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Final Report: Prewitt Reservoir Rehabilitation Feasibility Study




Prepared for:

The Prewitt Operating Committee

112 N. 8th Ave
Sterling, CO 80751

COLORADO LICENSED
MARCUS JAMES KRALL
49306
1/23/19
PROFESSIONAL ENGINEER

In Association With:

Lytle Water Resources, LLC
640 Plaza Drive, Suite 170
Highlands Ranch, CO 80129
Phone: 303-350-4090

Hollingsworth Assoc. Inc
2875 W. Oxford Ave, Unit 7
Sheridan, CO 80110
Phone: 303-781-5188

WENCK
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Prepared by:

WENCK Associates, Inc.
7000 Yellowtail Rd, Ste 230
Cheyenne, WY 82009
Phone: 307-634-7848
Fax: 307-634-7849

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1.0 Project Background

This feasibility level study was completed by Wenck Associates, Inc. in conjunction with Lytle Water Resources, LLC and Hollingsworth Associates, Inc. Lytle Water Resources assisted with demonstrating project purpose and need and financial feasibility through execution of hydrologic models of reservoir operation. Hollingsworth Associates, Inc. provided geotechnical support during analysis of wave runup mitigation alternatives. Sediment core analysis was also completed by the National Lacustrine Core Facility (LacCore) through the University of Minnesota.

1.1 PROJECT SPONSOR

The Prewitt Reservoir Rehabilitation Feasibility Study Project (Project) is sponsored by the Prewitt Operating Committee (Committee). The Committee is a coalition of entities including the Logan Irrigation District, Iliff Irrigation District, and the Morgan Prewitt Reservoir Company. The manager of the Committee is Mr. Jim Yahn, PE. The organizational charter of the Committee is provided within Appendix A.

1.2 PROJECT PURPOSE AND NEED

The Project identified, developed, and analyzed feasible alternatives for rehabilitation of Prewitt Reservoir. Reservoir rehabilitation is required for two reasons:

1) The reservoir is currently unable to release and deliver its full permitted storage capacity due to dead storage created by heterogenous sedimentation within the pool. Approximately 1,604 ac-ft of storage are inaccessible because of this.
2) The reservoir is currently under a storage restriction due to the potential for wave overtopping of the embankment. Photos of a wave overtopping event are provided in **Figure 1-1.** Approximately 3,455 ac-ft of storage are inaccessible due to the storage restriction.

Figure 1-1: **Embankment Overtopping During Wind Event**

1.3 FACILITY STATISTICS

Basic facility statistics are provided in **Table 1-1**. Existing conditions of the facility are shown on **Drawing C-101** within **Appendix B**.

Table 1-1: Prewitt Reservoir Statistical Data

Item	Description
Location	Logan and Washington Counties, CO
Primary Use	Agricultural Irrigation
Maximum Normal Water Surface Elevation	4,098.4-ft (Gage Height 28-ft)
Restricted Storage Elevation	4,096.9-ft (Gage Height 26.5-ft)
Base Reservoir Elevation	4,070.4-ft (Gage Height 0.0-ft)
Decreed Capacity	32,300 ac-ft
Available Capacity	29,283 ac-ft
Current Active Capacity	24,224 ac-ft
Embankment	Homogenous Sand Fill with Concrete Slope Paving and 2-ft Parapet.
Outlet Works	Invert El=4,070.4-ft ~1,200-cfs capacity
Spillway	Unregulated Morning Glory with Invert El=4,098.4-ft

2.0 Water Rights

2.1 PURPOSE AND NEED

The Project may provide between 1,604 – 5,059 ac-ft of additional storage in Prewitt Reservoir from the current 24,224 ac-ft to 29,283 ac-ft capacity. There are two factors which currently impact storage capacity in Prewitt Reservoir. The first is a storage restriction by the State of Colorado, which reduces the available capacity by an estimated 3,455 ac-ft. The second is a large dead pool which inhibits 1,604 ac-ft from being delivered. Combined, these two factors result in 5,059 ac-ft of storage limitations from Prewitt Reservoir. Lytle Water Solutions, LLC (LWS) analyzed the increases in yield if the storage restriction is removed and/or the dead pool is reconnected to the outlet works.

2.2 MODEL APPROACH

To conduct this analysis, LWS first contacted Mr. Jim Yahn, manager of Prewitt Reservoir, to obtain Reservoir demand data in average, wet, and dry years. Mr. Yahn also provided LWS with stage data for the study period to allow for calibration of the baseline model. LWS has a point flow and reservoir model of the Lower South Platte River system, therefore, these data were input into the pre-existing point flow and reservoir model. The model was modified to simulate the diversion of native South Platte River flows, storage in Prewitt Reservoir under the current and future storage conditions after rehabilitation, and releases to meet irrigation demands.

The model is a daily point-flow model which spans water years 1999-2014. It includes 44 ditches from Denver to the Nebraska State line, including the Prewitt Inlet Canal, as well as the call data from the same 1999-2014 period. Using this model, LWS can estimate the native flow in the South Platte River which is available for diversion into Prewitt Reservoir. Since historic data are used in the model, the actual diversion records for this time period are modelled as being diverted into the Prewitt Inlet Canal. Accounting for only historic diversions would not allow for increased yield, so the point-flow model also diverts additionally-available water in the Prewitt Inlet Canal based on the flows in the South Platte River and the historic call record. To make the estimate more conservative and reduce the risk of triggering a call, the model only diverts up to a maximum of half of free river flows beyond the historic diversions during the irrigation season, and up to 75 percent of free river flows beyond historic diversions in the non-irrigation season. These additional diversions allow for testing of the increased yield from the additional storage gained by lifting of the restriction and/or reconnecting the dead pool to the outlet works.

These diversions were then simulated in the daily reservoir model as being stored in Prewitt Reservoir. This type of reservoir model has been in use at LWS for many years and was modified with Prewitt Reservoir specific data. An existing survey of the bottom surface of Prewitt Reservoir used for reservoir accounting was used to generate an elevation-area-capacity (EAC) curve, which relates the stage of the water in the reservoir to the storage volume and surface area of the reservoir. The EAC has been interpolated to a resolution of 0.1-ft increments for use in this model.

Additional factors in the model have also been modified to increase the accuracy of the predictions. The volume of water stored is used in combination with the EAC curve to give estimates of the depth of water and surface area of the reservoir. This additional information is used to increase the accuracy of factors such as seepage and evaporation. The seepage value is tied to the depth of the water in the reservoir because the seepage increases linearly from zero when there is no water, to its maximum value when the reservoir is at its maximum capacity. The evaporation is similarly estimated by multiplying the daily evaporation rate by the daily surface area of the reservoir, as estimated by the EAC curve. Conveyance losses also play a significant role in overall reservoir efficiency. The conveyance efficiency of the Prewitt Inlet Canal was estimated to be 90 percent based on conversations with Mr. Yahn, i.e. 10 percent ditch losses from the river headgate to the reservoir.

2.3 MODEL RESULTS

LWS focused on the increase in dry year yield to evaluate the effectiveