of 11 |
| | **Project** | Whitecloud Dam Feasibility Study | | | **Pg. Rev.** | 1 |
| | **By** | J. Roell | **Chk.** | M. Carden | **App.** | M. Carpenter |
| | **Date** | 1/16/2025 | **Date** | 1/17/2025 | **Date** | 1/24/2025 |

| | | | |
|---|---|---|---|
| **Project No.** | 2302435 | **Document No.** | N/A |
| **Subject** | Geotechnical Analysis Criteria and Summary | | |

# *Analysis Criteria*

This section summarizes the geotechnical analyses for the earthen embankments. The right earthen embankment adjacent to the principal spillway structure was selected for evaluation as it is the steepest unarmored earthen section with visible seepage at the toe of the embankment. Key components of the geotechnical analysis include estimating material properties and completing embankment seepage and slope stability analysis.

### *Representative Cross Section*

As discussed above, one representative section was chosen through the right embankment with the steepest downstream embankment slope and the observed seepage breakout at the toe. The existing geometry has a crest El. of approximately 849 feet with upstream and downstream slopes of 2.5H:1V and 2H:1V, respectively. There is no documentation of the construction of the embankment via record drawings, therefore the section is assumed to be homogeneous without the presence of a core, cutoff wall or drains. This is a conservative assumption for the purposes of this analysis.

### *Seepage Calibration*

A SEEP/W model was run on the representative section and calibrated to the existing conditions based on the normal pool headwater elevation and the seepage breakout at the toe of the right earthen embankment. The observed seepage breakout elevation was determined to be approximately at El. 840 feet. Calibration of the model included adjusting the conductivity ratios (Ky/Kx) and adjusting hydraulic conductivities of the defined embankment to most closely match seepage breakout at El. 840 feet that was observed on-site assuming a normal pool headwater elevation. The calibrated seepage model for the existing condition is attached.

### *Seismic Considerations*

The Design Basis Earthquake (DBE) Peak Ground Acceleration (PGA) is based on the 2014 USGS Hazard Maps (Ref. USGS, 2014) for a probabilistic earthquake event having a 2% probability in 50 years (2,500-year return period). The 2014 USGS seismic hazard map for Michigan shows a PGA of about 0.03g in bedrock, with a 2% probability of exceedance in 50 years (approximately equal to 2,500-year return period), with modest amplification to the dam crest, this design earthquake acceleration would be about 1/2 of the 0.1g FERC-specified threshold for considering earthquake impacts. Therefore, based on commonly accepted standards of practice as defined by the FERC Engineering Guidelines, it is not considered necessary to perform a site-specific seismic hazard analysis; therefore, the design did not account for seismic loading.

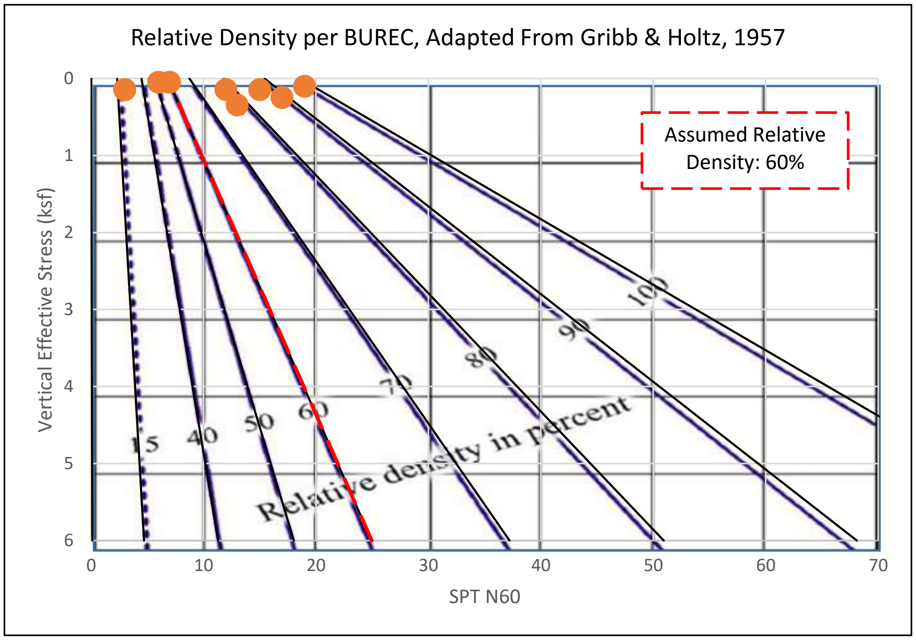
| | | | | | | |
|---|---|---|---|---|---|---|
| GEI Consultants | **Client** | City of Whitecloud | | | **Page** | 3 of 11 |
| | **Project** | Whitecloud Dam Feasibility Study | | | **Pg. Rev.** | 1 |
| | **By** | J. Roell | **Chk.** | M. Carden | **App.** | M. Carpenter |
| | **Date** | 1/16/2025 | **Date** | 1/17/2025 | **Date** | 1/24/2025 |
| **Project No.** | 2302435 | **Document No.** | N/A | | | |
| **Subject** | Geotechnical Analysis Criteria and Summary | | | | | |

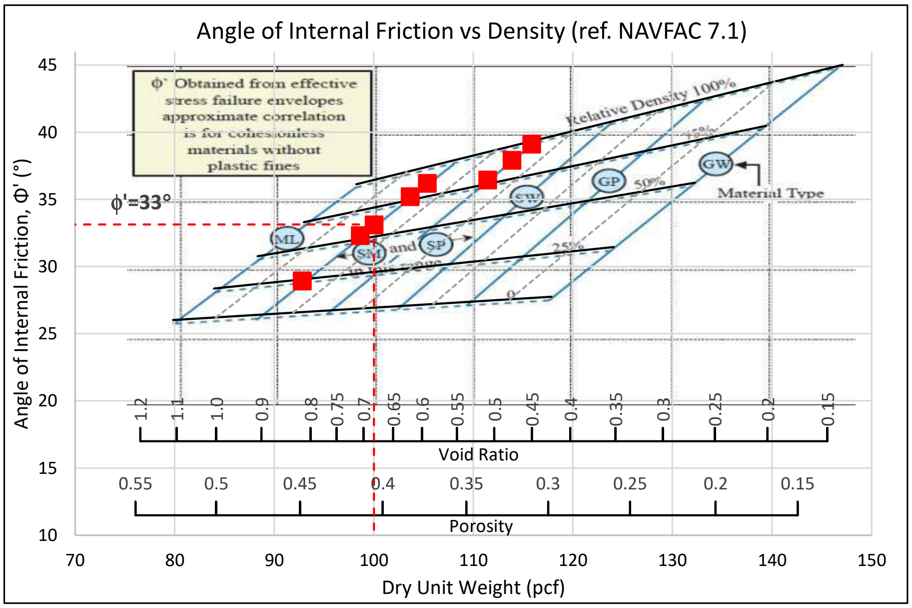
# MATERIAL PARAMETER DEVELOPMENT

| | | | | | | |
|---|---|---|---|---|---|---|
| GEI Consultants | **Client** | City of Whitecloud | | | **Page** | 4 of 11 |
| | **Project** | Whitecloud Dam Feasibility Study | | | **Pg. Rev.** | 1 |
| | **By** | J. Roell | **Chk.** | M. Carden | **App.** | M. Carpenter |
| | **Date** | 1/16/2025 | **Date** | 1/17/2025 | **Date** | 1/24/2025 |
| **Project No.** | 2302435 | **Document No.** | N/A | | | |
| **Subject** | Geotechnical Analysis Criteria and Summary | | | | | |

## Material Property Interpretation

***SPT Correlation Results: Embankment Fill (SM)***


Relative Density per BUREC, Adapted From Gribb & Holtz, 1957


Angle of Internal Friction vs Density (ref. NAVFAC 7.1)

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| | | | | | |
|---|---|---|---|---|---|
| **Client** | City of Whitecloud | | | **Page** | 5 of 11 |
| **Project** | Whitecloud Dam Feasibility Study | | | **Pg. Rev.** | 1 |
| **By** | J. Roell | **Chk.** | M. Carden | **App.** | M. Carpenter |
| **Date** | 1/16/2025 | **Date** | 1/17/2025 | **Date** | 1/24/2025 |
| **Project No.** | 2302435 | **Document No.** | N/A | | |
| **Subject** | Geotechnical Analysis Criteria and Summary | | | | |

## Interpreted Soil Layer Properties

| |
|---|
| Input |
| Output |

| | | | | | |
|---|---|---|---|---|---|
| Rel. Density: | 60% | Phi: | 33.0 | deg. |
| Material: | SM | Dry Unit Wt: | 100 | pcf |
| | | Void Ratio (e): | 0.68 | |
| | | Porosity (n): | 0.41 | |

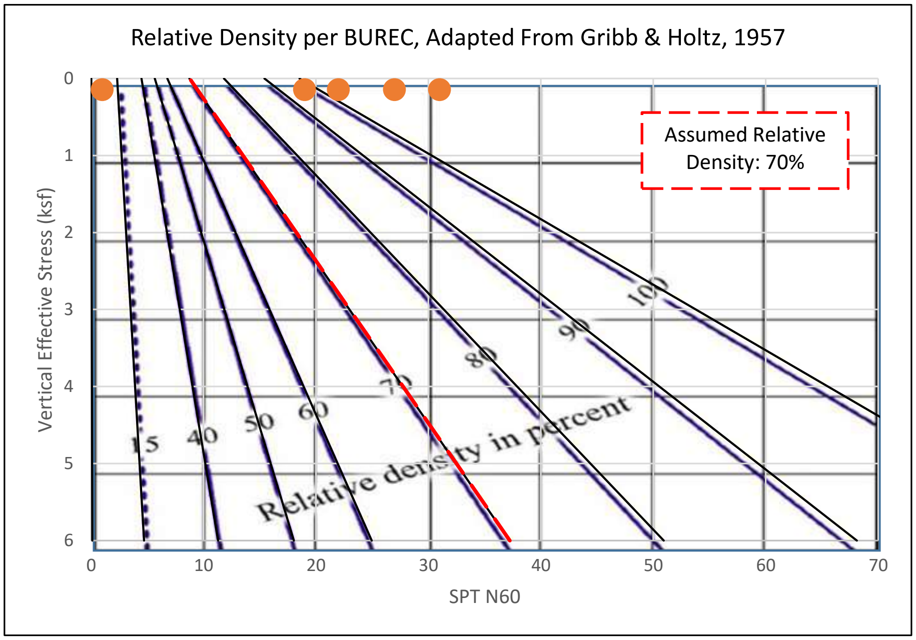
### Seepage & Stability Initial Parameters

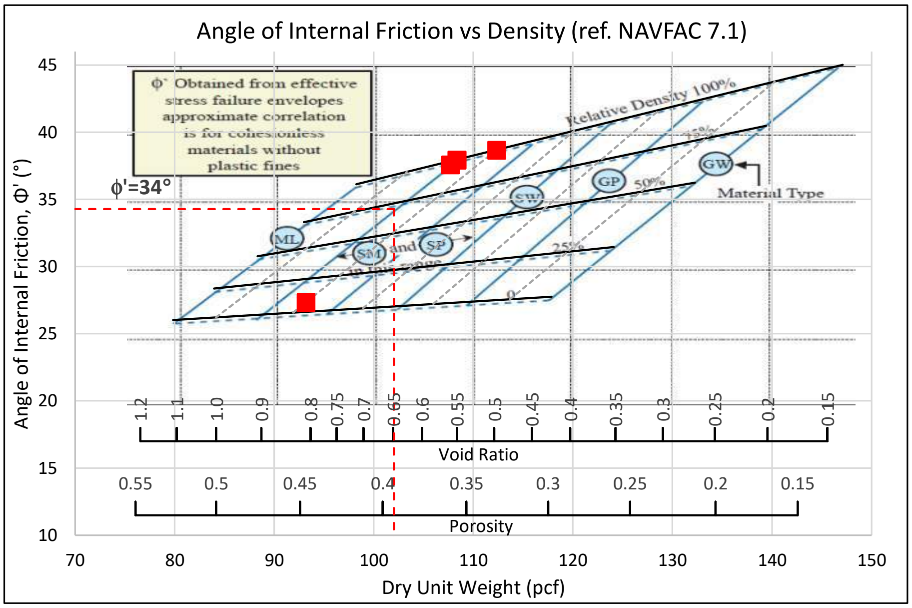
| | | |
|---|---|---|
| Residual Water Content: | 17% | = w |
| Saturated Water Content | 25.4% | = e/SG |
| Specific Gravity: | 2.68 | = SG |
| Saturated Unit Wt. | 125.4 | $= \gamma_d * (1+e/SG)$ |
| Residual (moist) Unit wt: | 116.5 | $= \gamma_d * (1+w)$ |
| Effective Phi: | 33 | deg. |

| | | | | | | |
|---|---|---|---|---|---|---|
| GEI Consultants | **Client** | City of Whitecloud | | | **Page** | 6 of 11 |
| | **Project** | Whitecloud Dam Feasibility Study | | | **Pg. Rev.** | 1 |
| | **By** | J. Roell | **Chk.** | M. Carden | **App.** | M. Carpenter |
| | **Date** | 1/16/2025 | **Date** | 1/17/2025 | **Date** | 1/24/2025 |
| **Project No.** | 2302435 | **Document No.** | N/A | | | |
| **Subject** | Geotechnical Analysis Criteria and Summary | | | | | |

## Material Property Interpretation

***SPT Correlation Results: Foundation (SM)***





| | | | | | | |
|---|---|---|---|---|---|---|
| GEI Consultants | **Client** | City of Whitecloud | | | **Page** | 7 of 11 |
| | **Project** | Whitecloud Dam Feasibility Study | | | **Pg. Rev.** | 1 |
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| | **Date** | 1/16/2025 | **Date** | 1/17/2025 | **Date** | 1/24/2025 |
| **Project No.** | 2302435 | **Document No.** | N/A | | | |
| **Subject** | Geotechnical Analysis Criteria and Summary | | | | | |

## Interpreted Soil Layer Properties

| |
|---|
| Input |
| Output |

| | | | | | |
|---|---|---|---|---|---|
| Rel. Density: | 70% | Phi: | 34.0 | deg. |
| Material: | SM | Dry Unit Wt: | 102 | pcf |
| | | Void Ratio (e): | 0.65 | |
| | | Porosity (n): | 0.39 | |

### Seepage & Stability Initial Parameters

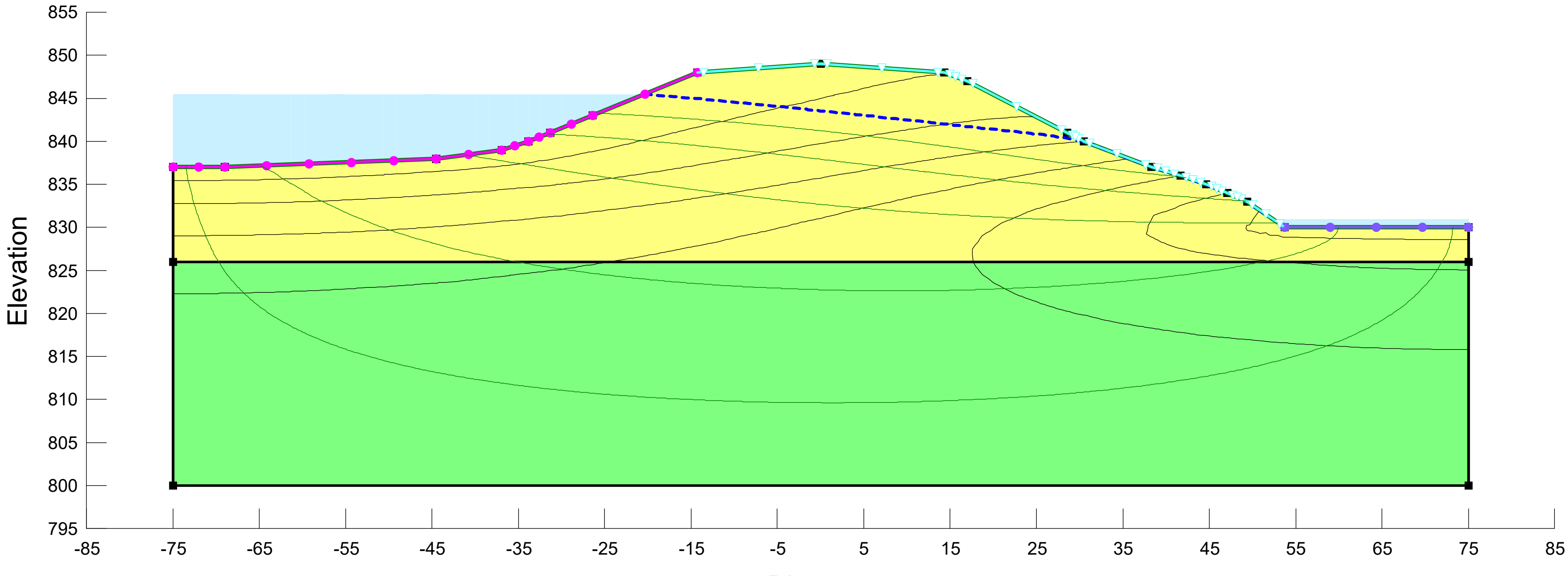
| | | |
|---|---|---|
| Residual Water Content: | 10% | = w |
| Saturated Water Content | 24.3% | = e/SG |
| Specific Gravity: | 2.68 | = SG |
| Saturated Unit Wt. | 126.7 | $= \gamma_d * (1+e/SG)$ |
| Residual (moist) Unit wt: | 112.2 | $= \gamma_d * (1+w)$ |
| Effective Phi: | 34 | deg. |

# RIGHT EMBANKMENT SECTION
# REV. 1

By: J. Roell
Rev: M. Carden
App: M. Carpenter

## ANALYSIS: Steady-State Seepage NP

| Color | Name | Hydraulic Material Model | Vol. WC. Function | K-Function | Ky'/Kx' Ratio |
|---|---|---|---|---|---|
| 🟨 | Embankment Fill | Saturated / Unsaturated | Embankment | Embankment | 0.02 |
| 🟩 | Foundation | Saturated / Unsaturated | Embankment | Embankment | 0.02 |



| Color | Name | Category | Kind | Parameters |
|---|---|---|---|---|
| ■ | Normal Pool | Hydraulic | Water Total Head | 845.5 ft |
| 🟦 | Open Seepage Face | Hydraulic | Water Flux | 0 ft/sec |
| 🟪 | Tailwater | Hydraulic | Water Total Head | 831 ft |

White Cloud Dam
Embankment Stability

The City of
White Cloud

GEI Consultants

Project: 2302435

FILL EMBANKMENT

01/16/2025

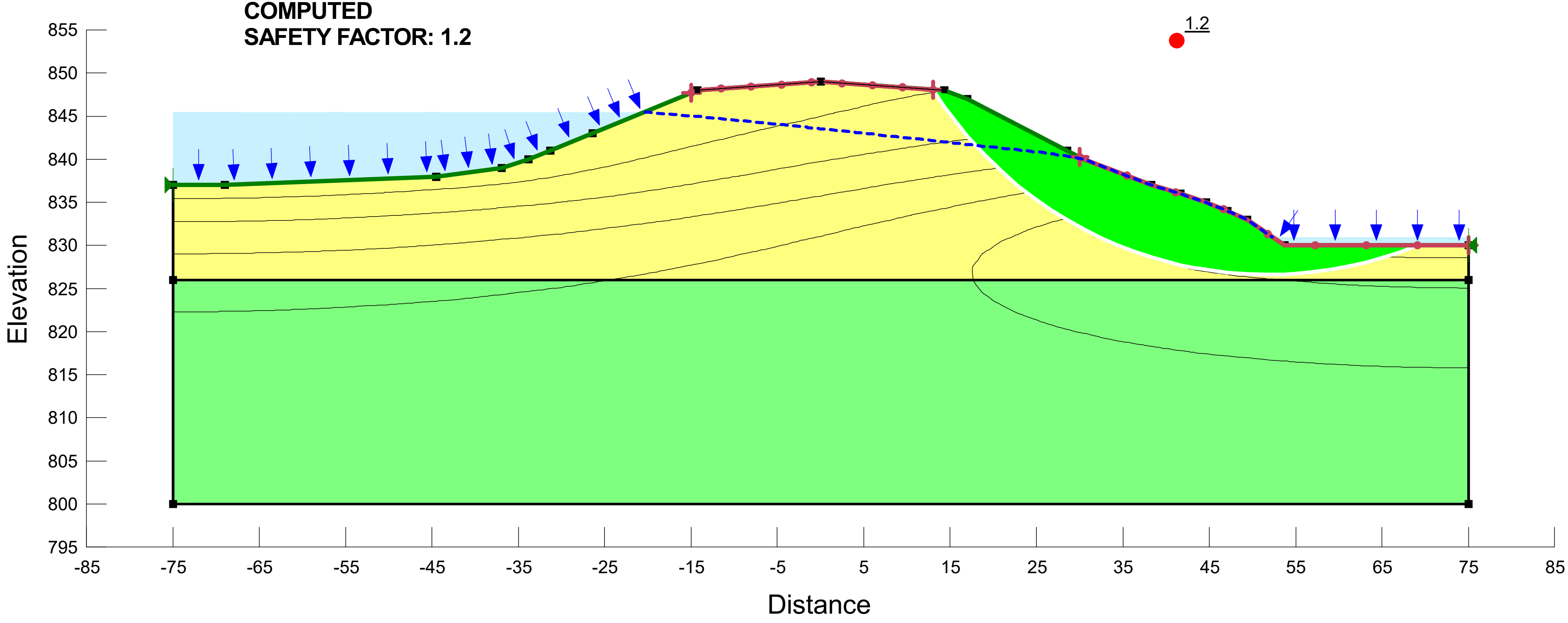
B:\Working\CITY OF WHITE CLOUD\2302435 White Cloud Dam Disposition Feasibility Study\ENG\Evaualtions\Geotech\Rev 1\Cross_Section_1.gsz

# RIGHT EMBANKMENT SECTION
# REV. 1

By: J. Roell
Rev: M. Carden
App: M. Carpenter

## ANALYSIS: Slope Stability NP

| Color | Name | Slope Stability Material Model | Unit Weight (pcf) | Effective Cohesion (psf) | Effective Friction Angle (°) |
|---|---|---|---|---|---|
| 🟨 | Embankment Fill | Mohr-Coulomb | 125 | 0 | 33 |
| 🟩 | Foundation | Mohr-Coulomb | 127 | 0 | 34 |


**COMPUTED**
**SAFETY FACTOR: 1.2**

White Cloud Dam
Embankment Stability

The City of
White Cloud

GEI Consultants

Project: 2302435

FILL EMBANKMENT

01/16/2025

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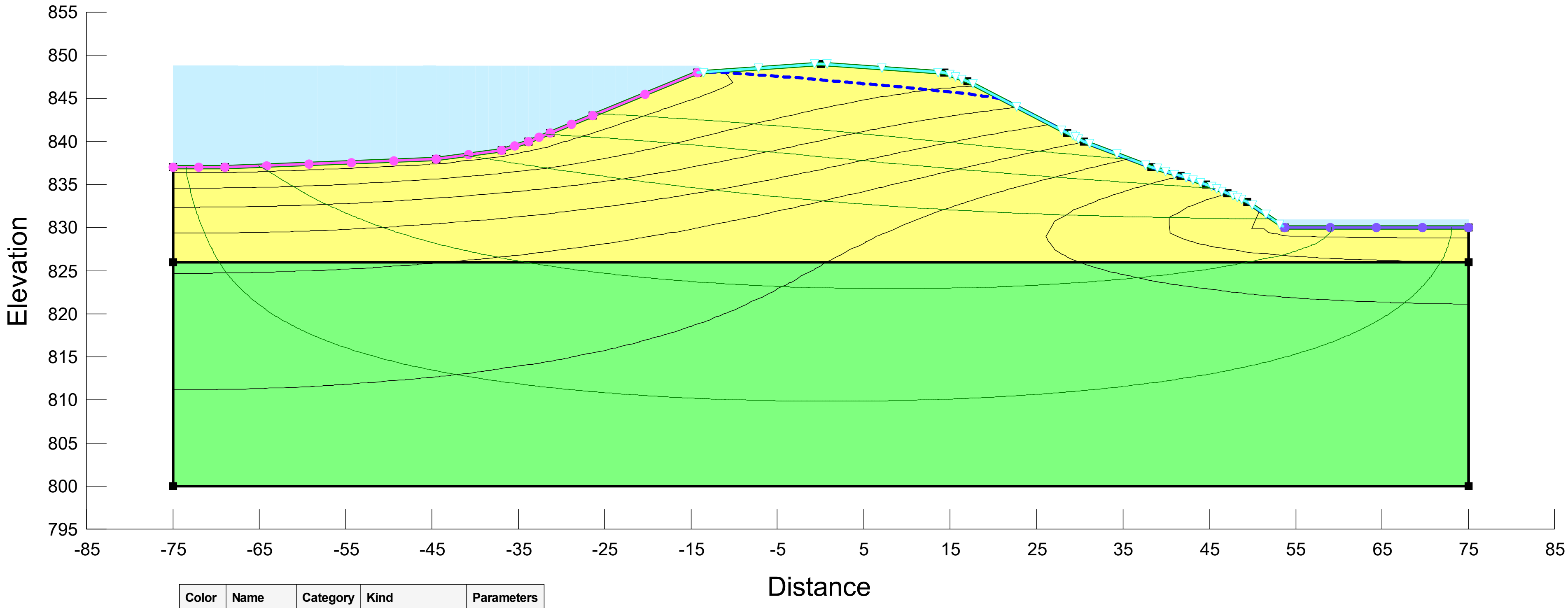
# RIGHT EMBANKMENT SECTION
# REV. 1

## ANALYSIS: Steady-State Seepage FP

By: J. Roell
Rev: M. Carden
App: M. Carpenter

| Color | Name | Hydraulic Material Model | Vol. WC. Function | K-Function | Ky'/Kx' Ratio |
|---|---|---|---|---|---|
| 🟨 | Embankment Fill | Saturated / Unsaturated | Embankment | Embankment | 0.02 |
| 🟩 | Foundation | Saturated / Unsaturated | Embankment | Embankment | 0.02 |



| Color | Name | Category | Kind | Parameters |
|---|---|---|---|---|
| 🟪 | Flood Pool | Hydraulic | Water Total Head | 848.8 ft |
| 🟦 | Open Seepage Face | Hydraulic | Water Flux | 0 ft/sec |
| 🟪 | Tailwater | Hydraulic | Water Total Head | 831 ft |

White Cloud Dam
Embankment Stability

The City of
White Cloud

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Project: 2302435

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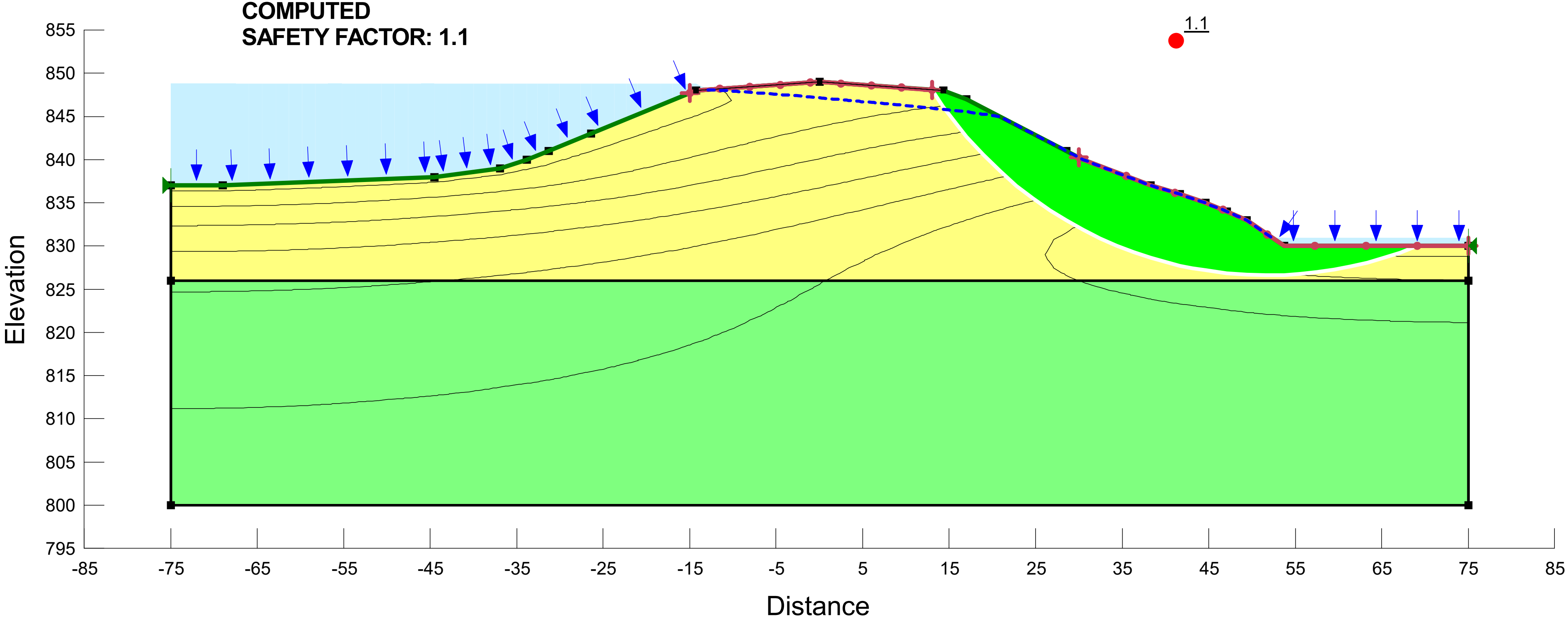
# RIGHT EMBANKMENT SECTION
REV. 1

## ANALYSIS: Slope Stability FP

By: J. Roell
Rev: M. Carden
App: M. Carpenter

| Color | Name | Slope Stability Material Model | Unit Weight (pcf) | Effective Cohesion (psf) | Effective Friction Angle (°) |
|---|---|---|---|---|---|
| 🟨 | Embankment Fill | Mohr-Coulomb | 125 | 0 | 33 |
| 🟩 | Foundation | Mohr-Coulomb | 127 | 0 | 34 |



**COMPUTED SAFETY FACTOR: 1.1**

White Cloud Dam
Embankment Stability

The City of
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# Appendix G – Structural Analysis

| | | | | | |
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| **Subject** | Spillway Wall – Compressive Strength Results | | | | |

# Concrete Strength Analysis

**Overview:**

Use the concrete core data obtained from the White Cloud Dam site to determine an equivalent in place design strength.

**References:**

1. Core Break Reports prepared by Trinity Material Consultants.
2. ACI 214.4R-10 Guide for Obtaining Cores and Interpreting Compressive Strength Results

**Concrete Core Input Values:**

Cores were prepared in accordance with ASTM C42, Air Dried.

Compressive Break Strength of Cores:; $f_{C\_1A}$ = 13549 psi;
$f_{C\_1B}$ = 11028 psi;
$f_{C\_2A}$ = 12042 psi;
$f_{C\_2B}$ = 2856 psi;
$f_{C\_3A}$ = 12786 psi;
$f_{C\_3B}$ = 7630 psi;

Core Length:; $L_{C\_1A}$ = 7.41in;
$L_{C\_1B}$ = 5.66in;
$L_{C\_2A}$ = 4.69in;
$L_{C\_2B}$ = 6.55in;
$L_{C\_3A}$ = 4.58in;
$L_{C\_3B}$ = 3.66in;

Core Diameter:; $d_{core}$ = 2.73in;

NOTE: Core 2B appears to be an outlier for the prepared samples based on the strength results and the photos of the core, it will not be considered in the strength analysis of the concrete sampled, and calculations will proceed with the remaining 5 cores.

Number of Cores:; n = 5;

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**Determine In Place Strength of Concrete:**

Constant $\alpha$ for L/d ratio:; $\alpha$ = 3e-6*1/1 psi; Ref. 2 T.9.1;

L/d Correction factors:; $F_{Ld\_C\_1A}$ = 1-(0.144*$\alpha$* $f_{C\_1B}$)*(2- $L_{C\_1A}$/ $d_{core}$)$^2$ = **1.00** ; Ref. 2 T.9.1;

$F_{Ld\_C\_1B}$ = 1-(0.144*$\alpha$* $f_{C\_1B}$)*(2- $L_{C\_1B}$/ $d_{core}$)$^2$ = **1.00** ; Ref. 2 T.9.1;

$F_{Ld\_C\_2A}$ = 1-(0.144*$\alpha$* $f_{C\_1B}$)*(2- $L_{C\_2A}$/ $d_{core}$)$^2$ = **1.00** ; Ref. 2 T.9.1;

$F_{Ld\_C\_3A}$ = 1-(0.144*$\alpha$* $f_{C\_3B}$)*(2- $L_{C\_3A}$/ $d_{core}$)$^2$ = **1.00**; Ref. 2 T.9.1;

$F_{Ld\_C\_3B}$ = 1-(0.144*$\alpha$* $f_{C\_3B}$)*(2- $L_{C\_3B}$/ $d_{core}$)$^2$ = **1.00**; Ref. 2 T.9.1;

Correction Factor for Diameter:;

By Linear Interpolation:; $F_{dia}$= -(((1-1.06)*(4- $d_{core}$/1in)/(4-2))-1) =**1.038**; Ref. 2 T.9.1;

Correction Factor for Moisture Content:; $F_{mc}$= 0.96; Ref. 2 T.9.1;

Correction Factor for Damage from Drilling :; $F_d$= 1.06 ; Ref. 2 T.9.1;

In Place Strength of Concrete:;

$f_{c\_C\_1A}$ = $F_{Ld\_C\_1A}$* $F_{dia}$* $F_{mc}$* $F_d$* $f_{C\_1A}$ =**14278**psi; Ref. 2 Eq 9.1;

$f_{c\_C\_1B}$ = $F_{Ld\_C\_1B}$* $F_{dia}$* $F_{mc}$* $F_d$* $f_{C\_1B}$ =**11649**psi;

$f_{c\_C\_2A}$ = $F_{Ld\_C\_2A}$* $F_{dia}$* $F_{mc}$* $F_d$* $f_{C\_2A}$ =**12716**psi;

$f_{c\_C\_3A}$ = $F_{Ld\_C\_3A}$* $F_{dia}$* $F_{mc}$* $F_d$* $f_{C\_3A}$ =**13502**psi;

$f_{c\_C\_3B}$ = $F_{Ld\_C\_3B}$* $F_{dia}$* $F_{mc}$* $F_d$* $f_{C\_3B}$ =**8049**psi;

**Determine Values for Uncertainty in Estimates Strength:;**

Sample Mean In Place Strength:;

$f_{cbar}$ = average($f_{c\_C\_1A}$ ,$f_{c\_C\_1B}$, $f_{c\_C\_2A}$ , $f_{c\_C\_3A}$, $f_{c\_C\_3B}$) = **12038.80**psi; Ref. 2 Eq 9.2;

Sample Standard Deviation of In Place Strength:;

$S_c$=√((($f_{c\_C\_1A}$-$f_{cbar}$)$^2$ + ($f_{c\_C\_1B}$-$f_{cbar}$)$^2$ + ($f_{c\_C\_2A}$-$f_{cbar}$)$^2$ + ($f_{c\_C\_3A}$-$f_{cbar}$)$^2$ + ($f_{c\_C\_3B}$-$f_{cbar}$)$^2$)/(n-1)) = **2433.49**psi;

Ref. 2 Eq 9.3;

Coefficient of Variation For L/d Ratio:;

$V_{Ld}$ = 1/100*( 2.5*((2 – (min($L_{C\_1A}$, $L_{C\_1B}$, $L_{C\_2A}$, $L_{C\_3A}$,$L_{C\_3B}$)/$d_{core}$))$^2$)) =**0.0109**; Ref. 2 T. 9.1;

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Coefficient of Variation For Diameter:;

By Linear Interpolation:;

$V_{dia}$= 1/100* -(((0-11.8)*(4- $d_{core}$/1in)/(4-2))-0) =**0.075**; Ref. 2 T. 9.1;

Coefficient of Variation For Moisture Content:;

$V_{mc}$ = (1/100)*2.5 =**0.025**; Ref. 2 T. 9.1;

Coefficient of Variation For Drilling Damage:;

$V_d$ = 1/100*2.5 =**0.025**; Ref. 2 T. 9.1;

Standard Deviation of In Place Strength due to Empirical Nature of Correction Factors:

$S_a = f_{cbar}*\sqrt{(V_{Ld}^2+ V_{dia}^2+ V_{mc}^2+ V_d^2)}$ =**1005.99**psi; Ref. 2 Eq. 9-4;

**Determine Equivalent Design Strength Value:;**

K-Factor for One Sided Tolerance Limit:; K = 2.74; Ref. 2 Eq 9.2;

*For N = 5, and using a 90% confidence level due to importance of structure;*

Z-Factor for One Sided Tolerance Limit:; Z = 1.28; Ref. 2 Eq 9.3;

*Use a 90% confidence level due to importance of structure*

Equivalent Design Strength:;

$f'_{ceq} = f_{cbar} -\sqrt{((K* S_c)^2 + (Z* S_a)^2)}$ = **5247.84**psi; Ref 2 Eq. 9-7;

**Conclusion:;**

***The equivalent design strength of the concrete is; $f'_{ceq}$ = 5248 psi;, with a 90% confidence that the value is equal to or less than the true value.***

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# Concrete Strength Analysis

**Overview:**

Use the concrete core data obtained from the White Cloud Dam site to evaluate an equivalent in place design strength.

**References:**

1. Core Break Reports prepared by Trinity Material Consultants.
2. ACI 214.4R-10 Guide for Obtaining Cores and Interpreting Compressive Strength Results

**Concrete Core Input Values:**

Cores were prepared in accordance with ASTM C42, Air Dried.

Compressive Break Strength of Cores:; $f_{C\_4}$ = 7167 psi;
$f_{C\_5}$ = 6862 psi;
$f_{C\_6}$ = 6318 psi;

Core Length:; $L_{C\_4}$ = 6.1in;
$L_{C\_5}$ = 5.35in;
$L_{C\_6}$ = 5.8in;

Core Diameter:; $d_{core}$ = 2.73in;

Number of Cores:; n = 3;

**Determine In Place Strength of Concrete:**

Constant $\alpha$ for L/d ratio:; $\alpha$ = 3e-6*1/1 psi; Ref. 2 T.9.1;

L/d Correction factors:; $F_{Ld\_C\_4} = 1-(.144*\alpha* f_{C\_4})*(2- L_{C\_4}/ d_{core})^2 =$**1.000**; Ref. 2 T.9.1;
$F_{Ld\_C\_5} = 1-(.144*\alpha* f_{C\_5})*(2- L_{C\_5}/ d_{core})^2 =$**1.000**; Ref. 2 T.9.1;
$F_{Ld\_C\_6} = 1-(.144*\alpha* f_{C\_6})*(2- L_{C\_6}/ d_{core})^2 =$**1.000**; Ref. 2 T.9.1;

Correction Factor for Diameter:;
By Linear Interpolation:; $F_{dia}= -(((1-1.06)*(4- d_{core}/1in)/(4-2))-1) =$**1.038**; Ref. 2 T.9.1;

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Correction Factor for Moisture Content:; $F_{mc}$= 0.96; Ref. 2 T.9.1;

Correction Factor for Damage from Drilling :; $F_d$= 1.06 ; Ref. 2 T.9.1;

In Place Strength of Concrete:;

$f_{c_C_4} = F_{Ld_C_4}{*}\ F_{dia}{*}\ F_{mc}{*}\ F_d{*}\ f_{C_4} = \underline{\mathbf{7570}}$psi;

$f_{c_C_5} = F_{Ld_C_5}{*}\ F_{dia}{*}\ F_{mc}{*}\ F_d{*}\ f_{C_5} = \underline{\mathbf{7249}}$psi; Ref. 2 Eq 9.1;

$f_{c_C_6} = F_{Ld_C_6}{*}\ F_{dia}{*}\ F_{mc}{*}\ F_d{*}\ f_{C_6} = \underline{\mathbf{6674}}$psi;

**Determine Values for Uncertainty in Estimates Strength:;**

Sample Mean In Place Strength:;

$f_{cbar}$ = average($f_{c\_ C\_4}$,$f_{c\_ C\_5}$, $f_{c\_ C\_6}$) = $\underline{\mathbf{7164.12}}$psi; Ref. 2 Eq 9.2;

Sample Standard Deviation of In Place Strength:;

$S_c=\sqrt{(((f_{c_ C_4}- f_{cbar})^2 + (f_{c_ C_5}- f_{cbar})^2 + (f_{c_ C_6}-f_{cbar})^2) / (n-1))} = \underline{\mathbf{453.89}}$psi;

Ref. 2 Eq 9.3;

Coefficient of Variation For L/d Ratio:;

$V_{Ld} = 1/100{*}( 2.5{*}((2 - (\min(L_{C\_4}, L_{C\_5}, L_{C\_6})/d_{core}))^2)) = \underline{\mathbf{0.0000}}$; Ref. 2 T. 9.1;

Coefficient of Variation For Diameter:;

By Linear Interpolation:;

$V_{dia}= 1/100{*} -(((0-11.8){*}(4- d_{core}/1in)/(4-2))-0) = \underline{\mathbf{0.075}}$; Ref. 2 T. 9.1;

Coefficient of Variation For Moisture Content:;

$V_{mc} = (1/100){*}2.5 = \underline{\mathbf{0.025}}$; Ref. 2 T. 9.1;

Coefficient of Variation For Drilling Damage:;

$V_d = 1/100{*}2.5 = \underline{\mathbf{0.025}}$; Ref. 2 T. 9.1;

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Standard Deviation of In Place Strength due to Empirical Nature of Correction Factors:

$S_a = f_{cbar}*\sqrt{(V_{Ld}^2+ V_{dia}^2+ V_{mc}^2+ V_d^2)}$ = **593.56**psi; Ref. 2 Eq. 9-4;

**Determine Equivalent Design Strength Value:;**

K-Factor for One Sided Tolerance Limit:; $K = 4.26$; Ref. 2 Eq 9.2;

*For n=3 and a 90% confidence level due to importance of structure*

Z-Factor for One Sided Tolerance Limit:; $Z = 1.28$; Ref. 2 Eq 9.3;

*Use a 90% confidence level due to importance of structure*

Equivalent Design Strength:;

$f'_{ceq} = f_{cbar} -\sqrt{((K^* S_c)^2 + (Z^* S_a)^2)}$ = **5087**psi; Ref 2 Eq. 9-7;

**Conclusion:;**

***The equivalent design strength of the concrete is; $f'_{ceq}$ = 5087 psi;, with a 90% confidence that the value is equal to or less than the true value.***

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# White Cloud Disposition Study

## Contents

## Purpose

The purpose for the following calculations is to Analyze the Existing Cast-in-Place inlet Structure & Chute at the White Cloud Dam for global stability and perform a high-level concrete strength analysis to determine overall feasibility of continued structure use.

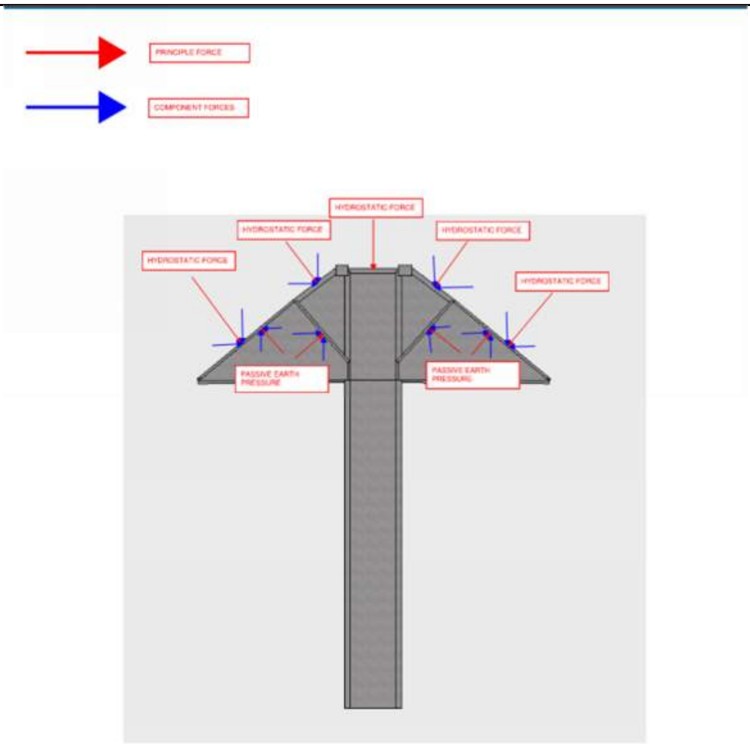

*Figure 1 – WHITE CLOUD – Combined Inlet & Chute – Plan View – Simplified Free Body Diagram*

## References

1. Drawings – DEQ drawing 526_1-14
2. Drawings – OMM Dam Reconstruction Plans
3. USACE EM 1110-2-2200 – Gravity Dam Design
4. USACE EM 1110-2-2104 – Strength Design for Reinforced Concrete Hydraulic Structures
5. ACI 318-19 – Building Code Requirements for Structural Concrete

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## Parameters

These shared Parameter Values to be used throughout all calc sections.

Unit Weights

Water: $\gamma_w = 62.5$ pcf Ref #2 Section 3.3c2a

Concrete: $\gamma_c = 150$ pcf Ref #2 Section 3.3b

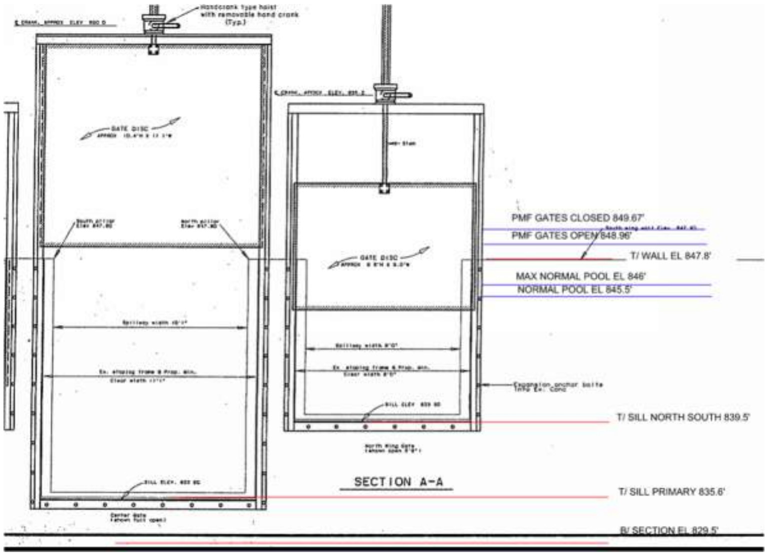

*Figure 2 – WHITE CLOUD - ELEVATIONS*

## Elevations

Top of Wall Elevation: $El_{twall} = 847.8$ ft Ref #1 Sheet 6

Top of Sill Elevation Primary: $El_{sill} = 835.6$ ft Ref #1 Sheet 6

Top of Sill Elevation N & S: $El_{sillNS} = 839.5$ ft Ref #1 Sheet 6

Bottom of Section: $El_{bwall} = 835$ ft Ref #1 Sheet 5 (Assumed)

Thickened slab t/ c: $El_{thick.slab} = 838$ ft (Assumed)

Top of Gate Elevation: $El_{tgate} = 846$ ft Ref #1 Sheet 6

Normal Pool Water Elevation: $El_{NP} = 845.5$ ft Ref #1 Sheet 6

Max Normal Pool: $El_{mNP} = 846$ ft Ref #1 Sheet 6

PMF Gate Open: $El_{PMFo} = 848.96$ ft Ref #1 Sheet 6

PMF Gate Closed: $El_{PMFc} = 849.67$ ft Ref #1 Sheet 6

Normal Pool Tail Water Elevation: $El_{NPtw} = 829.5$ ft Ref#1 Sheet

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## Load Factors – Strength Design

Water Fluid Pressure:

$LF_{wPMF} = 1.6$ Ref #4 Table 3-1

$LF_{wnp} = 2.2$ Ref #4 Table 3-1

Concrete Dead Weight:

$LF_{dead.IDF} = 0.9$ Ref #4 Table 3-1

$LF_{dead.d} = 1.2$ Ref #4 Table 3-1

$LF_{deadnp} = 2.2$ Ref #4 Table 3-1

Lateral Earth Pressure:

Driving Force: $LF_{lat.earth.d} = 1.5$ Ref #4 Table 3-1

Resisitng Force: $LF_{lat.earth.r} = 0.5$

## Stability Criteria

**Ref # 3 Table #-2, 4-1**

Usual

Sliding: $FS_{slide.n} = 3.0$ Ref # 3 Table 3-3

Overturn: Resultant within middle 1/3 of base

Extreme

Sliding: $Fs_{slide.ex} = 2.2$ Ref # 3 Table 3-3

Overturn: Resultant within Base

## Overall Stability

The stability analysis at white cloud was performed in three parts, 1. The inlet structure was considered as a standalone unit, and 2. The inlet structure, chute and wing walls were considered as a whole section and 3. A unit width cross section of an interior training wall.

**Summary:**

| Structure | Stability Type | Load Case | Required | Actual | Pass / Fail |
|---|---|---|---|---|---|
| Inlet | Overturning | Normal Pool w/ ice | Resultant within Center 1/3 | Outside Base | Fail |
| Inlet | Overturning | Maximum Normal Pool | Resultant within Center 1/3 | Outside Base | Fail |
| Inlet | Overturning | PMF Gate Open | Resultant within Base | Outside Base | Fail |
| Inlet | Overturning | PMF Gate Closed | Resultant within base | Outside Base | Fail |
| Inlet & Chute | Sliding | Normal Pool w/ ice | 3 | 14.28 | Pass |
| Inlet & Chute | Sliding | Maximum Normal Pool | 3 | 6.42 | Pass |
| Inlet & Chute | Sliding | PMF Gate Open | 2.2 | 8.67 | Pass |
| Inlet & Chute | Sliding | PMF Gate Closed | 2.2 | 7.80 | Pass |

1. Pass indicates the stability analysis meets current code requirements.

### Case 1 – Inlet Structure

#### Principal Spillway Inlet Structure - Geometry

Center of Gravity & Structure Weight – Inlet Structure Concrete - STAAD FEM Model

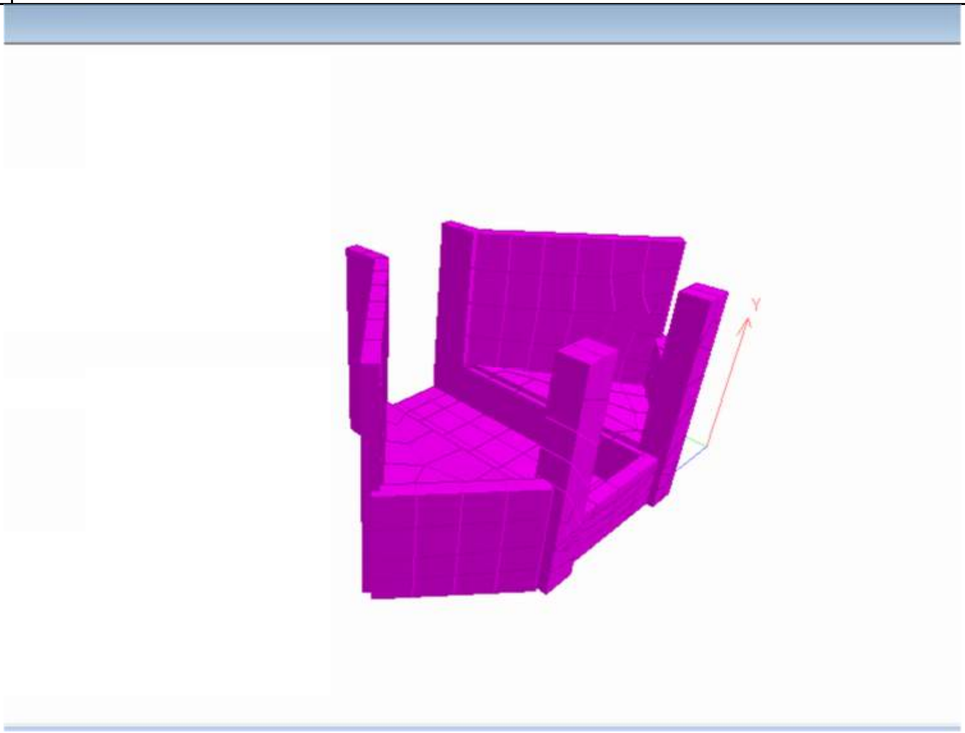
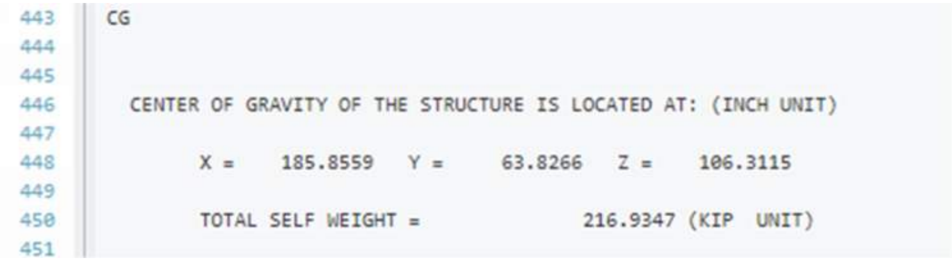


*Figure 3 – Rendering of Inlet Structure w/o Gates*



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CG


  CENTER OF GRAVITY OF THE STRUCTURE IS LOCATED AT: (INCH UNIT)

        X =    185.8559   Y =     63.8266   Z =    106.3115

        TOTAL SELF WEIGHT =             216.9347 (KIP  UNIT)

```

*Figure 4 – Calculated FEM Center of Gravity*

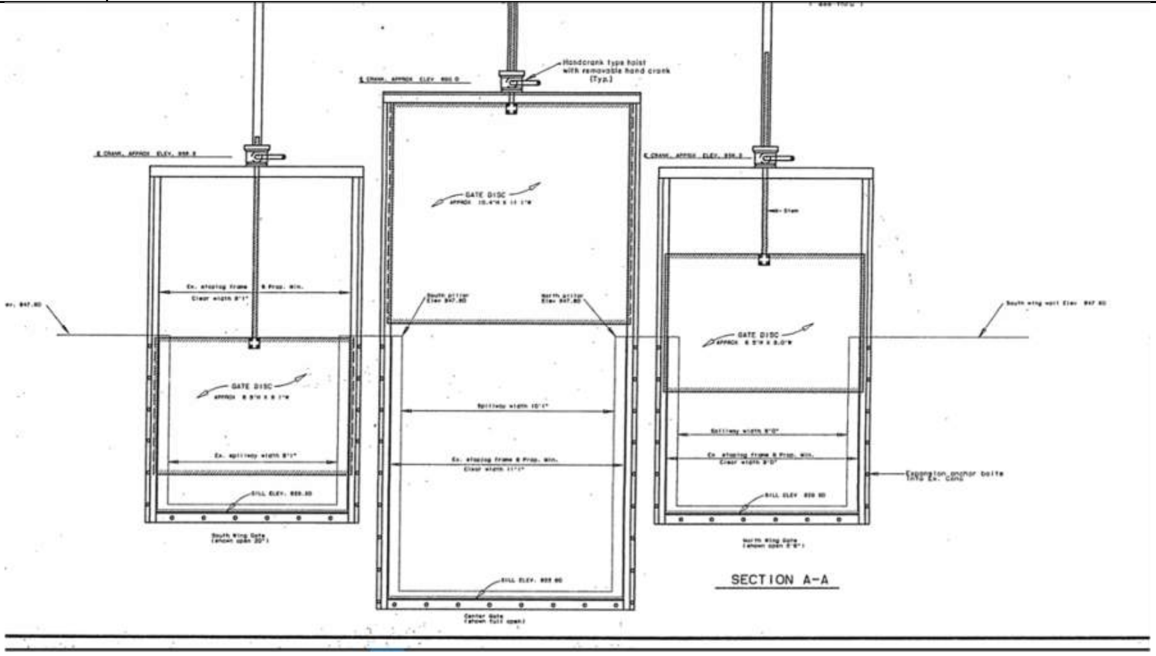

*Figure 5– Inlet Structure Reference ElevationsW*

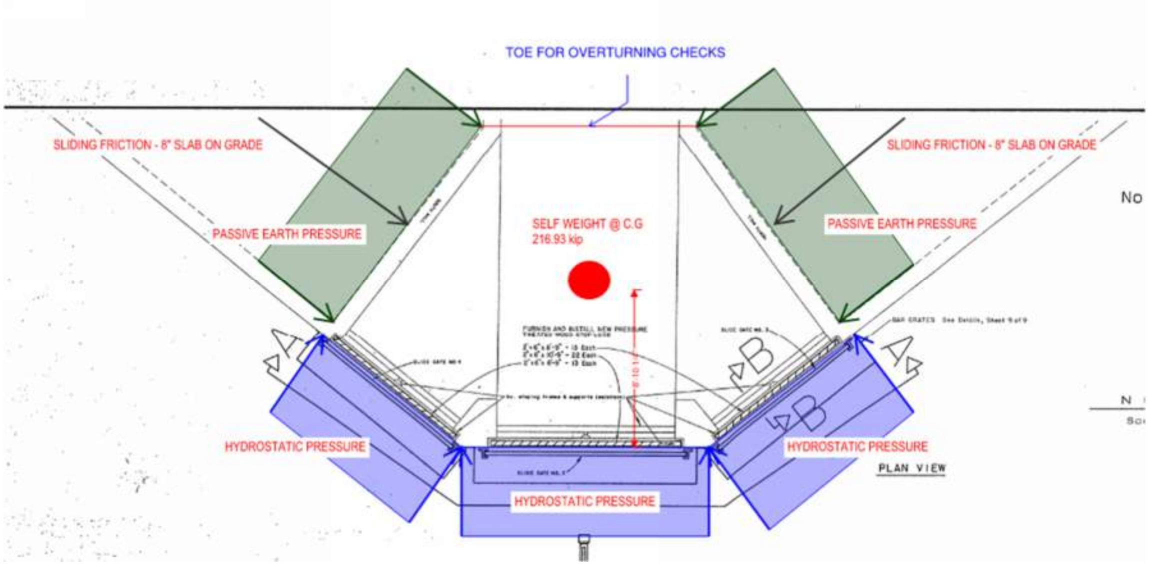

*Figure 6 – Inlet Structure Plan view - FBD*

| | | | | | | |
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| | **By** | J. Probstfeld | **Chk.** | M. Carden | **App.** | R. Price |
| | **Date** | 01/15/2024 | **Date** | 1/15/2025 | **Date** | 2/14/2025 |
| **Project No.** | 2302435 | **Document No.** | N/A | | | |
| **Subject** | White Cloud Inlet Structure – Stability Analysis | | | | | |

## Applied Resisting Forces

Overturning of this structure is resisted by the self-weight of the structure concrete and the passive soil pressures on the training walls. Sliding is resisted by the friction between the soil and the concrete of the structure and the passive soil pressures on the training wall.

Inlet Structure Concrete Self Weight:

Total Structure Length: $L = 21$ ft

Moment Arm for Self Weight : $X_{rec1} = L – (106\text{ in}) = \mathbf{12.17}$ ft

Structure Weight: $Wt_{inlet} = (216\text{ kip})$ (FEM Calculated Self Weight)

Resisting Moment: $M_{r.csw} = Wt_{inlet} * X_{rec1} = \mathbf{2628.00}$ kip_ft

Passive Earth Pressure – Inlet Structure

Restoring Forces Passive Pressure – Along Wing Walls:

Assumed soil is not saturated .

Unit weight of soil: $\gamma_{soil} = 124$ pcf

Friction angle: $\phi' = 33$ deg

Passive pressure coefficient: $K_a = \tan(45\text{ deg} + (\phi' \div 2))^2 = \mathbf{3.39}$

Height of soil behind wall: $H_{soil} = El_{twall} - El_{bwall} = \mathbf{12.80}$ ft

Total soil load: $P_{soil} = 0.5 * H_{soil}^2 * \gamma_{soil} * K_a = \mathbf{34.46}$ kip/ft

Soil Load Parallel to flow: $P_{soil1} = P_{soil} * \cos(54) = \mathbf{20.25}$ kip/ft

Tributary Width: $T_{ws} = 13.5$ ft Ref #1 Page 6

Total Soil Load Parallel to Flow: $P_{soiltotal} = LF_{lat.earth.r} * P_{soil1} * T_{ws} * 2 = \mathbf{273.42}$ kip

Moment arm for soil: $L_{soil} = (1/3) * H_{soil} = \mathbf{4.27}$ ft

Restoring Moment: $M_{r.s} = P_{soiltotal} * L_{soil} = \mathbf{1166.61}$ kip_ft

Slab on Grade Friction Forces Normal Pool:

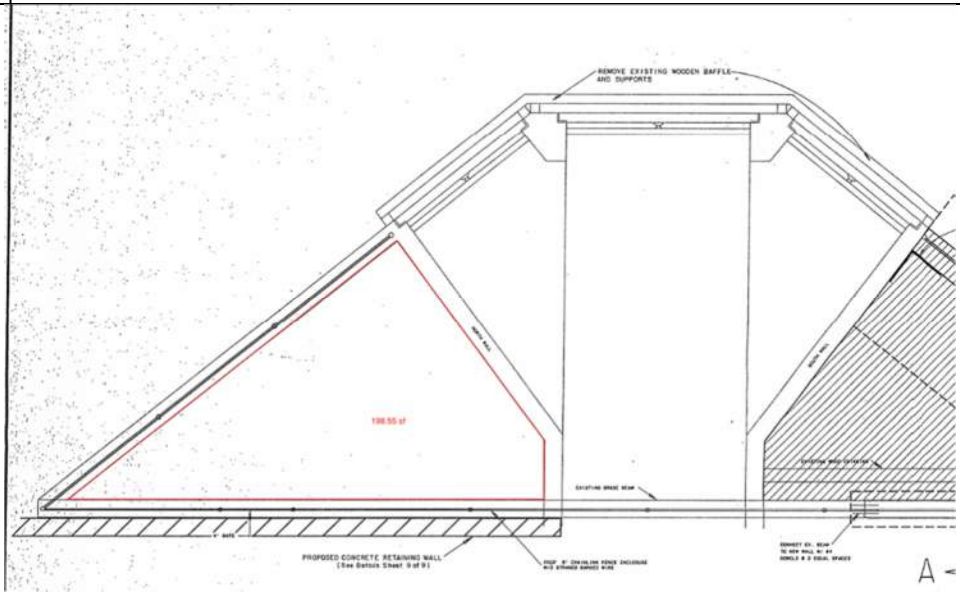

*Figure 7– 8” Slab on Grade Area*

Slab Thickness: $t_s = 8$ in

Unit Weight of Concrete: $\gamma_c = 150$ pcf

Area of Slab wedges: $A_{s.g} = 198\ \mathrm{ft}^2$

Weight of Concrete Slab: $W_s = A_{s.g} * t_s * \gamma_c = \mathbf{19.14}$ kip

Coefficient of Friction: $\mu_{c.s} = 0.7$

Resisting Force Provided by Slab Perpendicular to wall face Unfactored:

$P_{slab} = W_s * \mu_{c.s} = \mathbf{13.40}$ kip

Resisting Force Provided by Slab Parallel to flow Factored:

$P_{slab.restore} = LF_{deadnp} * P_{slab} * \cos(37) = \mathbf{23.54}$ kip

Moment Arm For Slab Appllied Force: $X_{slab} = (El_{twall} – 4\text{ in}) - El_{bwall} = \mathbf{12.47}$ ft

Restoring Moment from Slab: $M_{r.slab} = 2 * (X_{slab} * P_{slab.restore}) = \mathbf{586.94}$ kip_ft

| | | | | | | |
|---|---|---|---|---|---|---|
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## Applied Driving Forces:

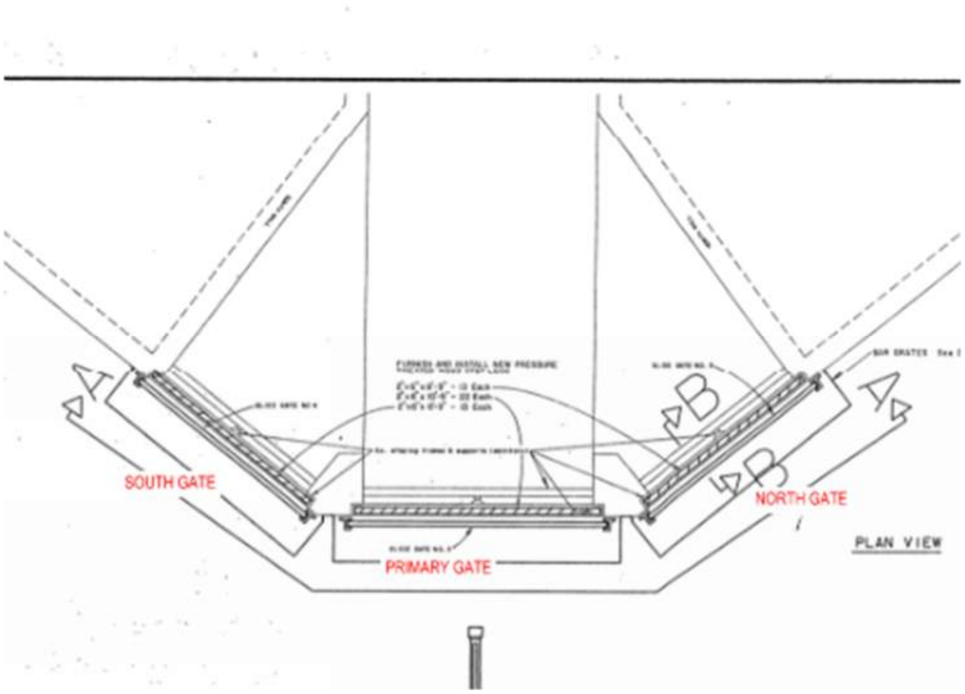

*Figure 8 – Gate Identification*

Hydrostatic Forces

The primary driving force on this structure are the hydrostatic loads applied to the Principapal, north and south gates.

Primary Gate

Maximum Normal Pool

Tributary Width: $T_{w1} = 14.75$ ft

Head Height Max Normal: $H_{w.MNP} = El_{mNP} – El_{bwall} = \mathbf{11.00}$ ft

Force Water: $F_{w.MNP} = LF_{wnp} * 0.5 * \gamma_w * H_{w.MNP}^2 = \mathbf{8.32}$ kip/ft

Driving Force Max Normal Pool: $D_{f.MNP} = (F_{w.MNP} * T_{w1}) = \mathbf{122.70}$ kip

Normal Pool w/ Ice

Ice Load: $L_i$ = 5 kip/ft

Head Height Max Normal: $H_{w \cdot NP} = El_{NP} - El_{bwall} =$ **10.50** ft

Force Water: $F_{w.NP} = LF_{wnp} * 0.5 * \gamma_w * H_{w \cdot NP}^2 =$ **7.58** kip/ft

Driving Force Normal Pool: $D_{f.NP} = (F_{w.NP} * T_{w1}) =$ **111.80** kip

Driving Force Ice: $D_{f.ice} = (L_i * T_{w1}) =$ **73.75** kip

PMF Gate Open

Head Height PMF Gate Open: $H_{w.PMFo} = El_{PMFo} - El_{bwall} =$ **13.96** ft

Force Water: $F_{w.PMFo} = LF_{wPMF} * 0.5 * \gamma_w * H_{w.PMFo}^2 =$ **6.09** kip/ft

Driving Force PMF Gate Open: $D_{f.PMFo} = (F_{w.PMFo} * T_{w1}) =$ **89.83** kip

PMF Gate Closed

Head Height PMF Gate CLosed: $H_{w.PMFc} = El_{PMFc} - El_{bwall} =$ **14.67** ft

Force Water: $F_{w.PMFc} = LF_{wPMF} * 0.5 * \gamma_w * H_{w.PMFc}^2 =$ **6.73** kip/ft

Driving Force PMF Gate Closed: $D_{f.PMFc} = (F_{w.PMFc} * T_{w1}) =$ **99.20** kip

North & South Gates

Resulatant not parallel to flow, Flow in the pricipal spillway has been defined as Y in the following calcs and corresponds to Z in FEM. X Direction Resultants on North & South gates, defined as perpendicular to the principal flow will cancel each other out in normal operating conditions for stability analysis. Conditions where only one gate is closed are not considered in this stability Analysis.

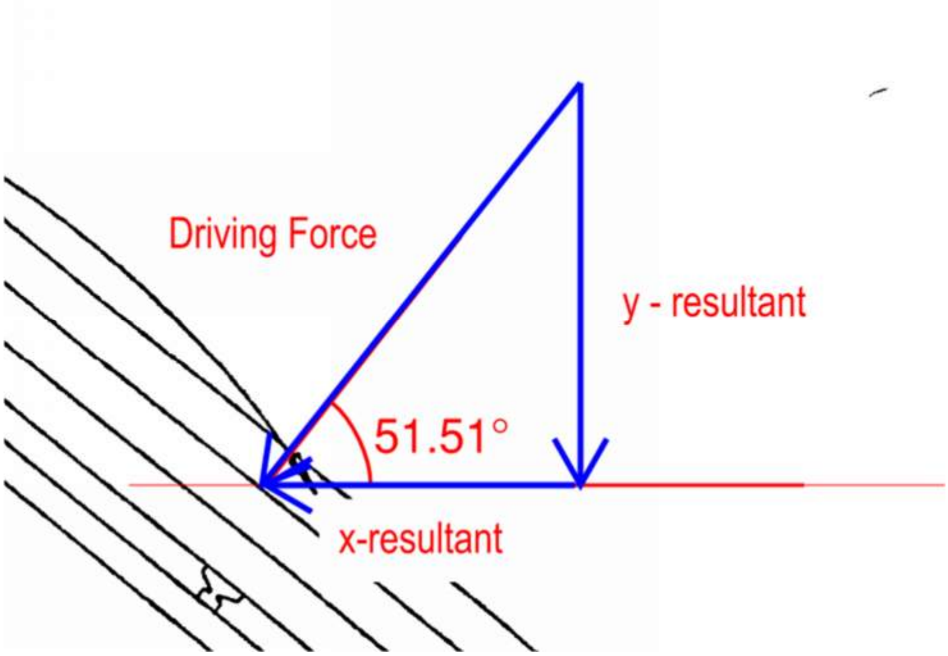

*Figure 9– Trigonomic Angles for Force Resultants*

Maximum Normal Pool

Head Height Max Normal Pool: $H_{w\cdot MNP} = El_{mNP} - El_{bwall} = \mathbf{11.00}$ ft

Tributary Width: $T_{w2} = 10.33$ ft

Force Water Maximum NP: $F_{w.MNPNS} = LF_{wnp} * 0.5 * \gamma_w * H_{w\cdot MNP}^2 = \mathbf{8.32}$ kip/ft

Force parallel to flow: $F_{d23.MNPNS} = F_{w.MNPNS} * \sin(51.51) = \mathbf{6.51}$ kip/ft

Driving Force Maximum NP: $D_{f.MNP.NS} = (F_{d23.MNPNS} * T_{w2}) = \mathbf{67.26}$ kip

Normal Pool w/ Ice

Head Height Max Normal Pool: $H_{w\cdot NP} = El_{NP} - El_{bwall} = \mathbf{10.50}$ ft

Tributary Width: $T_{w2} = 10.33$ ft

Force Water Maximum NP: $F_{w.NPNS} = LF_{wnp} * 0.5 * \gamma_w * H_{w\cdot NP}^2 = \mathbf{7.58}$ kip/ft

Force parallel to flow: $F_{d23.NPNS} = F_{w.NPNS} * \sin(51.51) = \mathbf{5.93}$ kip/ft

Force Ice: $F_{i.NP} = L_i * T_{w2} = \mathbf{51.65}$ kip

Force ice parallel to flow: $F_{i.NP.d23} = F_{i.NP} * \sin(51.51) = \mathbf{40.43}$ kip

Driving Force NP w Ice: $D_{f.NP.NS} = (F_{d23.NPNS} * T_{w2}) + F_{i.NP.d23} = \mathbf{101.71}$ kip

PMF Gate Open

Head Height Max Normal Pool: $H_{w\cdot PMFo.NS} = El_{PMFo} - El_{bwall} = \mathbf{13.96}$ ft

Tributary Width: $T_{w2} = 10.33$ ft

Force Water Maximum NP: $F_{w.PMFo.NS} = LF_{wPMF} * 0.5 * \gamma_w * H_{w\cdot PMFo.NS}^2 = \mathbf{6.09}$ kip/ft

Force parallel to flow: $F_{d23.PMFo.NS} = F_{w.PMFo.NS} * \sin(51.51) = \mathbf{4.77}$ kip/ft

Driving Force Maximum NP: $D_{f.PMFo.NS} = (F_{d23.PMFo.NS} * T_{w2}) = \mathbf{49.24}$ kip

PMF Gate Closed

Head Height Max Normal Pool: $H_{w\cdot PMFc.NS} = El_{PMFc} - El_{bwall} = \mathbf{14.67}$ ft

Tributary Width: $T_{w2} = 10.33$ ft

Force Water Maximum NP: $F_{w.PMFc.NS} = LF_{wPMF} * 0.5 * \gamma_w * H_{w\cdot PMFc.NS}^2 = \mathbf{6.73}$ kip/ft

Force parallel to flow: $F_{d23.PMFc.NS} = F_{w.PMFc.NS} * \sin(51.51) = \mathbf{5.26}$ kip/ft

Driving Force Maximum NP: $D_{f.PMFc.NS} = (F_{d23.PMFc.NS} * T_{w2}) = \mathbf{54.38}$ kip

**Flotation (Uplift Forces):**

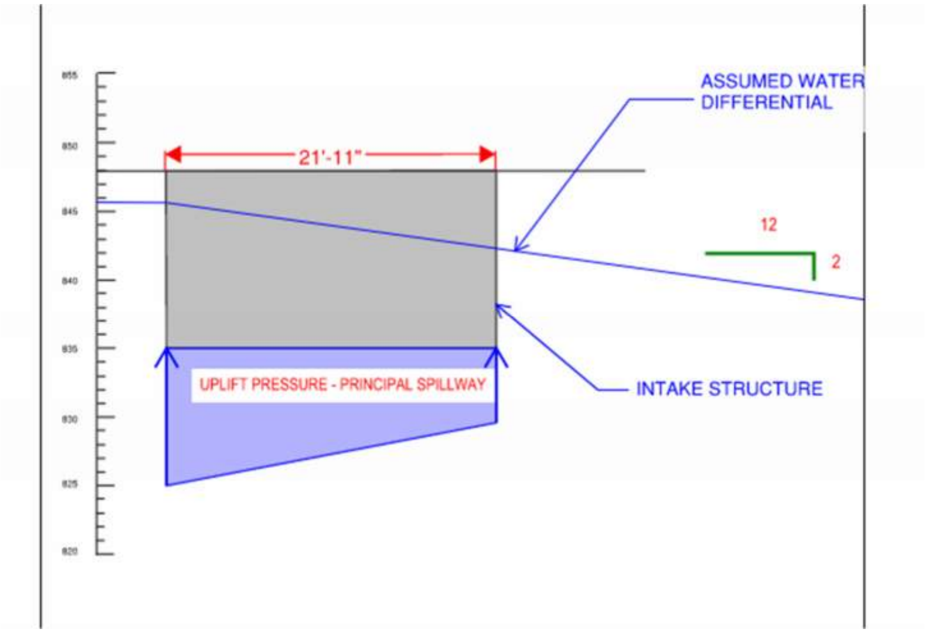

*Figure 10* ***– Uplift Loading – Intake Structure***

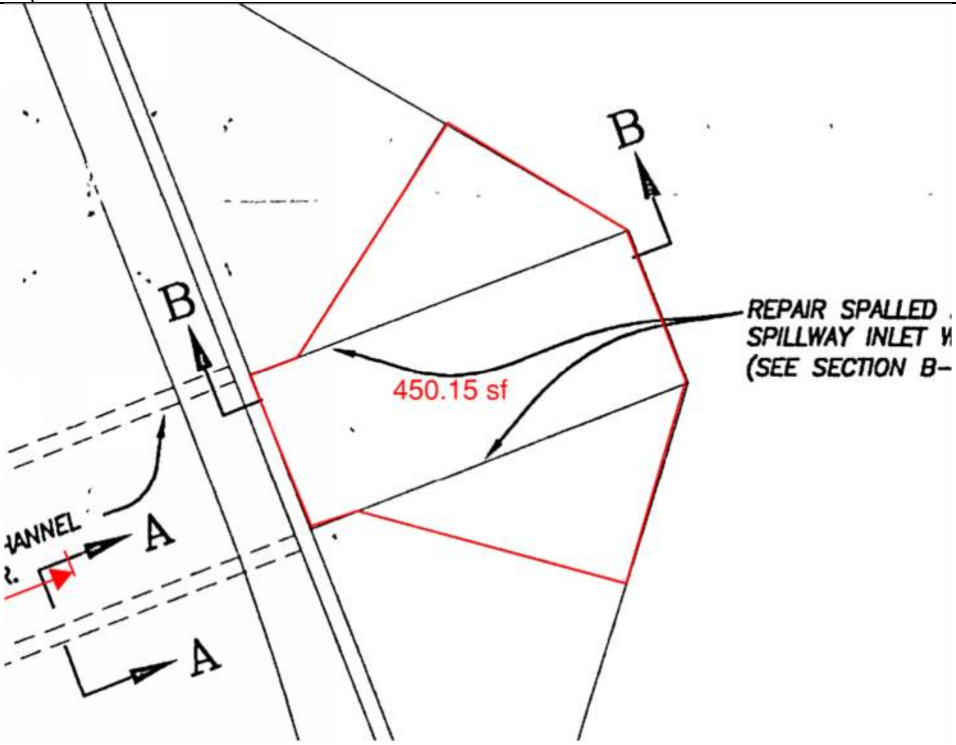

*Figure 11 – **Area of Intake***

Maximum Normal Pool

Water Differential Head: $H_{w.MNP} = El_{mNP} - El_{bwall} = \mathbf{11.00}$ ft

Flotation: $F_{float.MNP} = H_{w.MNP} * \gamma_w = \mathbf{0.69}$ kip/ft$^2$

Area Inlet Structure Base Slab (from Existing PDF drawings):

$A_{slab} = 450$ ft$^2$

Centroid of Base Area from toe: $x_{s.a} = 13.2$ ft

Total Flotation Force $F_{F.total.MNP} = F_{float.MNP} * A_{slab} = \mathbf{309.38}$ kip

Total Overturning Moment: $M_{o.MNP} = F_{F.total.MNP} * x_{s.a} = \mathbf{4083.75}$ kip_ft

Normal Pool

Water Differential Head: $H_{w.NP} = El_{NP} - El_{bwall} = \mathbf{10.50}$ ft

Flotation: $F_{float.NP} = H_{w.NP} * \gamma_w = \mathbf{0.66}$ kip/ft$^2$

Area Inlet Structure Base Slab (from Existing PDF drawings):

$A_{slab} = 450$ ft$^2$

Centroid of Base Area: $x_{s.a} = 13.2$ ft

Total Flotation Force $F_{F.total.NP} = F_{float.NP} * A_{slab} =$ **295.31** kip

Total Overturning Moment: $M_{o.NP} = F_{F.total.NP} * x_{s.a} =$**3898.12** kip_ft

PMF Gate Open

Water Differential Head: $H_{w.PMFo} = El_{PMFo} – El_{bwall} =$ **13.96** ft

Flotation: $F_{float.PMFo} = H_{w.PMFo} * \gamma_w =$**0.87** kip/ft$^2$

Area Inlet Structure Base Slab (from Existing PDF drawings):

$A_{slab} = 450$ ft$^2$

Centroid of Base Area: $x_{s.a} = 13.2$ ft

Total Flotation Force $F_{F.total.PMFo} = F_{float.PMFo} * A_{slab} =$ **392.63** kip

Total Overturning Moment: $M_{o.PMFo} = F_{F.total.PMFo} * x_{s.a} =$**5182.65** kip_ft

PMF Gate Closed

Water Differential Head: $H_{w.PMFc} = El_{PMFc} – El_{bwall} =$ **14.67** ft

Flotation: $F_{float.PMFc} = H_{w.PMFc} * \gamma_w =$**0.92** kip/ft$^2$

Area Inlet Structure Base Slab (from Existing PDF drawings):

$A_{slab} = 450$ ft$^2$

Centroid of Base Area: $x_{s.a} = 13.2$ ft

Total Flotation Force $F_{F.total.PMFc} = F_{float.PMFc} * A_{slab} =$ **412.59** kip

Total Overturning Moment: $M_{o.PMFc} = F_{F.total.PMFc} * x_{s.a} =$**5446.24** kip_ft

**Overturning Moments**

Maximum Normal Pool

Hydrostatic Forces Point of Action::

Moment Arm for HSF Gates

Moment Arm for Gate Hydrostatic Force :$X_{rec.MNP} = (El_{mNP} – El_{bwall}) / 3 =$ **3.67** ft

Overturning Moment Gate 1: $M_{g1.MNP} = D_{f.MNP} * X_{rec.MNP} =$ **449.91** kip_ft

Overturning Moment Gate 2: $M_{g2.MNP} = D_{f.MNP.NS} * X_{rec.MNP} =$ **246.62** kip_ft

Overturning Moment Gate 3: $M_{g3.MNP} = D_{f.MNP.NS} * X_{rec.MNP} =$ **246.62** kip_ft

Total Overturning Moment:

$M_{total.MNP} = M_{g1.MNP} + M_{g2.MNP} + M_{g3.MNP} + M_{o.MNP} =$**5026.90** kip_ft

Normal Pool w/ Ice

Hydrostatic Forces Point of Action::

Moment Arm for HSF Gates

Moment Arm for Gate Hydrostatic Force :$X_{rec.NP} = (El_{NP} – El_{bwall}) / 3 =$**3.50** ft

Overturning Moment Gate 1: $M_{g1.NP} = D_{f.NP} * X_{rec.NP} =$ **391.30** kip_ft

Overturning Moment Gate 2: $M_{g2.NP} = D_{f.NP.NS} * X_{rec.NP} =$ **355.99** kip_ft

Overturning Moment Gate 3: $M_{g3.NP} = D_{f.NP.NS} * X_{rec.NP} =$ **355.99** kip_ft

Overturning Moment Gate 1 Ice: $M_{g4.NP} = D_{f.ice} * ((El_{NP} – El_{bwall}) – ( 6 \text{ in})) =$ **737.50** kip_ft

Overturning Moment Gate 2 Ice: $M_{g5.NP} = F_{i.NP.d23} * ((El_{NP} – El_{bwall}) – ( 6 \text{ in})) =$ **404.27** kip_ft

Overturning Moment Gate 3 Ice: $M_{g6.NP} = F_{i.NP.d23} * ((El_{NP} – El_{bwall}) – ( 6 \text{ in})) =$ **404.27** kip_ft

Total Overturning Moment:

$M_{total.NP} = M_{g1.NP} + M_{g2.NP} + M_{g3.NP} + M_{o.NP} + M_{g4.NP} + M_{g5.NP} + M_{g6.NP} =$**6547.46** kip_ft

PMF Gate Open

Hydrostatic Forces Point of Action::

Moment Arm for HSF Gates

Moment Arm for Gate Hydrostatic Force :$X_{rec.PMFo} = (El_{PMFo} – El_{bwall}) / 3 =$**4.65** ft

Overturning Moment Gate 1: $M_{g1.PMFo} = D_{f.PMFo} * X_{rec.PMFo} =$ **418.00** kip_ft

Overturning Moment Gate 2: $M_{g2.PMFo} = D_{f.PMFo.NS} * X_{rec.PMFo} =$ **229.13** kip_ft

Overturning Moment Gate 3: $M_{g3.PMFo} = D_{f.PMFo.NS} * X_{rec.PMFo} =$ **229.13** kip_ft

Total Overturning Moment:

$M_{total.PMFo} = M_{g1.PMFo} + M_{g2.PMFo} + M_{g3.PMFo} + M_{o.PMFo} =$**6058.92** kip_ft

PMF Gate Closed

Hydrostatic Forces Point of Action::

Moment Arm for HSF Gates

Moment Arm for Gate Hydrostatic Force :$X_{rec.PMFc} = (El_{PMFc} – El_{bwall}) / 3 =$**4.89** ft

Overturning Moment Gate 1: $M_{g1.PMFc} = D_{f.PMFc} * X_{rec.PMFc} =$ **485.08** kip_ft

Overturning Moment Gate 2: $M_{g2.PMFc} = D_{f.PMFc.NS} * X_{rec.PMFc} =$ **265.90** kip_ft

Overturning Moment Gate 3: $M_{g3.PMFc} = D_{f.PMFc.NS} * X_{rec.PMFc} =$ **265.90** kip_ft

Total Overturning Moment:

$M_{total.PMFc} = M_{g1.PMFc} + M_{g2.PMFc} + M_{g3.PMFc} + M_{o.PMFc} =$**6463.12** kip_ft

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| | **By** | J. Probstfeld | **Chk.** M. Carden | **App.** | R. Price |
| | **Date** | 01/15/2024 | **Date** 1/15/2025 | **Date** | 2/14/2025 |
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Resultant Locations

Resultant Location MNP: $X_{MNP} = ((M_{r.csw} + M_{r.s} + M_{r.slab}) – (M_{total.MNP})) / (Wt_{inlet})$ = **-2.99** ft

Resultant Location NP w/ ice: $X_{NP} = ((M_{r.csw} + M_{r.s} + M_{r.slab}) – (M_{total.NP})) / (Wt_{inlet})$ = **-10.03** ft

Resultant Location PMFo: $X_{PMFo} = ((M_{r.csw} + M_{r.s} + M_{r.slab}) – (M_{total.PMFo})) / (Wt_{inlet})$ = **-7.77** ft

Resultant Location PMFc: $X_{PMFc} = ((M_{r.csw} + M_{r.s} + M_{r.slab}) – (M_{total.PMFc})) / (Wt_{inlet})$ = **-9.64** ft

Inlet Length: $W_{inlet}$ = 21.75 ft

Middle Third: $X_{third} = (W_{inlet}) / 3$ = **7.25** ft

$check_{otMNP}$ = If(AND($X_{MNP} > X_{third}$, $X_{MNP} < 2* X_{third}$), "Resultant Within Center 1/3 of Base Width for Maximum normal Pool", "Geometry Not OK") = **"Geometry Not OK"**

$check_{otNP}$ = If(AND($X_{NP} > X_{third}$, $X_{NP} < 2* X_{third}$), "Resultant Within Center 1/3 of Base Width for Normal Pool with Ice", "Geometry Not OK") = **"Geometry Not OK"**

$check_{otPMFo}$ = If(AND($X_{PMFo} > 0$ ft, $X_{PMFo} < W_{inlet}$), "Resultant Within Base Width for PMF Gate Open", "Geometry Not OK") = **"Geometry Not OK"**

$check_{otPMFc}$ = If(AND($X_{PMFc} > 0$ ft, $X_{PMFc} < W_{inlet}$), "Resultant Within Base Width for PMF Gate Closed", "Geometry Not OK") = **"Geometry Not OK"**

**Summary of Stability Results:**

Overturning:
$check_{otMNP}$ = **"Geometry Not OK"**
$check_{otNP}$ =**"Geometry Not OK"**
$check_{otPMFo}$ =**"Geometry Not OK"**
$check_{otPMFc}$ =**"Geometry Not OK"**

### Case 2 – Inlet, Chute & Wing Walls

The total estimated available resisting passive forces for the maximum normal pool condition exceeds the estimated driving (overturning) forces, therefore the geometry if ok by inspection. However, during normal pool with ice loading the overturning stability results do not meet current industry standards.

*Required Resisting Weights Usual Conditions*

Sliding of Inlet Resisted By downstream structures

### Principal Spillway, Chute & Wing Wall Geometry

Center of Gravity & Structure Weight – Inlet Structure & Chute Concrete - STAAD FEM Model

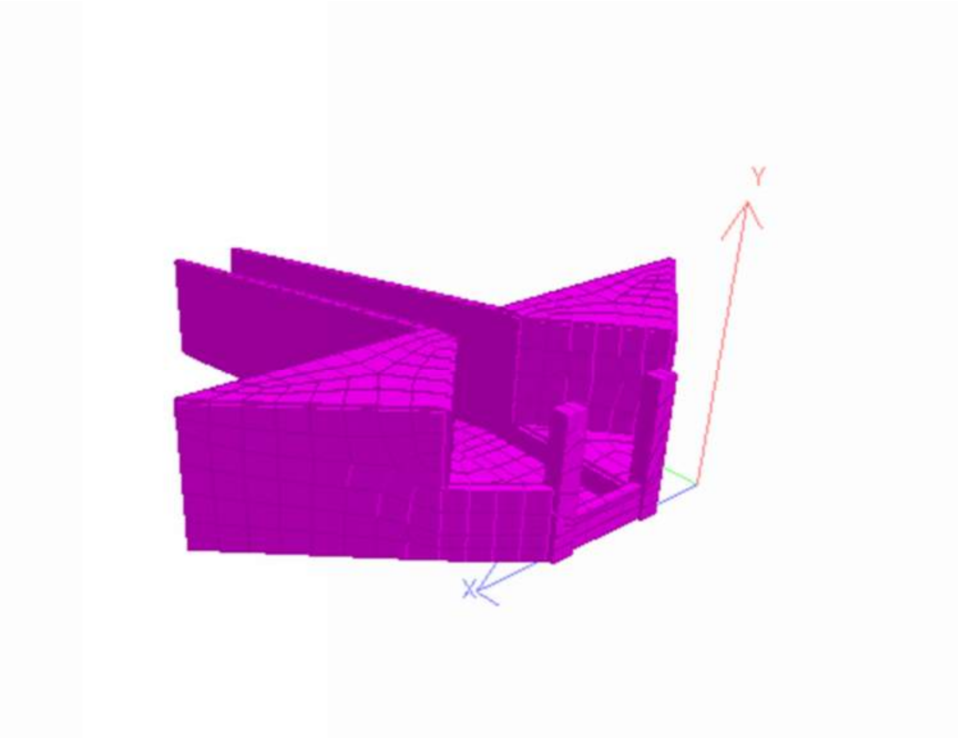
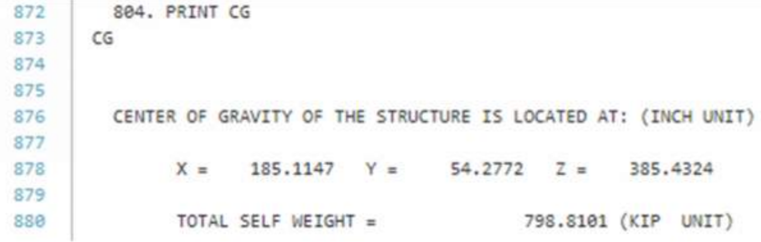


*Figure 12 – Intake & Chute FEM*



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  CENTER OF GRAVITY OF THE STRUCTURE IS LOCATED AT: (INCH UNIT)

       X =    185.1147   Y =     54.2772   Z =     385.4324

       TOTAL SELF WEIGHT =             798.8101 (KIP  UNIT)
```

*Figure 13 – Center of Gravity Intake & Chute*

## Restoring Loads:

Restoring Loads Inlet Structure:

Moment Arm for Self Weight : $X_{rec1}$ =12.3 ft

Structure Weight: $Wt_{inlet.IDF}$ = (798 kip)=**798.00** *kip*

Resisting Moment: $M_{r.csw.I} = Wt_{inlet.IDF} * X_{rec1}$ = **9815.40** kip_ft

Restoring Forces Passive Pressure – Along Interior Wing Walls:

Unit weight of soil: $\gamma_{soil}$ = 125 pcf

Friction angle: $\phi'$ = 30 deg

Passive pressure coefficient: $K_a = \tan(45 \text{ deg} + (\phi' \div 2))^2$ = **3.00**

Height of soil behind wall: $H_{soil} = El_{twall} - El_{bwall}$ = **12.80** ft

Total soil load: $P_{soil.1} = 0.5 * H_{soil}{}^2 * \gamma_{soil} * K_a$ = **30.72** kip/ft

Soil Load Parallel to flow: $P_{soil1.1} = P_{soil.1} * \cos(54)$ =**18.06** kip/ft

Tributary Width: $T_{ws.1}$ = 16.75 ft Ref #1 Page 6

Total Soil Load Parallel to Flow: $P_{soiltotal.1} = P_{soil1} * T_{ws.1} * 2$ =**678.49** kip

Restoring Forces Passive Pressure – Along Exterior Wing Walls:

Unit weight of soil: $\gamma_{soil}$ = 125 pcf

Friction angle: $\phi'$ = 30 deg

Passive pressure coefficient: $K_a = \tan(45 \text{ deg} + (\phi' \div 2))^2$ = **3.00**

Height of soil behind wall: $H_{soil} = El_{twall} - El_{bwall}$ = **12.80** ft

Total soil load: $P_{soil.2} = 0.5 * H_{soil}{}^2 * \gamma_{soil} * K_a$ = **30.72** kip/ft

Soil Load Parallel to flow: $P_{soil1.2} = P_{soil.2} * \cos(37)$ =**24.53** kip/ft

Tributary Width: $T_{ws.2}$ = 26.75 ft Ref #1 Page 6

Total Soil Load Parallel to Flow: $P_{soiltotal.2} = P_{soil1.2} * T_{ws.2} * 2 =$**1312.57** kip

## Driving & Overturning Forces:

Hydrostatic Forces

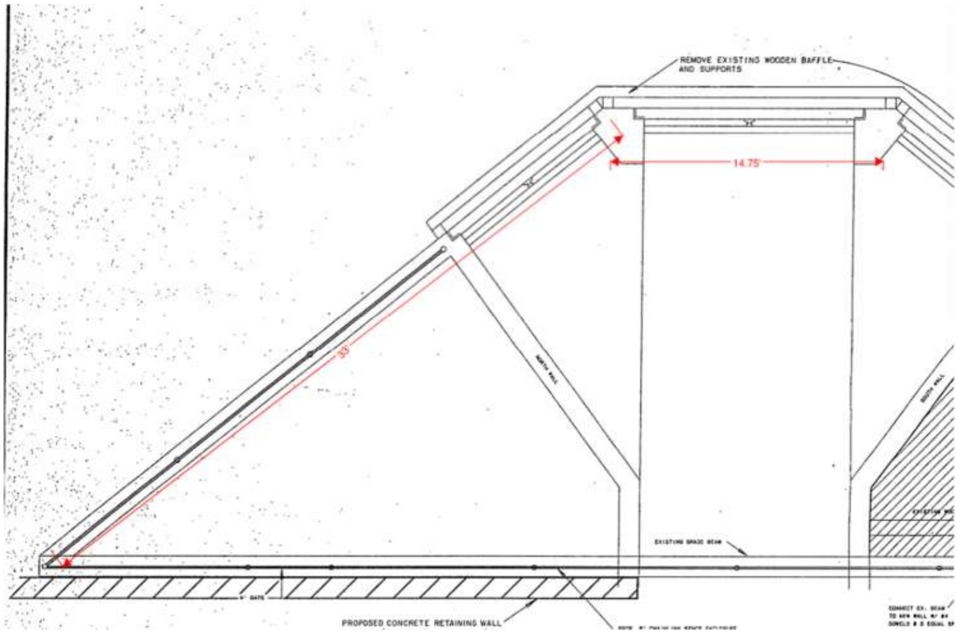

*Figure 14 – Gate Dimensions*

**Primary Gate:**

Maximum Normal Pool

Tributary Width: $T_{w1} = 14.75$ ft

Head Height Max Normal: $H_{w.MNP} = El_{mNP} – El_{bwall} = \mathbf{11.00}$ ft

Force Water: $F_{w.MNP} = 0.5 * \gamma_w * H_{w.MNP}^2 = \mathbf{3.78}$ kip/ft

Driving Force Max Normal Pool: $D_{f.MNP} = (F_{w.MNP} * T_{w1}) = \mathbf{55.77}$ kip

Normal Pool w/ Ice

Ice Load: $L_i = 5$ kip/ft

Head Height Max Normal: $H_{w \cdot NP} = El_{NP} – El_{bwall} = \mathbf{10.50}$ ft

Force Water: $F_{w.NP} = 0.5 * \gamma_w * H_{w \cdot NP}^2 = \mathbf{3.45}$ kip/ft

Driving Force Normal Pool: $D_{f.NP} = (F_{w.NP} * T_{w1}) = \mathbf{50.82}$ kip

Driving Force Ice: $D_{f.ice} = (L_i * T_{w1}) = \mathbf{73.75}$ kip

PMF Gate Open

Head Height PMF Gate Open: $H_{w.PMFo} = El_{PMFo} - El_{bwall} =$ **13.96** ft

Force Water: $F_{w.PMFo} = 0.5 * \gamma_w * H_{w.PMFo}^2 =$**6.09** kip/ft

Driving Force PMF Gate Open: $D_{f.PMFo} = (F_{w.PMFo} * T_{w1}) =$**89.83** kip

PMF Gate Closed

Head Height PMF Gate CLosed: $H_{w.PMFc} = El_{PMFc} - El_{bwall} =$ **14.67** ft

Force Water: $F_{w.PMFc} = 0.5 * \gamma_w * H_{w.PMFc}^2 =$**6.73** kip/ft

Driving Force PMF Gate Closed: $D_{f.PMFc} = (F_{w.PMFc} * T_{w1}) =$**99.20** kip

**North & South Gates :**

Resulatant not parallel to flow, X Direction Resultants Cancel gate 2 & 3

Maximum Normal Pool

Head Height Max Normal Pool: $H_{w \cdot MNP} = El_{mNP} - El_{bwall} =$ **11.00** ft

Tributary Width: $T_{w2} = 33$ ft

Force Water Maximum NP: $F_{w.MNPNS} = 0.5 * \gamma_w * H_{w \cdot MNP}^2 =$**3.78** kip/ft

Force parallel to flow: $F_{d23.MNPNS} = F_{w.MNPNS} * \sin(51.51) =$ **2.96** kip/ft

Driving Force Maximum NP: $D_{f.MNP.NS} = (F_{d23.MNPNS} * T_{w2}) =$**97.67** kip

Normal Pool w/ Ice

Head Height Normal Pool: $H_{w \cdot NP} = El_{NP} - El_{bwall} =$ **10.50** ft

Tributary Width: $T_{w2} = 33$ ft

Force Water NP: $F_{w.NPNS} = 0.5 * \gamma_w * H_{w \cdot NP}^2 =$**3.45** kip/ft

Force parallel to flow: $F_{d23.NPNS} = F_{w.NPNS} * \sin(51.51) =$ **2.70** kip/ft

Force Ice: $F_{i.NP} = L_i * T_{w2} =$**165.00** kip

Force ice parallel to flow: $F_{i.NP.d23} = F_{i.NP} * \sin(51.51) =$ **129.15** kip

Driving Force NP w Ice: $D_{f.NP.NS} = (F_{d23.NPNS} * T_{w2}) + F_{i.NP.d23} =$**218.14** kip

PMF Gate Open

Head Height Max Normal Pool: $H_{w \cdot PMFo.NS} = El_{PMFo} - El_{bwall} =$ **13.96** ft

Tributary Width: $T_{w2}$ = 33 ft

Force Water PMF Open: $F_{w.PMFo.NS} = 0.5 * \gamma_w * H_{w\cdot PMFo.NS}^2$ =**6.09** kip/ft

Force parallel to flow: $F_{d23.PMFo.NS} = F_{w.PMFo.NS} * \sin(51.51)$ = **4.77** kip/ft

Driving Force PMF Open: $D_{f.PMFo.NS} = (F_{d23.PMFo.NS} * T_{w2})$ =**157.30** kip

PMF Gate Closed

Head Height Max Normal Pool: $H_{w\cdot PMFc.NS} = El_{PMFc} - El_{bwall}$ = **14.67** ft

Tributary Width: $T_{w2}$ = 33 ft

Force Water PMF Closed: $F_{w.PMFc.NS} = 0.5 * \gamma_w * H_{w\cdot PMFc.NS}^2$ =**6.73** kip/ft

Force parallel to flow: $F_{d23.PMFc.NS} = F_{w.PMFc.NS} * \sin(51.51)$ = **5.26** kip/ft

Driving Force PMF Closed: $D_{f.PMFc.NS} = (F_{d23.PMFc.NS} * T_{w2})$ =**173.71** kip

**Uplift:**

Maximum Normal Pool

Differential Head Upstream Extents: $H_{w.MNP} = El_{mNP} - El_{NPtw}$ = **16.50** ft

Flotation: $F_{float.MNP} = H_{w.MNP} * \gamma_w$ =**1.03** kip/ft$^2$

Area Inlet Structure Base Slab (from Existing PDF drawings):

$A_{slab}$ = 450 ft$^2$

Centroid of Base Area: $x_{s.a}$ = 13.2 ft

Total Flotation Force $F_{F.total.MNP} = F_{float.MNP} * A_{slab}$ = **464.06** kip

Normal Pool

Water Differential Head: $H_{w.NP} = El_{NP} - El_{NPtw}$ = **16.00** ft

Flotation: $F_{float.NP} = H_{w.NP} * \gamma_w$ =**1.00** kip/ft$^2$

Area Inlet Structure Base Slab (from Existing PDF drawings):

$A_{slab}$ = 450 ft$^2$

Centroid of Base Area: $x_{s.a}$ = 13.2 ft

Total Flotation Force $F_{F.total.NP} = F_{float.NP} * A_{slab}$ = **450.00** kip

PMF Gate Open

Water Differential Head: $H_{w.PMFo} = El_{PMFo} - El_{NPtw}$ = **19.46** ft

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| | **By** | J. Probstfeld | **Chk.** | M. Carden | **App.** | R. Price |
| | **Date** | 01/15/2024 | **Date** | 1/15/2025 | **Date** | 2/14/2025 |

| **Project No.** | 2302435 | **Document No.** | N/A |
|---|---|---|---|
| **Subject** | White Cloud Inlet Structure – Stability Analysis | | |

Flotation: $F_{\text{float.PMFo}} = H_{\text{w.PMFo}} * \gamma_{w} =$ **1.22** kip/ft$^2$

Area Inlet Structure Base Slab (from Existing PDF drawings):

$A_{\text{slab}} = 450\ \text{ft}^2$

Centroid of Base Area: $x_{\text{s.a}} = 13.2$ ft

Total Flotation Force $F_{\text{F.total.PMFo}} = F_{\text{float.PMFo}} * A_{\text{slab}} =$ **547.31** kip

PMF Gate CLosed

Water Differential Head: $H_{\text{w.PMFc}} = El_{\text{PMFc}} - El_{\text{NPtw}} =$ **20.17** ft

Flotation: $F_{\text{float.PMFc}} = H_{\text{w.PMFc}} * \gamma_{w} =$ **1.26** kip/ft$^2$

Area Inlet Structure Base Slab (from Existing PDF drawings):

$A_{\text{slab}} = 450\ \text{ft}^2$

Centroid of Base Area: $x_{\text{s.a}} = 13.2$ ft

Total Flotation Force $F_{\text{F.total.PMFc}} = F_{\text{float.PMFc}} * A_{\text{slab}} =$ **567.28** kip

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| | | | | | |
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| **Subject** | White Cloud Inlet Structure – Stability Analysis | | | | |

**Driving Forces**

Maximum Normal Pool

HydroStatic Primary Gate: $D_{f.MNP} = \mathbf{55.77}$ kip

Hydrostatic Wing Walls: $D_{f.MNP.NS} = \mathbf{97.67}$ kip

Total: $D_{f.total.MNP} = D_{f.MNP} + D_{f.MNP.NS} = \mathbf{153.44}$ kip

Normal Pool w/ Ice

HydroStatic Primary Gate: $D_{f.NP} = \mathbf{50.82}$ kip

Driving Force Ice: $D_{f.ice} = \mathbf{73.75}$ kip

Hydrostatic Wing Walls: $D_{f.NP.NS} = \mathbf{218.14}$ kip

Total: $D_{f.total.NP} = D_{f.NP} + D_{f.ice} + D_{f.NP.NS} = \mathbf{342.71}$ kip

PMF Gate Open

HydroStatic Primary Gate: $D_{f.PMFo} = \mathbf{89.83}$ kip

Hydrostatic Wing Walls: $D_{f.PMFo.NS} = \mathbf{157.30}$ kip

Total: $D_{f.total.PMFo} = D_{f.PMFo} + D_{f.PMFo.NS} = \mathbf{247.13}$ kip

PMF Gate Closed

HydroStatic Primary Gate: $D_{f.PMFc} = \mathbf{99.20}$ kip

Hydrostatic Wing Walls: $D_{f.PMFc.NS} = \mathbf{173.71}$ kip

Total: $D_{f.total.PMFc} = D_{f.PMFc} + D_{f.PMFc.NS} = \mathbf{272.91}$ kip

**Resisting Forces:**

**Frictional Forces:**

Friction Factor: $\mu = 0.6$

Base Friction MNP: $P_{f.MNP} = (Wt_{inlet.IDF} - F_{F.total.MNP}) * \mu = \mathbf{200.36}$ kip

Base Friction NP: $P_{f.NP} = (Wt_{inlet.IDF} - F_{F.total.NP}) * \mu = \mathbf{208.80}$ kip

Base Friction PMFo: $P_{f.PMFo} = (Wt_{inlet.IDF} - F_{F.total.PMFo}) * \mu = \mathbf{150.41}$ kip

Base Friction PMFc: $P_{f.PMFc} = (Wt_{inlet.IDF} - F_{F.total.PMFc}) * \mu = \mathbf{138.43}$ kip

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| **Date** | 01/15/2024 | **Date** | 1/15/2025 | **Date** | 2/14/2025 |
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**Passive Earth Pressure:**

Wing Wall Interior: $P_{soiltotal.1}$ =**678.49** kip

Wing Wall Exterior: $P_{soiltotal.2}$ = **1312.57** kip

**Total of Resisting Forces:**

Maximum Normal Pool: $P_{total.MNP} = P_{soiltotal.1} + P_{soiltotal.2} + P_{f.MNP}$ =**2191.43** kip

Normal Pool with Ice: $P_{total.NP} = P_{soiltotal.1} + P_{soiltotal.2} + P_{f.NP}$ =**2199.87** kip

PMF Gate Open: $P_{total.PMFo} = P_{soiltotal.1} + P_{soiltotal.2} + P_{f.PMFo}$ =**2141.48** kip

PMF Gate Closed: $P_{total.PMFc} = P_{soiltotal.1} + P_{soiltotal.2} + P_{f.PMFc}$ =**2129.50** kip

$check_{MNP.s}$ = If(($P_{total.MNP}$ / $D_{f.total.MNP}$) > 2.0, “Maximum Normal Pool Stable in Sliding”, “Geometry Not OK”) = **"Maximum Normal Pool Stable in Sliding"**

$check_{NP.s}$ = If(($P_{total.NP}$ / $D_{f.total.NP}$) > 2.0, “Normal Pool with Ice Stable in Sliding”, “Geometry Not OK”) = **"Normal Pool with Ice Stable in Sliding"**

$check_{PMFo.s}$ = If(($P_{total.PMFo}$ / $D_{f.total.PMFo}$) > 1.3, “PMF Gate Open Stable in Sliding”, “Geometry Not OK”) = **"PMF Gate Open Stable in Sliding"**

$check_{PMFc.s}$ = If(($P_{total.PMFc}$ / $D_{f.total.PMFc}$) > 1.3, “PMF Gate Closed Stable in Sliding”, “Geometry Not OK”) = **"PMF Gate Closed Stable in Sliding"**

**Summary of Stability Results:**

Sliding:

Factor of Safety Maximum Normal Pool: $FS_{MNP} = P_{total.MNP} / D_{f.total.MNP}$ = **14.28**

Factor of Safety Normal Pool: $FS_{MNP} = P_{total.NP} / D_{f.total.NP}$ = **6.42**

Factor of Safety PMF Gate Open: $FS_{MNP} = P_{total.PMFo} / D_{f.total.PMFo}$ = **8.67**

Factor of Safety PMF Gate Closed: $FS_{MNP} = P_{total.PMFc} / D_{f.total.PMFc}$ = **7.80**

$check_{MNP.s}$ = **"Maximum Normal Pool Stable in Sliding"**
$check_{NP.s}$ =**"Normal Pool with Ice Stable in Sliding"**
$check_{PMFo.s}$ =**"PMF Gate Open Stable in Sliding"**
$check_{PMFc.s}$ =**"PMF Gate Closed Stable in Sliding"**

## STRENGTH CHECKS

The following calculations are to perform a high level strength analysis of the white cloud dam concrete structure.

Summary:

| Structural Component | Maximum Shear | Shear Capacity | Maximum Moment | Moment Capacity | Pass / Fail | Demand to Capcity ratio |
|---|---|---|---|---|---|---|
| Intake Piers | 58.77 kip | 22.4 kip | 215.5 kip*ft | 41.17 kip*ft | Fail | 5.23 |
| Intake Wall | 5.81 kip | 9.6 kip | 16.07 kip*ft | 3.94 kip*ft | Fail | 4.07 |
| Chute Wall – Tall | 5 kip | 9.6 kip | 23.21 kip*ft | 3.94 kip*ft | Fail | 5.89 |
| Chute Wall - Short | 5.81 kip | 9.6 kip | 3.51 kip*ft | 3.94 kip*ft | Pass | 0.89 |
| Slab | 7.56 kip | 8.8 kip | 18.91 kip*ft | 4.28 kip*ft | Fail | 4.42 |

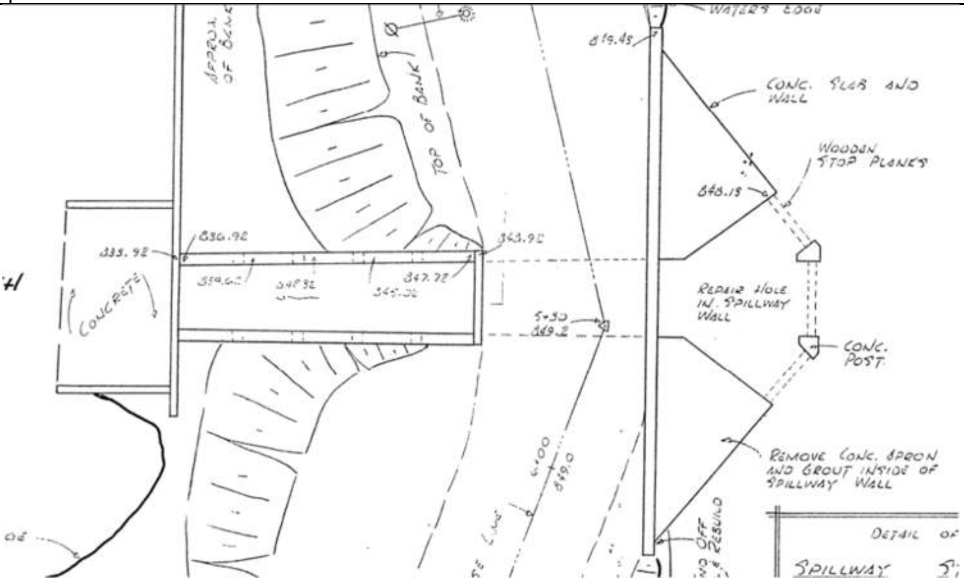

*Figure 15 – Overall Site Plan*

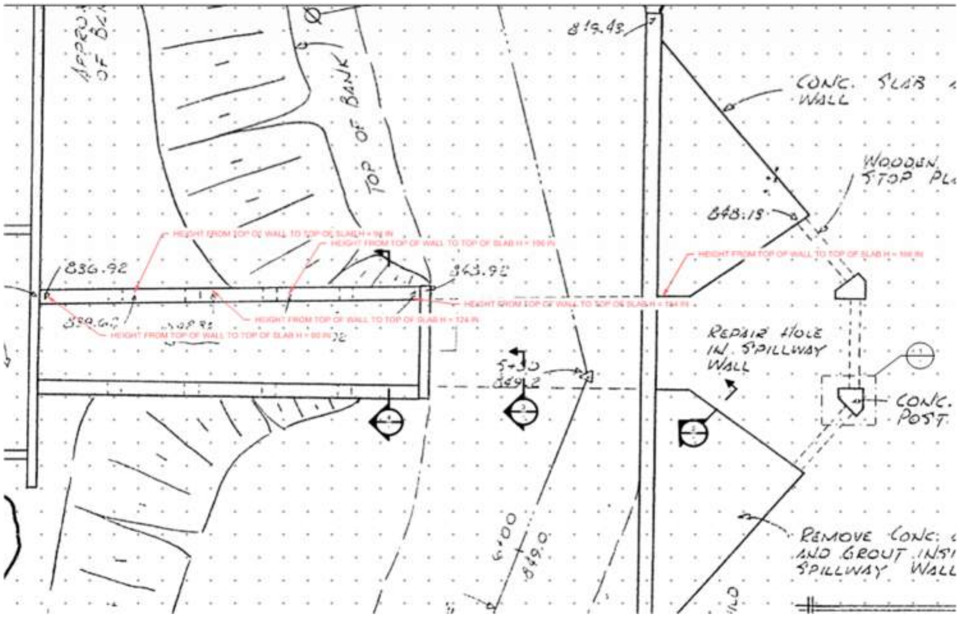

*Figure 16 – Chute Wall Heights t/slab to t/wall Steps*

## Pier and Strut

No reinforcement information is available for the intake piers, which support the intake gates/ stop logs. For this analysis the piers were assumed to have minimum rebar on the tension side only. Due to the more complicated stress distributions in the non - symmetrical extreme fibers of the pier the unusual geometry was simplified to the max rectangular section that can fit within the profile as shown in figure XX. The section with greatest elastic section modulus in the direction of loading was chosen.

The strut has been assumed to be sufficiently stiff as to act as a roller end support condition with a pinned base.

**Loads:**

The principal load considered for the intake piers is the hydrostatic load on the gates / stop logs.

### Hydrostatic:

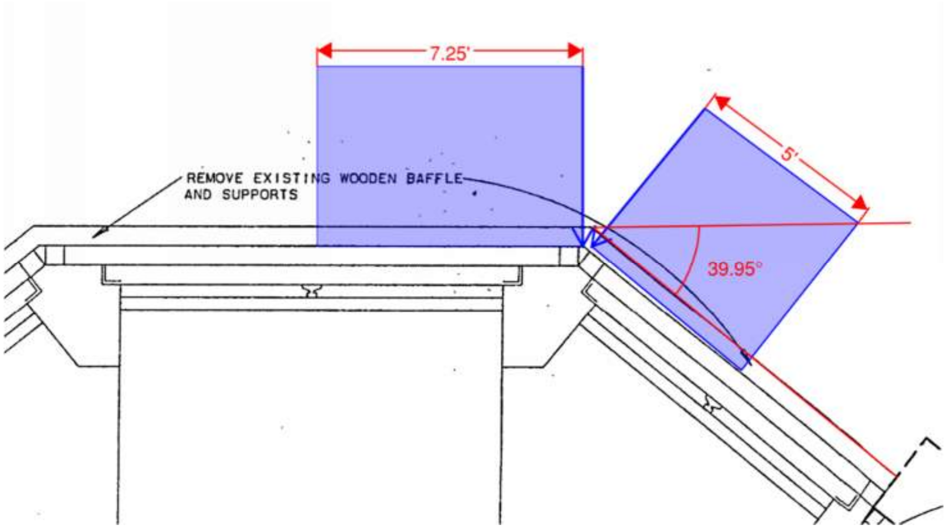

*Figure 17 – Hydrostatic Loading of Pier*

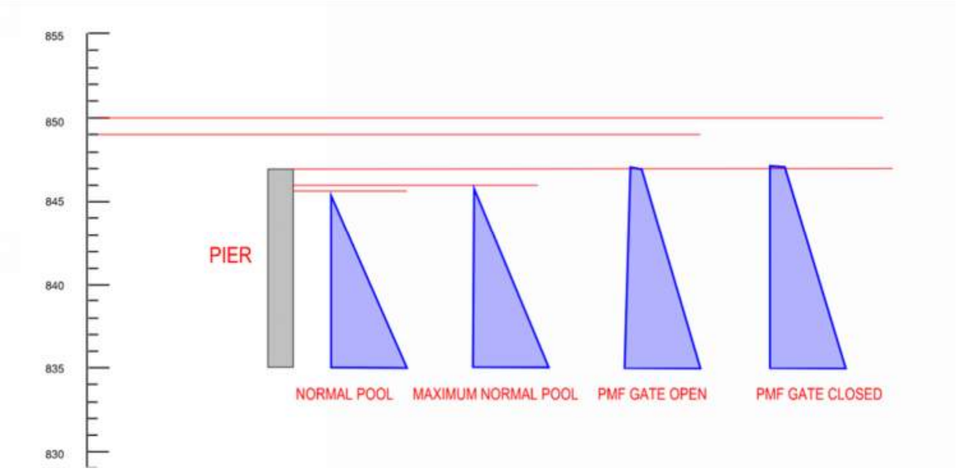

*Figure 18 – Hydrostatic Pressure Diagrams*

Determine Maximum Factored Hydrostatic Load – Primary Gate

Maximum Normal Pool:

Tributary Width: $T_{w1}$ = 14.75 ft /2 = **7.38** ft

Head Height Max Normal: $H_{w.MNP} = El_{mNP} - El_{bwall}$ = **11.00** ft

Force Water: $F_{w.MNP} = LF_{wnp} * 0.5 * \gamma_w * H_{w.MNP}^2$=**8.32** kip /ft

Driving Force Normal Pool: $D_{f.MNP} = (F_{w.MNP} * T_{w1})$=**61.35** kip

Normal Pool:

Head Height Max Normal: $H_{w \cdot NP} = El_{NP} - El_{bwall}$ = **10.50** ft

Force Water: $F_{w.NP} = LF_{wnp} * 0.5 * \gamma_w * H_{w \cdot NP}^2$ =**7.58** kip/ft

Driving Force Normal Pool: $D_{f.NP} = (F_{w.NP} * T_{w1})$=**55.90** kip

PMF Gate Open:

Head Height PMF Gate Open: $H_{w.PMFo} = El_{PMFo} - El_{bwall}$ = **13.96** ft

Force Water: $F_{w.PMFo} = LF_{wPMF} * 0.5 * \gamma_w * H_{w.PMFo}^2$ =**6.09** kip/ft

Driving Force PMF Gate Open: $D_{f.PMFo} = (F_{w.PMFo} * T_{w1}) = \mathbf{44.91}$ kip

PMF Gate Closed:

Head Height PMF Gate CLosed: $H_{w.PMFc} = El_{PMFc} – El_{bwall} = \mathbf{14.67}$ ft

Force Water: $F_{w.PMFc} = LF_{wPMF} * 0.5 * \gamma_w * H_{w.PMFc}^2 = \mathbf{6.73}$ kip/ft

Driving Force PMF Gate Closed: $D_{f.PMFc} = (F_{w.PMFc} * T_{w1}) = \mathbf{49.60}$ kip

Maximum Factored Driving Force For Primary Gate:

$P_{pier.g1} = \max(D_{f.PMFc}, D_{f.PMFo}, D_{f.NP}, D_{f.MNP}) = \mathbf{61.35}$ kip

Determine Maximum Factored Hydrostatic Load – South Gate

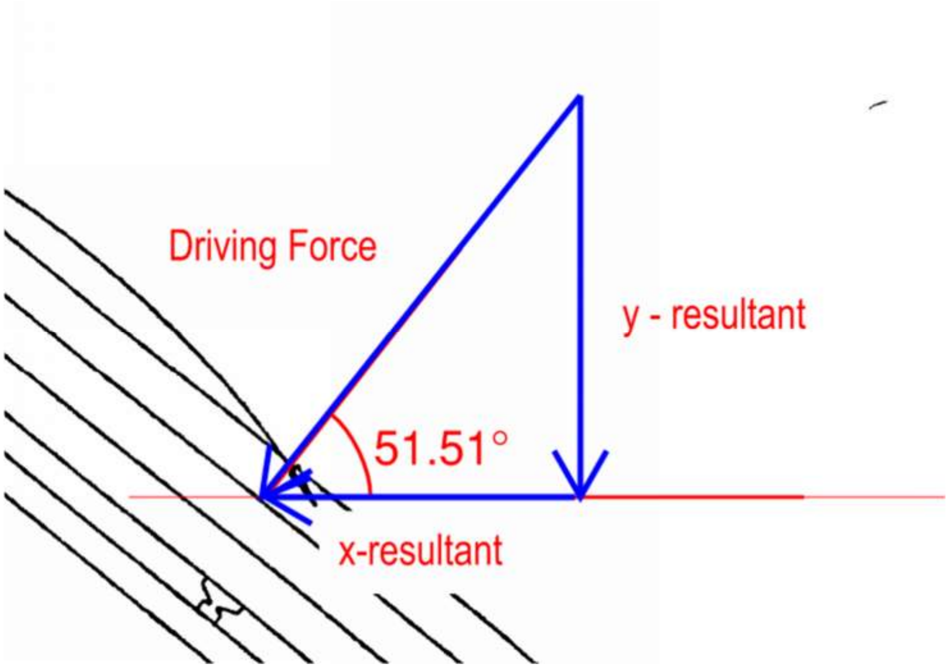

*Figure 19 – Resultant Forces Geometry*

Maximum Normal Pool:

Tributary Width: $T_{w1.s} = 10\text{ ft}/2 = \mathbf{5.00}$ ft

Head Height Max Normal: $H_{w.MNP} = El_{mNP} – El_{bwall} = \mathbf{11.00}$ ft

Force Water: $F_{w.MNP.s} = LF_{wnp} * 0.5 * \gamma_w * H_{w.MNP}^2 = \mathbf{8.32}$ kip /ft

Driving Force Normal Pool: $D_{f.MNP.s} = (F_{w.MNP.s} * T_{w1.s}) = \mathbf{41.59}$ kip

Driving force Y Direction: $D_{f.MNP.y} = D_{f.MNP.s} * \sin(51.51) =$ **32.56** kip

Driving Force X Direction: $D_{f.MNP.x} = D_{f.MNP.s} * \cos(51.51) =$ **25.89** kip

Normal Pool:

Head Height Max Normal: $H_{w\text{•}NP.s} = El_{NP} – El_{bwall} =$ **10.50** ft

Force Water: $F_{w.NP.s} = LF_{wnp} * 0.5 * \gamma_w * H_{w\text{•}NP.s}^2 =$ **7.58** kip/ft

Driving Force Normal Pool: $D_{f.NP.s} = (F_{w.NP.s} * T_{w1.s}) =$ **37.90** kip

Driving force Y Direction: $D_{f.NP.y} = D_{f.NP.s} * \sin(51.51) =$ **29.66** kip

Driving Force X Direction: $D_{f.NP.x} = D_{f.NP.s} * \cos(51.51) =$ **23.59** kip

PMF Gate Open:

Head Height PMF Gate Open: $H_{w.PMFo.s} = El_{PMFo} – El_{bwall} =$ **13.96** ft

Force Water: $F_{w.PMFo.s} = LF_{wPMF} * 0.5 * \gamma_w * H_{w.PMFo.s}^2 =$ **6.09** kip/ft

Driving Force PMF Gate Open: $D_{f.PMFo.s} = (F_{w.PMFo.s} * T_{w1.s}) =$ **30.45** kip

Driving force Y Direction: $D_{f.PMFo.y} = D_{f.PMFo.s} * \sin(51.51) =$ **23.83** kip

Driving Force X Direction: $D_{f.PMFo.x} = D_{f.PMFo.s} * \cos(51.51) =$ **18.95** kip

PMF Gate Closed:

Head Height PMF Gate CLosed: $H_{w.PMFc.s} = El_{PMFc} – El_{bwall} =$ **14.67** ft

Force Water: $F_{w.PMFc.s} = LF_{wPMF} * 0.5 * \gamma_w * H_{w.PMFc.s}^2 =$ **6.73** kip/ft

Driving Force PMF Gate Closed: $D_{f.PMFc.s} = (F_{w.PMFc.s} * T_{w1.s}) =$ **33.63** kip

Driving force Y Direction: $D_{f.PMFc.y} = D_{f.PMFc.s} * \sin(51.51) =$ **26.32** kip

Driving Force X Direction: $D_{f.PMFc.x} = D_{f.PMFc.s} * \cos(51.51) =$ **20.93** kip

Maximum Factored Driving Force For Primary Gate y - Direction:

$P_{pier.g2.y} = \max(D_{f.PMFc.y}, D_{f.PMFo.y}, D_{f.NP.y}, D_{f.MNP.y}) =$ **32.56** kip

Maximum Factored Driving Force For Primary Gate x - Direction:

$P_{pier.g2.x} = \max(D_{f.PMFc.x}, D_{f.PMFo.x}, D_{f.NP.x}, D_{f.MNP.x}) =$ **25.89** kip

Due to load factors the highest resultant forces come from the Maximum Normal Pool. The resultant loads act at a distance from the base of:

$x = H_{w.MNP} / 3 = \mathbf{3.67}$ ft

Maximum Force resultants:

Height of Pier: $H_{pier} = El_{twall} – El_{thick.slab} = \mathbf{9.80}$ ft

Maximum Moment Primary Gate y: $M_{max.pier.g1} = (P_{pier.g1} * x * (H_{pier} – x)) / H_{pier} = \mathbf{140.79}$ kip_ft

Maximum Moment South Gate - Y: $M_{max.pier.g2.y} = (P_{pier.g2.y} * x * (H_{pier} – x)) / H_{pier} = \mathbf{74.71}$ kip_ft

Maximum Moment South Gate – x: $M_{max.pier.g2.x} = (P_{pier.g2.x} * x * (H_{pier} – x)) / H_{pier} = \mathbf{59.41}$ kip_ft

Maximum total Moment Y: $M_{max.pier.total.y} = M_{max.pier.g1} + M_{max.pier.g2.y} = \mathbf{215.50}$ kip_ft

Maximum total Moment x: $M_{max.pier.total.x} = M_{max.pier.g2.x} = \mathbf{59.41}$ kip_ft

Maximum Shear Primary Gate - y: $V_{max.pier.g1} = (P_{pier.g1} * (H_{pier} – x)) / H_{pier} = \mathbf{38.40}$ kip

Maximum Shear South Gate - y: $V_{max.pier.g2.y} = (P_{pier.g2.y} * (H_{pier} – x)) / H_{pier} = \mathbf{20.38}$ kip

Maximum Shear South Gate - x: $V_{max.pier.g2.x} = (P_{pier.g2.x} * (H_{pier} – x)) / H_{pier} = \mathbf{16.20}$ kip

Maximum total Shear Y: $V_{max.pier.total} = V_{max.pier.g1} + V_{max.pier.g2.y} = \mathbf{58.77}$ kip

Maximum total Shear x: $V_{max.pier.total} = V_{max.pier.g2.x} = \mathbf{16.20}$ kip

Moment Arm

**Strength:**

Geometric Properties:

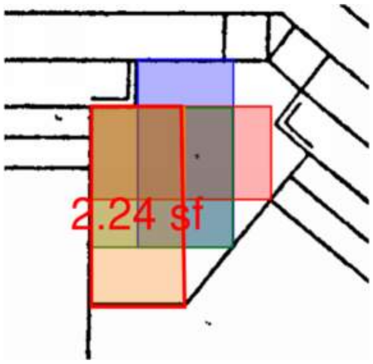

*Figure 20 – Rectangle 1*

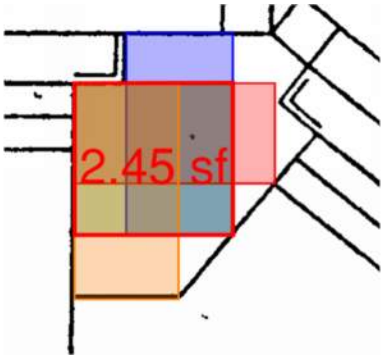

*Figure 21 – Rectangle 2*

*Figure 22 – Rectangle 3*

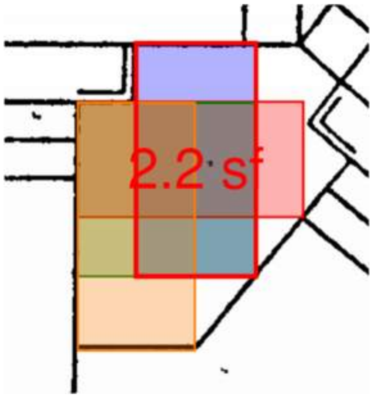

*Figure 23 – Rectangle 4*

Rectangle 1 :

Width: $b_{rect1}$ = 13.5 in

Depth: $d_{rect1} = 25.5$ in

Moment of Inertia Y-Y: $I_{yy.rect1} = (d_{rect1}^3 * b_{rect1}) / 12 =$ **18654.05** $in^4$

Moment of Inertia X-X: $I_{xx.rect1} = (b_{rect1}^3 * d_{rect1}) / 12 =$ **5228.30** $in^4$

Distance to Extreme Fiber Y-Y: $x_{rect1.y} = d_{rect1} / 2 =$ **12.75** in

Distance to Extreme Fiber X-X: $x_{rect1.x} = b_{rect1} / 2 =$ **6.75** in

Elastic Section Modulus Y-Y: $S_{yy.rect1} = I_{yy.rect1} / x_{rect1.y} =$ **1463.06** $in^3$

Elastic Section Modulus X-X: $S_{xx.rect1} = I_{yy.rect1} / x_{rect1.y} =$ **1463.06** $in^3$

Rectangle 2:

Width: $b_{rect2} = 19.5$ in

Depth: $d_{rect2} = 19.5$ in

Moment of Inertia Y-Y: $I_{yy.rect2} = (d_{rect2}^3 * b_{rect2}) / 12 =$ **12049.17** $in^4$

Moment of Inertia X-X: $I_{xx.rect2} = (b_{rect2}^3 * d_{rect2}) / 12 =$ **12049.17** $in^4$

Distance to Extreme Fiber Y-Y: $x_{rect2.y} = d_{rect2} / 2 =$ **9.75** in

Distance to Extreme Fiber X-X: $x_{rect2.x} = b_{rect2} / 2 =$ **9.75** in

Elastic Section Modulus Y-Y: $S_{yy.rect2} = I_{yy.rect2} / x_{rect2.y} =$ **1235.81** $in^3$

Elastic Section Modulus X-X: $S_{xx.rect2} = I_{yy.rect2} / x_{rect2.y} =$ **1235.81** $in^3$

Rectangle 3:

Width: $b_{rect3} = 24.25$ in

Depth: $d_{rect3} = 12.25$ in

Moment of Inertia Y-Y: $I_{yy.rect3} = (d_{rect3}^3 * b_{rect3}) / 12 =$ **3714.83** $in^4$

Moment of Inertia X-X: $I_{xx.rect3} = (b_{rect3}^3 * d_{rect3}) / 12 =$ **14557.61** $in^4$

Distance to Extreme Fiber Y-Y: $x_{rect3.y} = d_{rect3} / 2 =$ **6.13** in

Distance to Extreme Fiber X-X: $x_{rect3.x} = b_{rect3} / 2 =$ **12.13** in

Elastic Section Modulus Y-Y: $S_{yy.rect3} = I_{yy.rect3} / x_{rect3.y} =$ **606.50** $in^3$

Elastic Section Modulus X-X: $S_{xx.rect3} = I_{yy.rect3} / x_{rect3.y} =$ **606.50** $in^3$

Rectangle 4:

Width: $b_{rect4} = 12.25$ in

Depth: $d_{rect4} = 24.75$ in

Moment of Inertia Y-Y: $I_{yy.rect4} = (d_{rect4}^3 * b_{rect4}) / 12 =$ **15476.77** $in^4$

Moment of Inertia X-X: $I_{xx.rect4} = (b_{rect4}^3 * d_{rect4}) / 12 =$ **3791.42** $in^4$

Distance to Extreme Fiber Y-Y: $x_{rect4.y} = d_{rect4} / 2 =$ **12.38** in

Distance to Extreme Fiber X-X: $x_{rect4.x} = b_{rect4} / 2 =$ **6.13** in

Elastic Section Modulus Y-Y: $S_{yy.rect4} = I_{yy.rect4} / x_{rect4.y} =$ **1250.65** $in^3$

Elastic Section Modulus X-X: $S_{xx.rect4} = I_{yy.rect4} / x_{rect4.y} =$ **1250.65** $in^3$

Maximum Section Modulus Y-Y: $S_{yy.pier} = \max(S_{yy.rect4} , S_{yy.rect3} , S_{yy.rect2} , S_{yy.rect1}) =$ **1463.06** $in^3$

Maximum Section Modulus X-X: $S_{xx.pier} = \max(S_{xx.rect4} , S_{xx.rect3} , S_{xx.rect2} , S_{xx.rect1}) =$ **1463.06** $in^3$

## Material Properties:

Two (2) # 6 bar

Rebar Ø: $D_{bar} = 0.75$ in

Number of Bars: $n = 2$

Area of Rebar: $A_{bar} = \pi * (D_{bar} / 2)^2 =$ **0.44** $in^2$

Area of Rebar per foot: $A_s = A_{bar} * n =$ **0.88** $in^2$

Flexural Strength Reduction Factor: $\phi_{f.pier} = 0.65$ (REF # 2 Table 21.2.1)

Shear Strength Reduction Factor: $\phi_v = 0.75$ (REF # 2 Table 21.2.1)

Strain Limit: $\varepsilon_{ty} = 0.003$ (Ref #2 22.2.2.1)

## Flexural Capacity:

Rectangle 1 as the design section.

Unit Width of Concrete: $b = b_{rect1} =$ **13.50** in

Depth: $d_c = d_{rect1} =$ **25.50** in

Whitney stress block factor: $\beta_1 = 0.85$ (Ref # 2 Table 22.2.2.4.3)

Clear cover: $c_{lr} = 3$ in

Steel yield stress: $f_y = 40$ ksi (Assumed)

Concrete compressive strength: $f'c = 2.5$ ksi

Concrete compressive strength dimensionless $f'c.dl = 2500$

Diameter bar: $d_b = D_{bar}$

Depth to centroid tension reinforcemnet: $d = d_c - (c_{lr} + (d_b / 2)) =$ **22.12** in

Minimum area of steel a: $A_{smin} = ((3 * \sqrt{(f'c.dl)}\ psi) / f_y) * b * d =$ **1.12** $in^2$

Reinforcement ratio: $\rho = A_s / (d * b) =$ **0.0030**

Depth to neutral axis: $c = (\rho * f_y * d) / (0.85 * \beta_1 * f'c) =$ **1.45** in

Limit for Compressive stress block: $a = \beta_1 * c =$ **0.10** (Ref # 5 22.2.2.4.1)

Strain concrete: $\varepsilon_c = 0.003$ (Ref # 5 22.2.2.1)

Minimum Tensile strain: $\varepsilon_{t.min} = 0.005$

Calculated strain in tension reinforcement: $\varepsilon_t = \varepsilon_c * ((d / c) - 1) =$ **0.04**

Nominal Strength: $\phi M_{n.pier} = \phi_{f.pier} * (\rho * f_y * d^2 * b * (1 - 0.59 * \rho * (f_y / f'c))) =$ **41.17** kip_ft

Demand to Capacity Ratio: $U_{pier} = M_{max.pier.total.y} / \phi M_{n.pier} =$ **5.23**

Flexure_check = if ($\phi M_{n.pier} > M_{max.pier.total.y}$ ,"Pier is sufficient to resist bending forces","Pier is insufficient to resist bending forces")

Flexure_check = **"Pier is insufficient to resist bending forces"**

Due to the presence of compressive forces in the slab, an additional column bending check will be performed, to take advantage of some axial loading.

## Shear Capacity:

One Way Shear : $\lambda = 1$ ACI 318-19 – Table 19.2.41.

Dimensionless depth: $d_{dl} = 24.00$

Size effect factor: $\lambda_s = \sqrt{(2/(1+(d_{dl}/10)))} =$ **0.77**

Dimensionless Concrete Compressive strength: $f'c_{dl} = 2500$

Dimensioned Concrete compressive Strength: $f'c = 2500$ psi

Steel to Concrete Ratio: $\rho_w = (2 * A_s)/(b*d) = \mathbf{0.01}$

Nu/6Ag maximum: $N_{u.max} = 0.05 * f'c = \mathbf{0.13}$ ksi

Area Concrete: $A_g = b * d = \mathbf{298.69}\ in^2$

Shear Capacity: $\phi V_{c.pier} = \phi_v * 2 * \lambda * (\sqrt{(f'c_{dl})}\ psi) * A_g = \mathbf{22.40}$ kip

Demand to Capacity Ratio: $U_{shear.pier} = V_{max.pier.total} / \phi V_{c.pier} = \mathbf{0.72}$

Shear_check = if (min($\phi V_{c.pier}$ ) > $V_{max.pier.total}$ , “ Section is sufficient to resist shear forces”,”Section is insufficient to resist shear forces”)

Shear_check = **" Section is sufficient to resist shear forces"**

## Intake Walls

### Loads

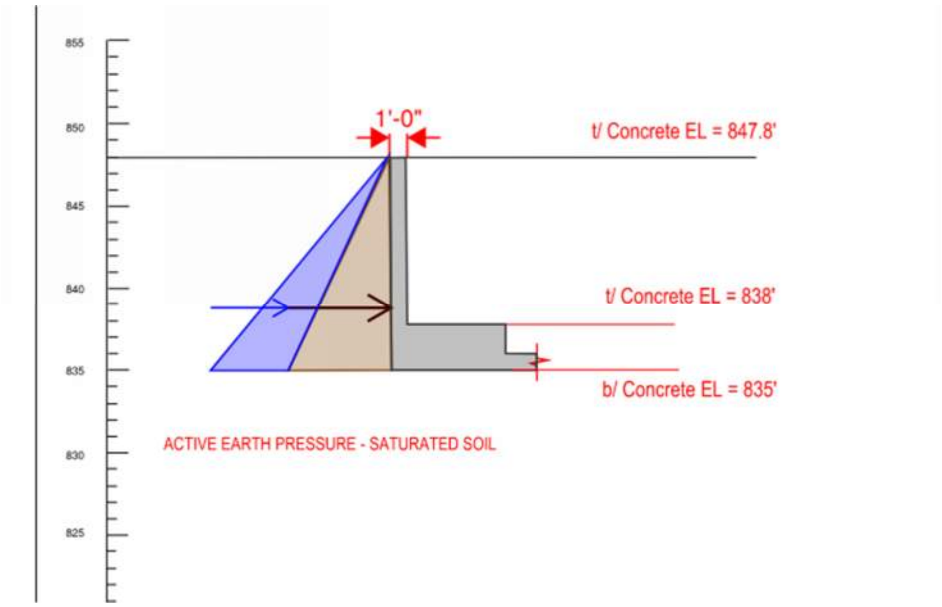

*Figure 24 – Active Earth Pressure*

### Active Soil Pressure (Rankine):

Assumed Water level is equal to top of soil for Intake Walls saturated .

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Unit weight of soil: $\gamma_{soil}$ = 125 pcf

Unit Weight of Water: $\gamma_{water}$ = 62.4 pcf

Submerged Soil Density: $\gamma_{sub} = \gamma_{soil} - \gamma_{water} = \mathbf{62.60}$ pcf

Height of retained soil : $H_{soil} = El_{twall} - El_{sillNS} = \mathbf{8.30}$ ft

Unit Width of Wall: $UW_{sc}$ = 1 ft

Friction angle: $\phi'$ = 30 deg

Vertical Soil Pressure: $p_v = \gamma_{sub} * H_{soil} = \mathbf{519.58}$ psf

Active pressure coefficient: $K_a = (1 - \sin(\phi')) / (1 + \sin(\phi')) = \mathbf{0.33}$

Total soil load: $P_{soil} = (K_a * p_v * H_{soil})/2 = \mathbf{0.72}$ kip/ft

Unit Width Load: $P_s = (P_{soil} * UW_{sc}) * LF_{lat.earth.d} = \mathbf{1.08}$ kip

Moment arm for soil: $L_{soil} = (1/3)*H_{soil} = \mathbf{2.77}$ ft

### Hydrostatic Pressure:

Water assumed to be full height of retained soil from top of wall to top of slab for design section 1

Height of Water: $H_{water} = H_{soil} = \mathbf{8.30}$ ft

Total Water Load: $P_{water} = (((H_{water} * \gamma_{water}) * UW_{sc} * H_{water}) /2) * LF_{wnp} = \mathbf{4.73}$ kip

### Resultant Forces @ Base of Wall:

Maximum Moment: $M_{max.iw} = (P_s * L_{soil}) + (P_{water} * L_{soil}) = \mathbf{16.07}$ kip_ft

Maximum Shear: $V_{max.iw} = P_s + P_{water} = \mathbf{5.81}$ kip

**Strength:**

### Geometric & Material Properties:

The GPR Performed by GEI indicates the bottom 48 in of the intake training wall do not possess reinforcement, typical of dam structures of this age.”

Strength Reduction Factor: $\phi_{f.p}$ = 0.6 (Ref #5 Table 21.2.1 Plain Concrete)

Dimensioned Concrete compressive Strength: f’c = 3000

Unit Width: UW = 12 in

Wall Thickness: $d_c$ = 12 in

Moment of Inertia of Unit Width Section: $I_{xx} = (d_c^3 * UW) / 12 =$ **1728.00** $in^4$

Distance from Neutral Axis to Extreme Fiber: $d_{ef}$ = 6 in

Elastic Section Modulus: $S_m = I_{xx} / d_{ef} =$**288.00** $in^3$

One Way Shear : $\lambda = 1$

Plain Concrete Flexural: $\phi M_{n.iw} = \phi_{f.p} * 5 * \lambda * (\sqrt{(f'c)}*1psi) *S_m =$ **3.94** kip_ft (Ref #5 14.5.2.1a)

Flexural_check = if($\phi M_{n.iw} > M_{max.iw}$, “ Section is sufficient to resist flexural forces” , ”The unreinforced plain concrete intake training wall section is insufficient to resist the flexural forces”) = **"The unreinforced plain concrete intake training wall section is insufficient to resist the flexural forces"**

Demand to Capacity Ratio: $U_{flexural.intake} = M_{max.iw} / \phi M_{n.iw} =$ **4.07**

**Shear Capacity:**

Area Concrete: $A_g = UW * d_c =$ **144.00** $in^2$

Nominal Shear Capacity: $V_{c.iw} = 4/3*\lambda*(\sqrt{(f'c_{dl})}\ psi) * A_g =$**9.60** kip (Ref #5 Table 14.5.5.1a)

Shear_check = if (min($V_{c.iw}$ ) > $V_{max.iw}$ , “ Section is sufficient to resist shear forces”,”Section is insufficient to resist shear forces”)

Shear_check = **" Section is sufficient to resist shear forces"**

Demand to Capacity Ratio: $U_{shear.intake} = V_{max.iw} / V_{c.iw} =$**0.60**

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## Chute Wall Max Span

The shortest Unsupported span in the chute wall occurs immediately downstream of the bridge crossing. The unsupported height in this location is 60” it was assumed that saturated soil conditions exist up to the top of the wall in this section.

**Loads:**

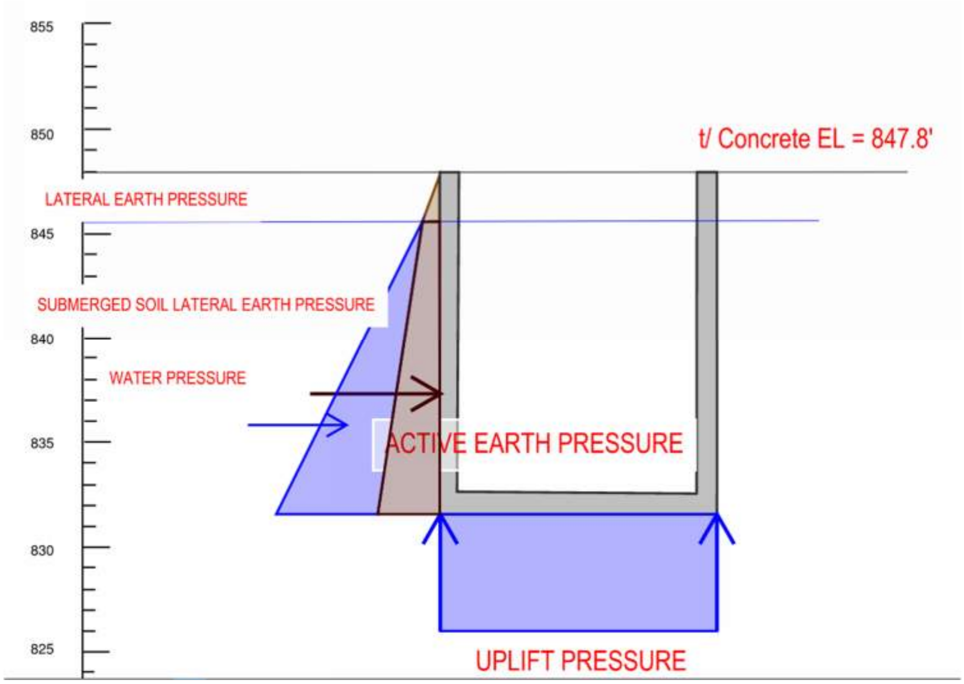

*Figure 25 – Active Earth Pressure Chute Wall – Minimum Span*

### Active Soil Pressure (Rankine):

Assumed Water level is equal to top of soil for Intake Walls saturated .

Unit weight of soil: $\gamma_{soil}$ = 125 pcf

Unit Weight of Water: $\gamma_{water}$ = 62.4 pcf

Submerged Soil Density: $\gamma_{sub} = \gamma_{soil} - \gamma_{water} = \mathbf{62.60}$ pcf

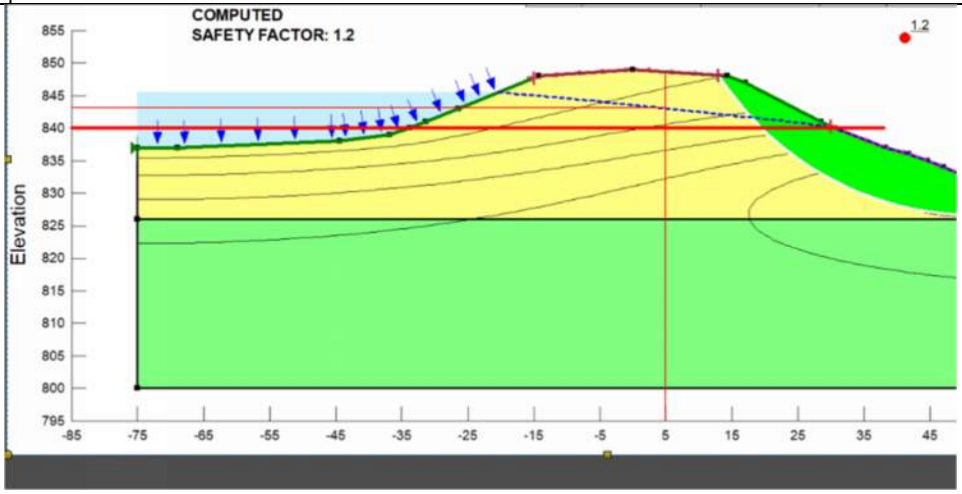

*Figure 26 – Water Table Estimate – Slope Stability Analysis*

Height of retained soil : $H_{soil.chute}$ = 184 in

Water Table Elevation: $El_{water.table}$ = 840 ft

Top of Slab Elevation: $El_{t.slab.chute} = El_{twall} - H_{soil.chute} = \mathbf{832.47}$ ft

Height of Submerged Soil: $H_{soil.chute.sub} = El_{water.table} - El_{t.slab.chute} = \mathbf{7.53}$ ft

Height of Moist Soil: $H_{soil.saturated} = El_{twall} - El_{water.table} = \mathbf{7.80}$ ft

Stratum 1 – Above Water Table

Unit Width of Wall: $UW_{sc}$ = 1 ft

Friction angle: $\phi'$ = 33 deg

Vertical Soil Pressure: $p_{v.s1} = \gamma_{soil} * H_{soil.saturated} = \mathbf{975.00}$ psf

Active pressure coefficient: $K_{a.s1} = (1 - \sin(\phi')) / (1 + \sin(\phi')) = \mathbf{0.29}$

Lateral Soil Pressure: $p_{l.s1} = p_{v.s1} * K_{a.s1} = \mathbf{287.43}$

Total soil load: $P_{soil.s1} = (p_{l.s1} * H_{soil.saturated})/2 = \mathbf{1.12}$ kip/ft

Driving Load (per ft width): $P_{s.s1} = P_{soil.s1} * UW_{sc} = \mathbf{1.12}$ kip

Stratum 2 – Below Water Table

Unit Width of Wall: $UW_{sc}$ = 1 ft

Friction angle: $\phi'$ = 33 deg

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Vertical Soil Pressure bottom of statum: $p_{v.s2} = (\gamma_{sub} * H_{soil.chute.sub}) + (K_{a.s1} * p_{v.s1}) =$ **759.02** psf

Active pressure coefficient: $K_{a.s2} = (1 - \sin(\phi')) / (1 + \sin(\phi')) =$ **0.29**

Lateral Soil Pressure: $p_{l.s2} = p_{v.s2} * K_{a.s2} =$ **223.76**

Total soil load: $P_{soil.s2} = (((p_{l.s2}) + (p_{l.s1})) * H_{soil.chute.sub}) / 2 =$ **1.93** kip/ft

Driving Load (per ft width): $P_{s.s2} = P_{soil.s2} * UW_{sc} =$ **1.93** kip

Moment arm for stratum 1: $X_{stratum.1} = (H_{soil.saturated} / 3) + H_{soil.chute.sub} =$ **10.13** ft

Moment arm for Stratum 2: $X_{stratum.2} = (H_{soil.chute.sub} * (2*p_{l.s2} + p_{l.s1})) / (3*(p_{l.s2} + p_{l.s1})) =$ **3.61** ft

## Hydrostatic Pressure:

Water assumed to be full height of retained soil from top of wall to top of slab for design section 1

Height of Water: $H_{water.chute} = H_{soil.chute.sub} =$ **7.53** ft

Total Water Load: $P_{water.chute} = ((H_{water.chute} * \gamma_{water}) * UW_{sc} * H_{water}) / 2 =$ **1.95** kip

Moment arm for Hydrostatic Pressure: $X_{hydro} = H_{soil.chute.sub} / 3 =$ **2.51** ft

## Resultant Forces @ Base of Wall:

Maximum Moment: $M_{max.cw} = (P_{water.chute} * X_{hydro}) + (P_{s.s1} * X_{stratum.1}) + (P_{s.s2} * X_{stratum.2}) =$ **23.21** kip_ft

Maximum Shear: $V_{max.cw} = P_{s.s1} + P_{s.s2} + P_{water.chute} =$ **5.00** kip

## Strength:

## Geometric & Material Properties:

The GPR Performed by GEI indicates the bottom 48 in of the intake training wall do not possess reinforcement, typical of dam structures of this age.”

Dimensioned Concrete compressive Strength: f’c = 3000

Unit Width: UW = 12 in

Wall Thickness: $d_c$ = 12 in

Moment of Inertia of Unit Width Section: $I_{xx} = (d_c^3 * UW) / 12 =$ **1728.00** in$^4$

Distance from Neutral Axis to Extreme Fiber: $d_{ef} = 6$ in

Elastic Section Modulus: $S_m = I_{xx} / d_{ef} =$**288.00** in$^3$

One Way Shear : $\lambda = 1$

Plain Concrete Flexural: $\phi M_{n.cwt} = \phi_{pc} * 5 * \lambda * (\sqrt{(f'c)}*1psi) * S_m =$ **3.94** kip_ft (ACI 318-19 – 14.5.2.1a)

Flexural_check = if($\phi M_{n.cwt} > M_{max.cw}$, “ Section is sufficient to resist flexural forces”,”The unreinforced plain concrete intake training wall section is insufficient to resist the flexural forces”) = **"The unreinforced plain concrete intake training wall section is insufficient to resist the flexural forces"**

Demand To Capacity ratio: $U_{flexural.cwt} = M_{max.cw} / \phi M_{n.cwt} =$**5.89**

**Shear Capacity:**

Strength reduction Factor: $\phi_v = 0.75$

Area Concrete: $A_g = UW * d_c =$ **144.00** in$^2$

Nominal Shear Capacity: $\phi V_{c.cwt} = \phi_v * 4/3 * \lambda * (\sqrt{(f'c_{dl})}$ psi$) * A_g =$**7.20** kip (ACI 318-19 Table 14.5.5.1a)

Shear_check = if (min($\phi V_{c.cwt}$ ) > $V_{max.cw}$ , “ Section is sufficient to resist shear forces”,”Section is insufficient to resist shear forces”)

Shear_check = **" Section is sufficient to resist shear forces"**

Demand to capcity Ratio: $U_{shear.cwt} = V_{max.cw} / \phi V_{c.cwt} =$ **0.69**

## Chute Wall Min Span

The tallest Unsupported span in the chute wall occurs immediately downstream of the bridge crossing. The unsupported height in this location is 184”

**Loads:**

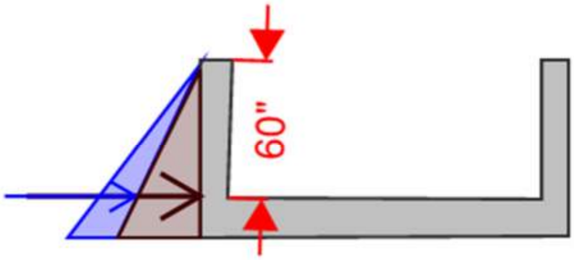

*Figure 27 – Active Earth Pressure Chute Wall*

## Active Soil Pressure (Rankine):

Unit weight of soil: $\gamma_{soil}$ = 125 pcf

Unit Weight of Water: $\gamma_{water}$ = 62.4 pcf

Submerged Soil Density: $\gamma_{sub} = \gamma_{soil} - \gamma_{water}$ = **62.60** pcf

Height of retained soil : $H_{soil}$ = 60 in = **5.00** ft

Unit Width of Wall: $UW_{sc}$ = 1 ft

Friction angle: $\phi'$ = 30 deg

Vertical Soil Pressure: $p_v = \gamma_{sub} * H_{soil}$ = **313.00** psf

Active pressure coefficient: $K_a = (1 - \sin(\phi')) / (1 + \sin(\phi'))$ = **0.33**

Total soil load: $P_{soil} = (K_a * p_v * H_{soil})/2$ = **0.26** kip/ft

Unit Width Load: $P_s = (P_{soil} * UW_{sc}) * LF_{lat.earth.d}$ = **0.39** kip

Moment arm for soil: $L_{soil} = (1/3)*H_{soil}$ = **1.67** ft

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## Hydrostatic Pressure:

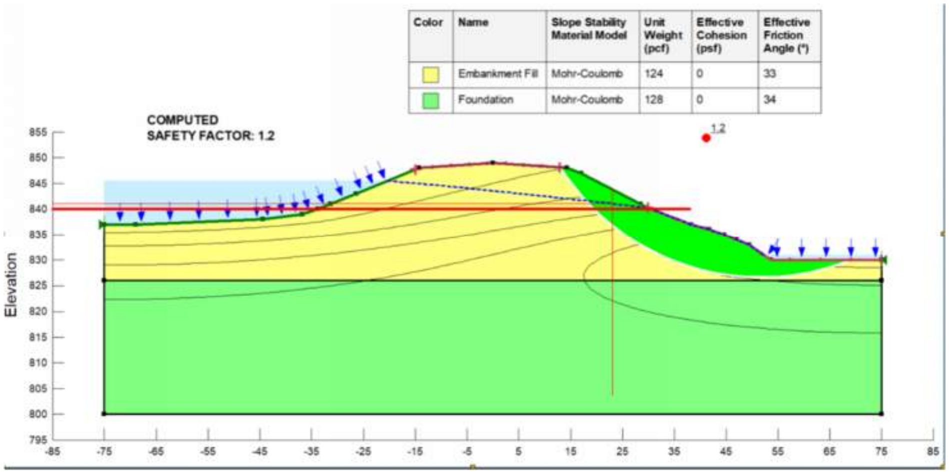

*Figure 28 – Location of Water Table*

Water assumed to be full height of retained soil from top of wall to top of slab for design section 1

Height of Water: $H_{water}$ = 60 in = **5.00** ft

Total Water Load: $P_{water} = (((H_{water} * \gamma_{water}) * UW_{sc} * H_{water}) /2) * LF_{wnp}$ = **1.72** kip

## Resultant Forces @ Base of Wall:

Maximum Moment: $M_{max.cws} = (P_s * L_{soil}) + (P_{water} * L_{soil})$ = **3.51** kip_ft

Maximum Shear: $V_{max.cws} = P_s + P_{water}$= **2.11** kip

**Strength:**

## Geometric & Material Properties:

The GPR Performed by GEI indicates the bottom 48 in of the intake training wall do not possess reinforcement, typical of dam structures of this age."

Dimensioned Concrete compressive Strength: f'c = 3000

Unit Width: UW = 12 in

Wall Thickness: $d_c$ = 12 in

Moment of Inertia of Unit Width Section: $I_{xx} = (d_c^3 * UW) / 12 =$ **1728.00** $in^4$

Distance from Neutral Axis to Extreme Fiber: $d_{ef} = 6$ in

Elastic Section Modulus: $S_m = I_{xx} / d_{ef} =$**288.00** $in^3$

### Flexural Capacity:

$\lambda = 1$

Plain Concrete Flexural: $\phi M_{n.cws} = \phi_{pc} * 5 * \lambda * (\sqrt{(f'c)}*1psi) * S_m =$ **3.94** kip_ft (ACI 318-19 – 14.5.2.1a)

Flexural_check_chutewall_short = if($\phi M_{n.cws} > M_{max.cws}$ , “Chute Wall is sufficient to resist flexural forces” , “Unreinforced Concrete chute wall is insufficient to resist concrete flexural forces” )= **"Chute Wall is sufficient to resist flexural forces"**

Demand to Capacity Ratio: $U_{flexural.cws} = M_{max.cws} / \phi M_{n.cws} =$ **0.89**

### Shear Capacity:

Area Concrete: $A_g = UW * d_c =$ **144.00** $in^2$

Nominal Shear Capacity: $V_{c.cws} = 4/3*\lambda*(\sqrt{(f'c_{dl})}\ psi) * A_g =$**9.60** kip (ACI 318-19 Table 14.5.5.1a)

Shear_check = if (min($V_{c.cws}$ ) > $V_{max.cws}$ , “ Section is sufficient to resist shear forces”,”Section is insufficient to resist shear forces”)

Shear_check = **" Section is sufficient to resist shear forces"**

Demand to Capacity Ratio: $U_{shear.cws} = V_{max.cws} / V_{c.cws} =$ **0.22**

## Slab Bottom

For the chute slab we analyzed the chute slab for a low flow condition which assumed a worst case condition with the highest water differential near the top of the chute. The slab will be analyzed as an idealized simply supported beam of unit width 1ft and loaded with the uplift pressure. The GPR in this section indicated some reinforcement at 12” OC near the bottom of the slab, during uplift the slab will experience load reversal and will perform as an unreinforced section. The exact size of the reinforcement could not be determined from these scans and is not indicated on existing drawings. For these calculations #4 bar was assumed with historic rebar yield stress of $f_y$ = 33 ksi and tensile stress $f_t$ = 55ksi which is typical of rebar in this period.

**Loads:**

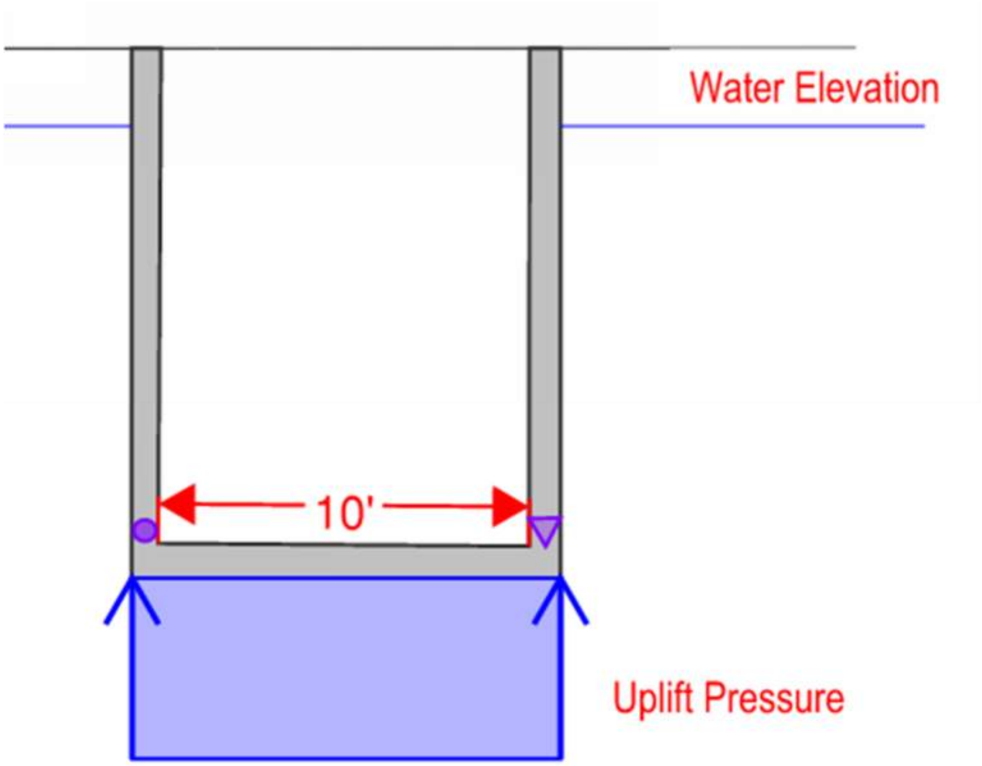

*Figure 29 – Uplift Pressure Diagram*

## Uplift Pressure:

Slab Span (One Way): $L_{slab.uplift}$ = 10 ft

Unit Width for Slab Checks: $UW_{slab}$ = 12 in

Maximum Normal Pool

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| | **Project** | White Cloud Dam Feasibility | | | **Pg. Rev.** | 0 |
| | **By** | J. Probstfeld | **Chk.** | M. Carden | **App.** | R. Price |
| | **Date** | 01/15/2024 | **Date** | 1/15/2025 | **Date** | 2/14/2025 |
| **Project No.** | 2302435 | **Document No.** | N/A | | | |
| **Subject** | White Cloud Inlet Structure – Stability Analysis | | | | | |

Water Differential Head: $H_{w.MNP} = El_{mNP} - El_{bwall} = \mathbf{11.00}$ ft

Flotation: $F_{float.MNP.1} = H_{w.MNP} * \gamma_w = \mathbf{0.69}$ kip/ft$^2$

Uplift Line Load: $w_{F.mnp.uplift} = LF_{wnp} * F_{float.MNP.1} * UW_{slab} = \mathbf{1.51}$ kip/ft

Maximum Moment: $M_{max.slab.mnp} = (w_{F.mnp.uplift} * L_{slab.uplift}^2) / 8 = \mathbf{18.91}$ kip_ft

Maximum Shear: $V_{max.slab.mnp} = (w_{F.mnp.uplift} * L_{slab.uplift}) / 2 = \mathbf{7.56}$ kip

Normal Pool

Water Differential Head: $H_{w.NP} = El_{NP} - El_{bwall} = \mathbf{10.50}$ ft

Flotation: $F_{float.NP.1} = H_{w.NP} * \gamma_w = \mathbf{0.66}$ kip/ft$^2$

Uplift Line Load: $w_{F.NP.uplift} = LF_{wnp} * F_{float.NP.1} * UW_{slab} = \mathbf{1.44}$ kip/ft

Maximum Moment: $M_{max.slab.np} = (w_{F.NP.uplift} * L_{slab.uplift}^2) / 8 = \mathbf{18.05}$ kip_ft

Maximum Shear: $V_{max.slab.np} = (w_{F.NP.uplift} * L_{slab.uplift}) / 2 = \mathbf{7.22}$ kip

PMF Gate Open

Water Differential Head: $H_{w.PMFo} = El_{PMFo} - El_{bwall} = \mathbf{13.96}$ ft

Flotation: $F_{float.PMFo.1} = H_{w.PMFo} * \gamma_w = \mathbf{0.87}$ kip/ft$^2$

Uplift Line Load: $w_{F.PMF.uplift} = LF_{wPMF} * F_{float.PMFo.1} * UW_{slab} = \mathbf{0.87}$ kip/ft

Maximum Moment: $M_{max.slab.PMFo} = (w_{F.PMF.uplift} * L_{slab.uplift}^2) / 8 = \mathbf{10.91}$ kip_ft

Maximum Shear: $V_{max.np.PMFo} = (w_{F.PMF.uplift} * L_{slab.uplift}) / 2 = \mathbf{4.36}$ kip

PMF Gate Closed

Water Differential Head: $H_{w.PMFc} = El_{PMFc} - El_{bwall} = \mathbf{14.67}$ ft

Flotation: $F_{float.PMFc.1} = H_{w.PMFc} * \gamma_w = \mathbf{0.92}$ kip/ft$^2$

Uplift Line Load: $w_{F.PMFc.uplift} = LF_{wPMF} * F_{float.PMFc.1} * UW_{slab} = \mathbf{0.92}$ kip/ft

Maximum Moment: $M_{max.slab.PMFc} = (w_{F.PMFc.uplift} * L_{slab.uplift}^2) / 8 = \mathbf{11.46}$ kip_ft

Maximum Shear: $V_{max.np.PMFc} = (w_{F.PMFc.uplift} * L_{slab.uplift}) / 2 = \mathbf{4.58}$ kip

Maximum Moment Uplift: $M_{max.uplift} = \max(M_{max.slab.PMFc}, M_{max.slab.PMFo}, M_{max.slab.np}, M_{max.slab.mnp}) = \mathbf{18.91}$ kip_ft

Maximum Shear: $V_{max.uplift}$ = max($V_{max.np.PMFc}$ , $V_{max.np.PMFo}$ , $V_{max.slab.np}$ , $V_{max.slab.mnp}$ ) = **7.56** kip

### Strength:

### Geometric & Material Properties:

The GPR Performed by GEI indicates the bottom 48 in of the intake training wall do not possess reinforcement, typical of dam structures of this age.”

Dimensioned Concrete compressive Strength:f’c = 5000

Unit Width: $UW_{slab.beam}$ = 12 in

Slab Thickness: $d_{c.slab}$ = 11 in

Moment of Inertia of Unit Width Section: $I_{xx.slab} = (d_{c.slab}^3 * UW_{slab.beam}) / 12$ = **1331.00** $in^4$

Distance from Neutral Axis to Extreme Fiber: $d_{ef.slab}$ = 5.5 in

Elastic Section Modulus: $S_{m.slab} = I_{xx.slab} / d_{ef.slab}$ =**242.00** $in^3$

### Flexural Capacity:

$\lambda = 1$

Strength Reduction Factor Plain Concrete: $\phi_{pc} = 0.6$

Plain Concrete Flexural: $M_{n.slab} = \phi_{pc} * 5 * \lambda * (\sqrt{(f'c)}*1psi) *S_{m.slab}$ = **4.28** kip_ft (ACI 318-19 – 14.5.2.1a)

Flexural_check = if($M_{n.slab} > M_{max.uplift}$, “ Section is sufficient to resist flexural forces”,”The unreinforced plain concrete slab section is insufficient to resist the flexural forces”) = **"The unreinforced plain concrete slab section is insufficient to resist the flexural forces"**

Demand to Capacity ratio: $U_{slab} = M_{max.uplift} / M_{n.slab}$ = **4.42**

### Shear Capacity:

Area Concrete: $A_{g.slab} = UW_{slab} * d_{c.slab}$ = **132.00** $in^2$

Nominal Shear Capacity: $V_{c.slab} = 4/3*\lambda*(\sqrt{(f'c_{dl})}\ psi) * A_{g.slab}$ =**8.80** kip (ACI 318-19 Table 14.5.5.1a)

| | | | | | | |
|---|---|---|---|---|---|---|
| GEI Consultants | **Client** | City of White Cloud | | | **Page** | 54 |
| | **Project** | White Cloud Dam Feasibility | | | **Pg. Rev.** | 0 |
| | **By** | J. Probstfeld | **Chk.** | M. Carden | **App.** | R. Price |
| | **Date** | 01/15/2024 | **Date** | 1/15/2025 | **Date** | 2/14/2025 |
| **Project No.** | 2302435 | **Document No.** | N/A | | | |
| **Subject** | White Cloud Inlet Structure – Stability Analysis | | | | | |

Shear_check = if (min($V_{c.slab}$ ) > $V_{max.uplift}$ , “ Section is sufficient to resist shear forces”,”Section is insufficient to resist shear forces”)

Shear_check = **" Section is sufficient to resist shear forces"**

Demand to Capacity ratio: $U_{slab.shear} = V_{max.uplift} / V_{c.slab} = \mathbf{0.86}$

# Appendix H – Detailed Cost Estimates

**OPINION OF PROBABLE COST - CONCEPTUAL DESIGN**

**Project:** White Cloud Disposition Study
**Client:** The City of White Cloud
**Dam/Scenario:** Dam Repair

**Project No.:** 2302435
**Date:** 2/14/2025
**Estimated by:** LH/MC
**Checked by:** JMM

**5% Assumed Annual Interest Rate**

| Item | Description | Estimated Cost | Years to Expenditure | Today's Dollars | Future dollars | Notes |
|---|---|---|---|---|---|---|
| **0.00** | **Maintain Dam Scenario** | | | | | |
| 0.00 | General/Engineering/Permitting/Construction Oversight | | | | | |
| 0.00a | Contractor Mobilization / Demobilization | $ 499,500 | 0 | $ 499,500 | $ - | Assumed 10% of other cost |
| | Includes: Bonds & Insurance, Permits, Project Management, | | | | | |
| | Temp Facilities, Project Survey/Layout, Indirect Costs | | | | | |
| 0.00b | Engineering and Permitting | $ 549,450 | 0 | $ 549,450 | $ - | Assumed 10% of cost |
| 0.00c | Engineering and Construction Observation | $ 549,450 | 0 | $ 549,450 | $ - | Assumed 10% of cost |
| 0.01 | Insufficient Spillway Capacity | | | | | |
| 0.01a | Increase Size of Auxiliary Spillway | $ 895,000 | 0 | $ 895,000 | | |
| 0.02 | Left Embankment | | | $ - | $ - | |
| 0.02a | Overlay RCC | $ 210,000 | 0 | $ 210,000 | $ - | 700 sq yd of RCC area |
| 0.03 | Primary Spillway | | | $ - | $ - | |
| 0.03a | Repair Deteriorated Concrete | $ 500,000 | 0 | $ 500,000 | $ - | Assumed 500 sq ft |
| 0.03b | Perform a 3D FEM based stability study | $ 10,000 | 0 | $ 10,000 | $ - | |
| 0.03c | Selective Demo to evaluate concrete thickness and reinforcement | $ 15,000 | | | | |
| 0.03d | Replace Bridge of Primary Spillway | $ 400,000 | 0 | $ 400,000 | $ - | |
| 0.04 | Right Embankment | | | $ - | $ - | |
| 0.04a | Relocate Boat Ramp | $ 155,000 | 0 | $ 155,000 | $ - | Assumed lump sum |
| 0.04b | Install Buttress with graded filter | $ 750,000 | 0 | $ 750,000 | $ - | |
| 0.04c | Install RipRap Downstream | $ 60,000 | 0 | $ 60,000 | $ - | 800 sq yd of rip rap area |
| 0.05 | Fish Passage | $ 2,000,000 | 0 | $ 2,000,000 | $ - | Estimated total construction cost for fish passage |
| | | | **Subtotal** | **$ 6,578,400** | **$ -** | |
| | | | **Contingency (30%)** | **$ 1,973,520** | **$ 1,974,000** | |
| | | | **Total Rehab Cost** | **$ 8,551,920** | **$ 8,552,000** | |
| **1.00** | **50-Year Life Cycle Regulatory Requirements - No Legislation Change** | | | | | |
| 1.01 | Inspections (3 year cycle) | $ - | 0 | | | EGLE currently provides inspections every 3 years. |
| 1.02 | Maintenance and Operations | $ 10,000 | 0 | $ 500,000.00 | $ 2,090,000 | Total cost of standard operation and maintenance |
| 1.03 | Inspections In Depth (every 10yrs) | | 10 | | $ - | |
| 1.04 | Licensing and Insurance Requirements (annual) | $ 629 | 0 | $ 31,450.00 | $ 130,000 | Based on current annual insurance premium cost for $3M coverage amount, |
| 1.05 | Increased Spillway Capacity (10yrs) | | 10 | $ - | | |
| 1.06 | Major rehabilitation/repairs | $ 1,500,000 | 50 | $ 1,500,000.00 | $ 17,200,000 | Assume substantial repairs every 50 years. End of 50-year life cycle. |
| | | | **Subtotal** | **$ 2,031,450** | **$ 19,430,000** | |
| | | | **Estimated 50-year Life Cycle Cost** | $ 8,609,850 | $ 26,000,000 | |
| | | | **Contingency (30%)** | $ 2,582,955 | $ 7,800,000 | |
| | | | **Total 50-year Life Cycle Cost** | **$ 11,192,805** | **$ 33,810,000** | |
| **2.00** | **50-Year Life Cycle Regulatory Requirements - Legislation change** | | | | | |
| 2.01 | Inspections (annual) | $ 10,000 | 0 | $ 500,000 | $ 2,090,000 | Assuming EGLE will no longer provide inspections |
| 1.02 | Maintenance and Operations | $ 10,000 | 0 | $ 500,000 | $ 2,090,000 | Total cost of standard operation and maintenance |
| 1.03 | Inspections In Depth (every 10yrs) | $ 100,000 | 10 | $ 500,000 | $ 2,710,000 | In depth inspection - Year 10, 20, 30, 40 & 50 |
| 1.04 | Licensing and Insurance Requirements (annual) | $ 10,000 | 0 | $ 500,000 | $ 2,090,000 | Estimated Cost - Based on current insurance premium and adjusted for additional coverage. |
| 1.05 | Increased Spillway Capacity (10yrs) | $ 2,500,000 | 10 | $ 2,500,000 | $ 4,070,000 | In 10 years, modify spillway to meet PMF/IDF flow rates. |
| 1.06 | Major rehabilitation/repairs | $ 1,500,000 | 50 | $ 1,500,000 | $ 17,200,000 | Assume substantial repairs every 50 years. End of 50-year life cycle. |
| | | | **Subtotal** | **$ 6,000,000** | **$ 30,260,000** | |
| | | | **Initial Construction Cost** | $ 12,578,400 | $ 36,840,000 | |
| | | | **Contingency (30%)** | $ 3,773,520 | $ 11,050,000 | |
| | | | **Total 50yr Life Cycle Cost** | **$ 16,351,920** | **$ 47,900,000** | |

Information presented on this sheet represents our opinion of probable costs in 2025 dollars. Unit and lump-sum prices are based on costs for similar projects, engineering judgment, and/or published cost data. Client administrative/engineering costs and regulatory fees not included. Actual bids and total project costs may vary based on contractor's perceived risk, site access, season, market conditions, etc. No warranties concerning the accuracy of costs presented herein are expressed or implied. Future dollars is calculated using an inflation rate of 5% per year over 50 years where applicable.

**OPINION OF PROBABLE COST - CONCEPTUAL DESIGN**

**Project: White Cloud Disposition Study**
**Client: The City of White Cloud**
**Dam/Scenario: Dam Repair**

**Project No.: 2302435**
**Date: 2/14/2025**
**Estimated by: LH/MC**
**Checked by: JMM**

**5% Assumed Annual Interest Rate**

| Item | Description | Estimated Cost | Years to Expenditure | Today's Dollars | Future dollars | Notes |
|---|---|---|---|---|---|---|
| **0.00** | **Maintain Dam Scenario** | | | | | |
| 0.00 | General/Engineering/Permitting/Construction Oversight | | | | | |
| 0.00a | Contractor Mobilization / Demobilization | $ 627,500 | 0 | $ 627,500 | $ - | Assumed 10% of other cost |
| | Includes: Bonds & Insurance, Permits, Project Management, | | | | | |
| | Temp Facilities, Project Survey/Layout, Indirect Costs | | | | | |
| 0.00b | Engineering and Permitting | $ 690,250 | 0 | $ 690,250 | $ - | Assumed 10% of cost |
| 0.00c | Engineering and Construction Observation | $ 690,250 | 0 | $ 690,250 | $ - | Assumed 10% of cost |
| 0.01 | Insufficient Spillway Capacity | | | | | |
| 0.01a | Increase Size of Primary Spillway | $ 2,100,000 | 0 | $ 2,100,000 | | |
| 0.02 | Left Embankment | | | $ - | $ - | |
| 0.02a | Overlay RCC | $ 210,000 | 0 | $ 210,000 | $ - | 700 sq yd of RCC area |
| 0.03 | Right Embankment | | | $ - | $ - | |
| 0.03a | Relocate Boat Ramp | $ 155,000 | 0 | $ 155,000 | $ - | Assumed lump sum |
| 0.03b | Install Buttress with graded filter | $ 750,000 | 0 | $ 750,000 | $ - | |
| 0.03c | Install RipRap Downstream | $ 60,000 | 0 | $ 60,000 | $ - | 800 sq yd of rip rap area |
| 0.03d | Install Steel Sheet Pile Seepate Cut Off | $ 1,000,000 | | $ 1,000,000 | | |
| 0.04 | Fish Passage | $ 2,000,000 | 0 | $ 2,000,000 | $ - | Estimated total construction cost for fish passage |
| | | | **Subtotal** | **$ 8,283,000** | **$ -** | |
| | | | **Contingency (30%)** | **$ 2,484,900** | **$ 2,480,000** | |
| | | | **Total Rehab Cost** | **$ 10,767,900** | **$ 10,770,000** | |
| **1.00** | **50-Year Life Cycle Regulatory Requirements - No Legislation Change** | | | | | |
| 1.01 | Inspections (3 year cycle) | $ - | 0 | | | EGLE currently provides inspections every 3 years. |
| 1.02 | Maintenance and Operations | $ 10,000 | 0 | $ 500,000.00 | $ 2,090,000 | Total cost of standard operation and maintenance |
| 1.03 | Inspections In Depth (every 10yrs) | | 10 | | $ - | |
| 1.04 | Licensing and Insurance Requirements (annual) | $ 629 | 0 | $ 31,450.00 | $ 130,000 | Based on current annual insurance premium cost for $3M coverage amount, |
| 1.05 | Increased Spillway Capacity (10yrs) | | 10 | $ - | | |
| 1.06 | Major rehabilitation/repairs | $ 1,500,000 | 50 | $ 1,500,000.00 | $ 17,200,000 | Assume substantial repairs every 50 years. End of 50-year life cycle. |
| | | | **Subtotal** | **$ 2,031,450** | **$ 19,430,000** | |
| | | | **Estimated 50-year Life Cycle Cost** | $ 10,314,450 | $ 27,710,000 | |
| | | | **Contingency (30%)** | $ 3,094,335 | $ 13,850,000 | |
| | | | **Total 50-year Life Cycle Cost** | **$ 13,408,785** | **$ 41,560,000** | |
| **2.00** | **50-Year Life Cycle Regulatory Requirements - Legislation change** | | | | | |
| 2.01 | Inspections (annual) | $ 10,000 | 0 | $ 500,000 | $ 2,090,000 | Assuming EGLE will no longer provide inspections |
| 1.02 | Maintenance and Operations | $ 10,000 | 0 | $ 500,000 | $ 2,090,000 | Total cost of standard operation and maintenance |
| 1.03 | Inspections In Depth (every 10yrs) | $ 100,000 | 10 | $ 500,000 | $ 2,710,000 | In depth inspection - Year 10, 20, 30, 40 & 50 |
| 1.04 | Licensing and Insurance Requirements (annual) | $ 10,000 | 0 | $ 500,000 | $ 2,090,000 | Estimated Cost - Based on current insurance premium and adjusted for additional coverage. |
| 1.05 | Increased Spillway Capacity (10yrs) | $ 2,500,000 | 10 | $ 2,500,000 | $ 4,070,000 | In 10 years, modify spillway to meet PMF/IDF flow rates. |
| 1.06 | Major rehabilitation/repairs | $ 1,500,000 | 50 | $ 1,500,000 | $ 17,200,000 | Assume substantial repairs every 50 years. End of 50-year life cycle. |
| | | | **Subtotal** | **$ 6,000,000** | **$ 30,260,000** | |
| | | | **Initial Construction Cost** | $ 14,283,000 | $ 38,550,000 | |
| | | | **Contingency (30%)** | $ 4,284,900 | $ 19,270,000 | |
| | | | **Total 50yr Life Cycle Cost** | **$ 18,567,900** | **$ 57,820,000** | |

Information presented on this sheet represents our opinion of probable costs in 2025 dollars. Unit and lump-sum prices are based on costs for similar projects, engineering judgment, and/or published cost data. Client administrative/engineering costs and regulatory fees not included. Actual bids and total project costs may vary based on contractor's perceived risk, site access, season, market conditions, etc. No warranties concerning the accuracy of costs presented herein are expressed or implied. Future dollars is calculated using an inflation rate of 5% per year over 50 years where applicable.

**OPINION OF PROBABLE COST - CONCEPTUAL DESIGN**

**Project: White Cloud Dam Feasibility Disposition Study**
**Client: The City of White Cloud**
**Dam Removal - Passive Restoration**

**Project No.:** **2302435**
**Date:** **2/14/2025**
**Estimated by:** **LH/JM**
**Checked by:** **JMM**

| Item | Description | Quantity | Units | Unit Price | Total Cost | Total Cost | Notes |
|---|---|---|---|---|---|---|---|
| **0.00** | **General Conditions** | | | | | | |
| 0.01 | Contractor Mobilization / Demobilization | 1 | LS | $ 619,000 | $ 619,000 | $ 619,000 | 10% of Other Costs |
| | Includes: Bonds & Insurance, Permits, Project Management, Temp | 1 | LS | $ - | $ - | $ - | |
| | Facilities, Project Survey/Layout, Indirect Costs | 1 | LS | $ - | $ - | $ - | |
| | | | | **Subtotal** | **$ 619,000** | **$ 619,000** | |
| **1.00** | **Water Management** | | | | | | |
| 1.01 | Erosion and Sediment Control | 1 | LS | $ 50,000 | $ 50,000 | $ 50,000 | |
| 1.02 | Temporary Access Roads, Facilities and Laydown Areas | 1 | LS | $ 150,000 | $ 150,000 | $ 150,000 | |
| 1.03 | Incremental Demolition and Construction Dewatering | 1 | LS | $ 345,000 | $ 345,000 | $ 350,000 | $15,000/day for 13 days + misc dewatering for restoration |
| 1.04 | Temporary Cofferdam | 590 | CY | $ 120 | $ 70,800 | $ 70,000 | |
| | | | | **Subtotal** | **$ 545,000** | **$ 550,000** | |
| **2.00** | **Dam Removal** | | | | | | |
| 2.01 | Dam Demolition | 13 | DAYS | $ 10,000 | $ 130,000 | $ 130,000 | |
| 2.02 | Excavation/Dam Embankment | 6,750 | CY | $ 35 | $ 236,250 | $ 240,000 | |
| 2.03 | Excavation for River Channel and Floodplain | 128,000 | CY | $ 20 | $ 2,560,000 | $ 2,560,000 | |
| 2.04 | Spanning Bridge | 1 | LS | $ 2,000,000 | $ 2,000,000 | $ 2,000,000 | |
| | | | | **Subtotal** | **$ 4,926,250** | **$ 4,930,000** | |
| **3.00** | **Stream Restoration** | | | | | | |
| 3.01 | Bank Restoration | 10,600 | LFT | $ 25 | $ 265,000 | $ 270,000 | |
| 3.02 | Riffles | 9 | EA | $ 50,000 | $ 450,000 | $ 450,000 | |
| 3.01 | Upland Restoration | - | ACRE | $ 4,000 | $ - | $ - | |
| 3.02 | Floodplain Restoration | - | ACRE | $ 7,000 | $ - | $ - | |
| | | | | **Subtotal** | **$ 715,000** | **$ 720,000** | |
| | | | | $ 3,235,200 | | | |
| | | | | ***1617600*** | **$ 6,186,250** | **$ 6,190,000** | |
| | | | | $ 4,852,800 | | | |
| | **Passive Restoration** | | | | | | |
| 4.00 | Contingency | | | 30% | **$ 1,855,875** | **$ 1,860,000** | Unknown Scope Items |
| 5.00 | Engineering Design and Permitting | | | 10% | **$ 618,625** | **$ 620,000** | Engineering Design and Permitting |
| 6.00 | Engineering and Construction Observation | | | 10% | **$ 618,625** | **$ 620,000** | Engineering and Construction Observation |
| | **Passive Restoration Total Estimated Cost** | | | | **$ 9,279,375** | **$ 9,280,000** | |

Information presented on this sheet represents our opinion of probable costs in 2025 dollars. Unit and lump-sum prices are based on costs for similar projects, engineering judgment, and/or published cost data. Client administrative/engineering costs and regulatory fees not included. Actual bids and total project costs may vary based on contractor's perceived risk, site access, season, market conditions, etc. No warranties concerning the accuracy of costs presented herein are expressed or implied. Future dollars is calculated using an inflation rate of 5% per year over 50 years where applicable.

**OPINION OF PROBABLE COST - CONCEPTUAL DESIGN**

**Project: White Cloud Dam Feasibility Disposition Study**
**Client: The City of White Cloud**
**Dam Removal - Active Restoration**

**Project No.: 2302435**
**Date: 2/14/2025**
**Estimated by: LH/JM**
**Checked by: JMM**

| Item | Description | Quantity | Units | Unit Price | Total Cost | Total Cost | Notes |
|---|---|---|---|---|---|---|---|
| **0.00** | **General Conditions** | | | | | | |
| 0.01 | Contractor Mobilization / Demobilization | 1 | LS | $ 769,000 | $ 769,000 | $ 769,000 | 10% of Other Costs |
| | Includes: Bonds & Insurance, Permits, Project Management, Temp | 1 | LS | $ - | $ - | $ - | |
| | Facilities, Project Survey/Layout, Indirect Costs | 1 | LS | $ - | $ - | $ - | |
| | | | | **Subtotal** | **$ 769,000** | **$ 769,000** | |
| **1.00** | **Water Management** | | | | | | |
| 1.01 | Erosion and Sediment Control | 1 | LS | $ 50,000 | $ 50,000 | $ 50,000 | |
| 1.02 | Temporary Access Roads, Facilities and Laydown Areas | 1 | LS | $ 150,000 | $ 150,000 | $ 150,000 | |
| 1.03 | Incremental Demolition and Construction Dewatering | 1 | LS | $ 345,000 | $ 345,000 | $ 350,000 | $15,000/day for 13 days + misc dewatering for restoration |
| 1.04 | Temporary Cofferdam | 590 | CY | $ 120 | $ 70,800 | $ 70,000 | |
| | | | | **Subtotal** | **$ 545,000** | **$ 550,000** | |
| **2.00** | **Dam Removal** | | | | | | |
| 2.01 | Dam Demolition | 13 | DAYS | $ 10,000 | $ 130,000 | $ 130,000 | |
| 2.02 | Excavation/Dam Embankment | 6,750 | CY | $ 35 | $ 236,250 | $ 240,000 | |
| 2.03 | Excavation for River Channel and Floodplain | 128,000 | CY | $ 20 | $ 2,560,000 | $ 2,560,000 | CAD Corridor |
| 2.04 | Spanning Bridge | 1 | LS | $ 2,000,000 | $ 2,000,000 | $ 2,000,000 | |
| | | | | **Subtotal** | **$ 4,926,250** | **$ 4,930,000** | |
| **3.00** | **Stream Restoration** | | | | | | |
| 3.01 | Bank Restoration | 10,600 | LFT | $ 150 | $ 1,590,000 | $ 1,590,000 | |
| 3.02 | Riffles | 9 | EA | $ 50,000 | $ 450,000 | $ 450,000 | |
| 3.01 | Upland Restoration | 32 | ACRE | $ 4,000 | $ 126,000 | $ 130,000 | |
| 3.02 | Floodplain Restoration | 8 | ACRE | $ 7,000 | $ 56,000 | $ 60,000 | |
| | | | | **Subtotal** | **$ 2,222,000** | **$ 2,220,000** | |
| | | | | $ 3,385,325 | | | |
| | | | | ***1692662.5*** | **$ 8,462,250** | **$ 8,460,000** | |
| | | | | $ 5,077,988 | | | |
| | **Passive Restoration** | | | | | | |
| 4.00 | Unknown Scope Items | | | 30% | **$ 2,538,675** | **$ 2,540,000** | Unknown Scope Items |
| 5.00 | Engineering Design and Permitting | | | 10% | **$ 846,225** | **$ 850,000** | Engineering Design and Permitting |
| 6.00 | Engineering and Construction Observation | | | 10% | **$ 846,225** | **$ 850,000** | Engineering and Construction Observation |
| | **Passive Restoration Total Estimated Cost** | | | | **$ 12,693,375** | **$ 12,690,000** | |

Information presented on this sheet represents our opinion of probable costs in 2025 dollars. Unit and lump-sum prices are based on costs for similar projects, engineering judgment, and/or published cost data. Client administrative/engineering costs and regulatory fees not included. Actual bids and total project costs may vary based on contractor's perceived risk, site access, season, market conditions, etc. No warranties concerning the accuracy of costs presented herein are expressed or implied. Future dollars is calculated using an inflation rate of 5% per year over 50 years where applicable.

# Appendix I – Real Estate Parcel Map

## REAL ESTATE PARCELS ADJACENT TO LAKE WHITE CLOUD IMPOUNDMENT

White Cloud Dam Feasibility Study

White Cloud, MI

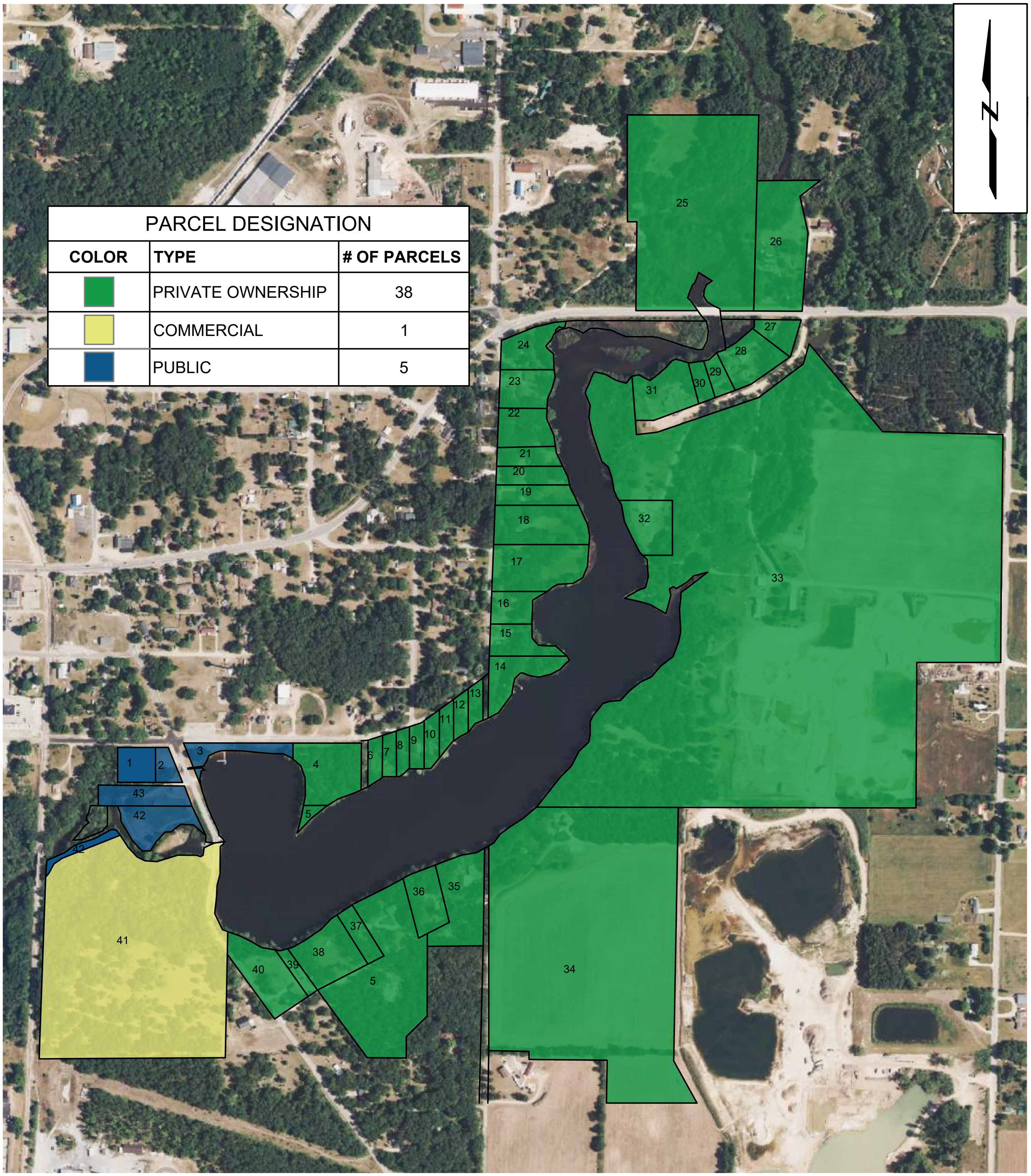
| PARCEL | TYPE OF PROPERTY | PARCEL | PIN | ACRES | LEGAL DESCRIPTION |
|---|---|---|---|---|---|
| 1 | P | 10 | 62-15-05-257-010 | 0.82 | LOTS 284 TO 287 INCL VILLAGE OF MORGAN |
| 2 | P | 11 | 62-15-05-257-011 | 1.53 | LOTS 283 & 283 1/2 ALSO LOTS 362 TO 369 1/2 INCL VILLAGE OF MORGAN |
| 3 | P | 3 | 62-15-05-200-003 | 3.86 | W 305.95 FT OF SE1/4 NE1/4 S OF JAMES ST & N OF LAKE WHITE CLOUD, ALSO THAT PT SW1/4 NE1/4 LYING E'LY OF STATE RD & S OF NEWELL ST & N OF LAKE WHITE CLOUD, EXC JAMES ST SEC 5, T13N - R12W CITY OF HITE CLOUD |
| 4 | R | 16 | 62-15-05-276-016 | 2.23 | COM ON S ROW JAMES ST AT PT 369.37 FT S & 305.95 FT N 89D 13'E OF NW COR SW 1/4 SE 1/4 NE 1/4 TH S 00D 09'W TO E & W 1/4 LINE E ALG SD 1/4 LINE TO SE COR SW 1/4 SE 1/4 NE 1/4 N 00D 17'E TO S ROW JAMES ST S 89D 13'W ALG SD ROW TO POB. SEC 5, T13N R12W CITY OF WHITE CLOUD |
| 5 | R | 52 | 62-15-05-400-052 | 6.85 | PT NE 1/4 SE 1/4 COM AT THE SE COR OF SEC N00D17'25"E ALG THE E LN OF SEC 26.20 FT TO CNTRLN OF SILVER AVE. AND TH N36D30'45"W ALG SD CNTRLN 1794.53 FT AND N53D26'42"E 273 FT AND N61D18'33"E 397.52 FT TO THE POB, TH N33D15'12"W 286.18 FT TO A MEANDER TRAVERSE LN, TH N60D13'28"E ALG SD TRAVERSE LN 288.94 FT, TH LEAVING SD TRAVERSE LN S15D26'13"E 298.70 FT, TH S61D18'33"W 197.64 FT TO BEG ALSO INCLUDING ALL LAND LYING BETWEEN SD MEANDER TRAVERSE LINE AND THE WATER'S EDGE ALSO NE 1/4 SE 1/4 LYING N OF SHORE OF LAKE WHITE CLOUD ALSO PT NE 1/4 SE 1/4 COM SE COR N00D17'25"E 1306.69 FT AND S88D13'41" W 400.27 FT POB, TH S88D13'41"W 201.01 FT, TH N36D30'45"W 426.80 FT, TH N61D18'33"E 636.57 FT, TH S00D19'02"W 439.34 FT, TH S44D17'27"W 144.02 FT, TH S00D19'02"W 100 FT TO BEG. SEC 5 T13N R12W |
| 6 | R | 1 | 62-15-05-278-001 | 0.4 | LOT 8 HOOKER'S ADDITION TO THE CITY OF WHITE CLOUD |
| 7 | R | 2 | 62-15-05-278-002 | 0.38 | LOT 7 HOOKER'S ADDITION TO THE CITY OF WHITE CLOUD |
| 8 | R | 3 | 62-15-05-278-003 | 0.4 | 13 LONGMEADOW VILLAGE DR, APT 112 |
| 9 | R | 4 | 62-15-05-278-004 | 0.41 | LOT 5 HOOKER'S ADDITION TO THE CITY OF WHITE CLOUD |
| 10 | R | 5 | 62-15-05-278-005 | 0.49 | LOT 4 HOOKERS ADD TO WHITE CLOUD |
| 11 | R | 6 | 62-15-05-278-006 | 0.44 | LOT 3 HOOKERS ADD TO CITY OF WHITE CLOUD |
| 12 | R | 7 | 62-15-05-278-007 | 0.4 | LOT 2 HOOKERS ADD TO WHITE CLOUD |
| 13 | R | 8 | 62-15-05-278-008 | 0.41 | LOT 1 HOOKERS ADD TO WHITE CLOUD |
| 14 | R | 15 | 62-15-04-100-015 | 1.54 | COM 1730 FT S OF NW COR SEC 4, TH S00D 18'W 335.63 FT TO SHORE OF LAKE, NE'LY ALG SHORE SD LAKE TO PT E OF BEG, S89D 26'W 167.64 FT TO BEG SEC 4, T13N - R12W CITY OF WHITE CLOUD |
| 15 | R | 14 | 62-15-04-100-014 | 1.1 | COM 1565 FT S OF NW COR SEC 4 TH S 00D 18' W 165 FT TH N 89D 26' E 167.64 FT TO SHORE OF LAKE TH N'LY ALG SHORE SD LAKE TO PT E OF BEG TH W 89D 26' E 159.41 FT TO BEG SEC. 4 T13N R12W CITY OF WHITE CLOUD |
| 16 | R | 13 | 62-15-04-100-013 | 0.88 | COM 1400 FT S OF NW COR SEC 4 TH S 00D 18' W 165 FT TH N 89D 26' E 159.41 FT TO SHORE OF LAKE TH NE'LY ALG SD LAKE TO PT E OF BEG TH S 89D 26' W 316 FT TO BEG SEC 4, T13N R12W CITY OF WHITE CLOUD |
| 17 | R | 30 | 62-15-04-100-030 | 2.66 | COM 1200 FT (ALSO REC'D AS 1199.75 FT) S NW COR SEC 4, TH N89D 26'E 460 FT M/L (ALSO REC'D AS N89D26'00"E 435 FT TO MEANDER TRAVERSE LN) TO SHORE OF LAKE WHITE CLOUD, S30D 24'W ALG SD SHORE 239.9 FT (LSO REC'D AS S30D06'30"W 232.51 FT), S89D 26'W 316 FT ( ALSO REC'D AS 320 FT), N00D 18' E 200 FT TO POB SEC 4, T13N - R12W CITY OF WHITE CLOUD |
| 18 | R | 29 | 62-15-04-100-029 | 2.29 | COM ON SEC LN 1000 FT S OF NW COR SEC 4, TH S 00D 18'W ALG SEC LN 200 FT, N 89D 26'E PARA WITH N SEC LN 460 FT M/L TO SHORE OF LAKE WHITE CLOUD,N 18D 55'W ALG SD SHORE 207.6 FT M/L TO A PT LYING N 89D 26'E OF BEG, S 89D 26'W PAR TO N SEC LN 410 FT M/L TO POB. SEC 4 T13N R12W CITY OF WHITE CLOUD |
| 19 | R | 8 | 62-15-04-100-008 | 0.99 | COM. 900 FT. S OF NW COR. SEC. S 100 FT. E TO WHITE RIVER N'LY ALG. SD. RIVER TO PT. DUE E OF BEG. W TO BEG. SEC. 4 T13N R12W CITY OF WHITE CLOUD |
| 20 | R | 7 | 62-15-04-100-007 | 0.88 | COM AT PT 800 FT S OF NW COR OF SD SEC, S 100 FT, E TO WHITE RIVER, N'LY ALG SD RIVER TO PT DUE E OF BEG, W TO POB. SEC 4 T13N R12W CITY OF WHITE CLOUD |
| 21 | R | 6 | 62-15-04-100-006 | 0.80 | COM 700 FT S OF NW COR S 100 FTE TO WHITE RIVER N'LY ALG SD RIVER TO PT OF BEG TH W TO BEG SEC 4 T13N R12W CITY OF WHITE CLOUD |
| 22 | R | 5 | 62-15-04-100-005 | 1.4 | COM AT PT 500 FT S OF NW COR OF SEC S 200 FT E TO WHITE RIVER N'LYALG SD RIVER TO PT DUE E OF BEG W TO BEG SEC 4 T13N R12W CITY OF WHITE CLOUD |
| 23 | R | 4 | 62-15-04-100-004 | 1.37 | COM 300 FT S OF NW COR, S 200 FT, E TO WHITE RIVER, N'LY ALG SD RIVER TO PT DUE E OF BEG, W TO POB SEC 4, T13N - R12W CITY OF WHITE CLOUD |
| 24 | R | 1 | 62-15-04-100-001 | 1.3 | COM AT NW SEC COR TH S 300 FT TH E TO WHITE RIVER TH N TO SEC LN TH W TO BEG SEC 4 T13N R12W CITY OF WHITE CLOUD |
| 25 | R | 6 | 62-11-33-300-006 | 19.9 | E1/2 SW1/4 SW1/4 EXC COM AT SW COR THEREOF TH N 89 DEG 24 MIN E ON SEC LN 50 FT N 00 MIN 26 E 466.7 FT S 89 DEG 24 MIN W 50 TO W LN E1/2 SW1/4 SW1/4 TH S 00 MIN 26 W 466.7 FT TO BEG SEC 33 T14N R12W 1<br>desc_02.46 A CITY OF WHITE CLOUD |
| 26 | R | 31 | 62-11-33-300-031 | 3.94 | SW1/4 SE1/4 SW1/4 EXC COM 847.16 FT N 90D W OF S1/4 COR, TH N 90D W 234.97 FT ALG S SEC LI TO C/L OF EXISTING DRIVE, N 04D 51'27"E ALG SD C/L 403.56 FT, N 12D 37'54"W 190.53 FT, N 50D 44'45"E 121.05 F, TH LEAVING SD C/L S 89D 36'08"E 345.55 FT, S 01D 01'09"W 662.33 FT TO S SEC LI, N 90D W TO POB. SEC 33, T14N R12W CITY OF WHITE CLOUD |
| 27 | R | 10 | 62-15-04-120-010 | 0.62 | LOT 1 FOX'S ADDITION CITY OF WHITE CLOUD |
| 28 | R | 15 | 62-15-04-120-015 | 1.45 | LOT'S 2 TO 4 INCL EXC S 120 FT OF W 15 FT OF LOT 4 FOX'S ADDITION TO CITY OF WHITE CLOUD |
| 29 | R | 14 | 62-15-04-120-014 | 0.45 | LOT 5 ALSO S 120 FT OF W 15 FT OF LOT 4 FOX'S ADDITION TO CITY OF WHITE CLOUD |
| 30 | R | 4 | 62-15-05-211-004 | 0.33 | LOT 6 FOX'S ADDITION |
| 31 | R | 9 | 62-15-04-120-009 | 1.79 | LOTS 7 TO 10 INC FOX'S ADDITION CITY OF WHITE CLOUD |
| 32 | R | 28 | 62-15-04-100-028 | 1.4 | PT NW1/4, DESC AS BEG AT A PT ON N1/8 LN, SD PT BEING 1279.06 FT S00D 53'51"W ALG N-S 1/4 LN AND 1724.38 FT S89D 31'15"W FROM N1/4 COR, TH S89D 31'15"W 150 FT TO MEANDER TRAVERSE LN, N22D 55'51"W ALG N 314.22 FT, N89D 31'15"E 270 FT, S00D 28'45"E 290.40 FT TO POB, INCLUDING ALL LAND LYG BTWN SD MEANDER TRAVERSE LN AND THREAD OF WHITE RIVER (NEWAYGO ENGINEERING & SURVEY CO, #22439, 07-19-01 L-385 P-1996) CITY OF WHITE CLOUD SEC 4, T13N - R12W 1.40A |

## REAL ESTATE PARCELS ADJACENT TO LAKE WHITE CLOUD IMPOUNDMENT

White Cloud Dam Feasibility Study
White Cloud, MI

| PARCEL | TYPE OF PROPERTY | PARCEL | PIN | ACRES | LEGAL DESCRIPTION |
|---|---|---|---|---|---|
| 33 | R | 27 | 62-15-04-100-027 | 86.01 | PT OF NW1/4 LYG E OF RIVER, EXC COM AT NE COR THEREOF, TH S00D 53'51"W 618.68 FT, S89D 43'08"W 658.87 FT, N41D 42'49"W 262.21 FT, N41D 04'05"W 332.78 FT TO E LN OF FOX'S ADD TO CITY OF WHITE CLOUD, N1 D 22'15"E 120 FT, N 50 FT TO NE COR SD FOX'S ADD, E TO BEG, ALSO EXC FOX'S ADD TO CITY OF WHITE CLOUD, ALSO EXC COM AT SE COR NW1/4, TH N ALG 1/4 LN 700 FT, W 431.85 FT, S 700 FT TO E-W 1/4 LN, E 432. 32 FT TO POB, ALSO EXC PCL DESC AS BEG AT A PT ON N1/8 LN, SD PT BEING 1279.06 FT S00D 53'51"W ALG N-S 1/4 LN AND 1724.38 FT S89D 31'15"W FROM N1/4 COR, TH S89D 31'15"W 150 FT TO MEANDER TRAVERSE LN, N22D 55'51"W ALG LN 314.22 FT, N89D 31'15"E 270 FT, S00D 28'45"E 290.40 FT TO POB CITY OF WHITE CLOUD SEC 4, T13N - R12W |
| 34 | R | 30 | 62-15-04-300-030 | 52.58 | PT SW 1/4 COM SW COR OF SEC 4 POB, TH N01D04'53"E 1306.69 FT, TH N01D06'30"E 1307.67 FT, TH N89D25'15"E 980.43 FT, TH S01D06'03"W 1308.36 FT, TH S27D00'43"E 689.93 FT, TH S00D28'38"E 690 FT, TH S89D31 22"W 274.77 FT, TH N00D28'38"W 208.71 FT, TH S89D31'22"W 208.71 FT, TH N00D28'38"W 322.24 FT, TH N89D01'51"W 507.39 FT, TH S01D05'52"W 543.96 FT, TH S89D31'22"W 318.50 FT TO BEG, EXC PT SW 1/4 COM AT SW COR OF SD SEC N01D04'53"E 934.01 FT, TH N89D09'15"E 35.02 FT TO POB, TH N01D04'53"E 414.60 FT, TH S88D55'10"E 208.71 FT, TH S01D04'53"W 31.98 FT, TH N89D09'15"E 371.17 FT, TH S01D04'53"W 375.60 FT, TH S89D09'15"W 580 FT TO BEG. SEC 4 T13N R12W 53.92 A M/L |
| 35 | R | 48 | 62-15-05-400-048 | 2.51 | PT NE 1/4 SE 1/4 COM AT SE COR OF SEC, TH N00D17'25"E ALG E LN OF SEC 1306.69 FT TO S 1/16 LN OF SEC, TH N00D19'02"E ALG SD E LN 582.17 FT TO THE POB, TH S88D13'14"W PARALLEL WITH THE S 1/16 LN OF SEC 300.20 FT TO THE W LN OF E 300 FT OF THE NE 1/4 SE /4, TH N00D19'02"E ALG SD W LN 57.17 FT, TH N61D18'33"E 143.30 FT, TH N15D26'13"W 270.13 FT TO A MEANDER TRAVERSE LN, TH N70D56'46"E ALG SD TRAVERSE LN 262.91 FT TO THE E LN OF SEC, TH LEAVING SD TRAVERSE LN S00D19'02"W ALG SD E LN 462.90 FT TO BEG ALSO INCLUDING ALL LAND LYING BETWEEN SD MEANDER TRAVERSE LINE AND THE WATER'S EDGE SEC 5 T13N R12W 2.24 A M/L |
| 36 | R | 8 | 62-15-05-400-008 | 1.29 | PART NE1/4 SE1/4 DESC AS COM AT SE COR SEC 5,TH N26.2 FT, N 36D 30'45" W ALG C/L STATE ST 1794.44 FT, N 53D 29'15"E 273 FT, N 61D 16'28"E 595.65 FT THIS BEING POB, TH N 61D 16'28"E 184.93 FT, N 15D 28 18"W 270.13 FT TO SHORE LAKE WHITE CLOUD, S 70D 08'10"W ALG SD SHORE 180.53 FT, S 15D 28'18"E 298.7 FT TO POB. SEC 5, T13N R12W |
| 5 | R | 52 | 62-15-05-400-052 | 6.85 | PT NE 1/4 SE 1/4 COM AT THE SE COR OF SEC N00D17'25"E ALG THE E LN OF SEC 26.20 FT TO CNTRLN OF SILVER AVE. AND TH N36D30'45"W ALG SD CNTRLN 1794.53 FT AND N53D26'42"E 273 FT AND N61D18'33"E 397.52 FT TO THE POB, TH N33D15'12"W 286.18 FT TO A MEANDER TRAVERSE LN, TH N60D13'28"E ALG SD TRAVERSE LN 288.94 FT, TH LEAVING SD TRAVERSE LN S15D26'13"E 298.70 FT, TH S61D18'33"W 197.64 FT TO BEG ALSO INCLUDING ALL LAND LYING BETWEEN SD MEANDER TRAVERSE LINE AND THE WATER'S EDGE ALSO NE 1/4 SE 1/4 LYING N OF SHORE OF LAKE WHITE CLOUD ALSO PT NE 1/4 SE 1/4 COM SE COR N00D17'25"E 1306.69 FT AND S88D13'41" W 400.27 FT POB, TH S88D13'41"W 201.01 FT, TH N36D30'45"W 426.80 FT, TH N61D18'33"E 636.57 FT, TH S00D19'02"W 439.34 FT, TH S44D17'27"W 144.02 FT, TH S00D19'02"W 100 FT TO BEG. SEC 5 T13N R12W |
| 37 | R | 7 | 62-15-05-400-007 | 0.7 | PART NE1/4 SE1/4 COM AT SE COR SEC 5, TH N 26.2 FT, N 36D 30' 45"W ALG CTR LINE STATE ST 1794.44 FT, N 53D 29'15"E 273 FT, N 61D 16'28"E 297.74 FT THIS BEING POB, TH N 61D 16'28"E 100 FT, N 33D 14'17" 286.18 FT TO SHORE OF LAKE WHITE CLOUD, S 57D 48' 15"W ALG SD SHORE 100.10 FT, S 33D 19'47"E 280.06 FT TO POB. SEC 5, T13N R12W |
| 38 | R | 46 | 62-15-05-400-046 | 1.81 | PT NE 1/4 SE 1/4 COM AT SE COR OF SEC N00D17'25"E ALG THE E LN OF SEC 26.20 FT TO CNTRLN OF SILVER AVE AND N36D30'45"W ALG SD CNTRLN 1794.53 FT AND N53D26'42"E 273 FT TO POB, TH N36D30'45"W 239 FT TO MEANDER TRAVERSE LN, TH N53D27'47"E ALG SD TRAVERSE LN 310.45 FT, LEAVING SD TRAVERSE LN S33D17'42"E 280.06 FT, TH S61D18'33"W 297.50 FT TO BEG ALSO INCLUDING ALL LAND LYING BETWEEN SD MEANDER TRAVERS E LINE AND THE WATER'S EDGE SEC 5 T13N R12W 1.81 A M/L |
| 39 | R | 6 | 62-15-05-400-006 | 0.4 | PART NE1/4 SE1/4 DESC AS COM AT SE COR SD SEC, TH N ON SEC LINE 26.2 FT, N 36D 30'45"W ON CTR LINE STATE ST 1794.44 FT, N 53D 29' 15" E 273 FT THIS BEING POB, TH N 36D 30'45"W 239 FT TO WATERS EDGE OF LAKE WHITE CLOUD, S 76D 55'W ALG WATERS EDGE 65.24 FT, S 36D 30'45"E 265 FT, N 53D 29'15"E 60 FT TO POB. SEC 5, T13N R12W |
| 40 | R | 22 | 62-15-05-400-022 | 1.97 | PART NE 1/4 SE 1/4 DESC AS COM N 00D 18'30"E 26.20 FT & N 36D 30' 45"W 1794.44 FT FROM SE SEC COR, TH N 36D 30'45"W 431.19 FT TO E 1/8 LINE, N 00D 07'47"E 72.90 FT TO MEANDER TRAV ON S SIDE OF LAKE WHITE CLOUD, S72D37'40"E ALG MEANDER LINE 287.56 FT. TH S36D 30'45"E 257.38 FT, S 53D 29'15"W 213 FT TO POB. SEC 5, T13N R12W |
| 41 | C | 4 | 62-15-05-400-004 | 23.03 | RIVER S'LY ALG SD RIVER 50 FT S 67D 30'W TO E ROW PMRR N TO PO |
| 42 | P | 2 | 62-15-05-400-002 | 2 | THAT PART NW 1/4 SE 1/4 LYING N'LY & E'LY OF WHITE RIVER & W'LY OF SILVER AVE, ALSO COM 259 FT S OF E & W 1/4 LINE ON E ROW PMRR TH N 67D 30'E 355 FT M/L TO WHITE RIVER S'LY ALG SD RIVER 50 FT S 67D 3 'W TO E ROWPMRR N TO POB. SEC 5, T13N R12W |
| 43 | P | 11 | 62-15-05-257-011 | 1.53 | LOTS 283 & 283 1/2 ALSO LOTS 362 TO 369 1/2 INCL VILLAGE OF MORGAN |

R = Residential, C= Commercial, P = {Public
Parcel details including ownership were obtained February 4th, 2024 from the Newago County Online GIS Map. (https://arcgisweb.countyofnewaygo.com/portal/apps/webappviewer/)



| PARCEL DESIGNATION | | |
|---|---|---|
| COLOR | TYPE | # OF PARCELS |
| 🟩 | PRIVATE OWNERSHIP | 38 |
| 🟨 | COMMERCIAL | 1 |
| 🟦 | PUBLIC | 5 |

***NOTES:***

1. PARCEL DETAILS INCLUDING OWNERSHIP WERE OBTAINED FEBRUARY 4TH, 2024 FROM THE NEWAGO COUNTY ONLINE GIS MAP. (HTTPS://ARCGISWEB.COUNTYOFNEWAYGO.COM/PORTAL/APPS/WEBAPPVIEWER/)

0 750' 1500'

SCALE, FEET

WHITE CLOUD DAM FEASIBILITY STUDY
THE CITY OF WHITE CLOUD
WHITE CLOUD, MI

GEI Consultants

PARCELS ADJACENT TO IMPOUNDMENT

Project 2302435

March 2024

Fig. #

HEJNAL, LAURA B:\Working\CITY OF WHITE CLOUD\2302435 White Cloud Dam Disposition Feasibility Study\00_CAD_XREF\DWG\Figures\Parcel Information.dwg - 2/20/2024

# Appendix J – Grant Funding Opportunities

| Organization | Program Topic/Name | Project Types Funded (Key Words) | Eligible Grantees | $/Grant | Match | Web Links | Geographical Boundaries/Limits | Phone |
|---|---|---|---|---|---|---|---|---|
| GLFT - Great Lakes Fishery Trust | Ecosystem Health and Sustainable Fish Populations: Habitat Protection and Restoration | preserve essential habitat; protect, restore, and stabilize important fish habitats; increase habitat availability | non-profit orgs, educational institutions, state, tribal and local governments | $500,000 for disbursement | | https://portal.glft.org/opportunities/84 | Great Lakes Basin | (517) 371-7468 |
| NFWF - National Fish and Wildlife Foundation | Sustain Our Great Lakes | improve and enhance: Stream and riparian habitat, coastal wetlands, and Great Lakes and tributaries water quality | non-profit orgs, educational institutions, state, tribal and local governments | $200,000 to $1,000,000. | 1:1 preferred | https://www.nfwf.org/programs/sustain-our-great-lakes-program | Great Lakes basin | 612-564-7284 |
| MDNR - Michigan Department of Natural Resources | Fisheries Habitat Grant Program | rehabilitate inland lakes, Great Lakes, rivers and streams habitat whose key physical processes that control aquatic habitat and fish production are impaired, including key processes : hydrology; connectivity; material recruitment and movement; geomorphology; and water quality. | non-profit orgs; local, state, federal and tribal government agencies | $25,000+ | minimum 10% | https://www.nfwf.org/programs/sustain-our-great-lakes-program/sustain-our-great-lakes-2023-request-proposals | State of Michigan | 517-284-5965 |
| Trout Unlimited | Embrace a Stream Program | coldwater fisheries conservation, on-the-ground restoration, protection, conservation that benefit trout and salmon fisheries | TU councils and chapters | $10,000 | 1:1 | http://www.tu.org/conservation/watershed-restoration-home-rivers-initiative/embrace-a-stream | Nationwide | (414) 588-4281 |
| EGLE - Michigan Department of Environment, Great Lakes, and Energy | Dam Risk Reduction Program | dam removal, critical maintenance | Entities that own or operate a dam in the state of Michigan | $350,000 for all projects | 10% | chrome-extension://efaidnbmnnnibpcajpcglclefindmkaj/https://www.michigan.gov/egle/- | Michigan | 989-370-1528 |
| USFWS - U.S. Fish and Wildlife Service | Midwest Region Fish Passage Program | dam and barrier removal | government, watershed groups, tribes, others | | | https://www.fws.gov/fisheries/fish-passage.html | Wisconsin, Ohio, Missouri, Minnesota, Michigan, Indiana, Illinois, and Iowa | |
| FEMA - Federal Emergency Management Agency | Rehabilitation Of High Hazard Potential Dam (HHPD) Grant Program | Classified as high hazard potential by the dam safety agency in the state or territory where the dam is located.<br>With a current, approved emergency action plan by the state or territorial dam safety agency<br>Failing to meet minimum dam safety standards of the state or territory and poses an unacceptable risk to the public | Eligible subrecipients under HHPD Grant Program are non-federal governmental organizations (other than the designated applicant) and nonprofit organizations. | Distributed by State | | https://www.fema.gov/emergency-managers/risk-management/dam-safety/rehabilitation-high-hazard-potential-dams | Nationwide | |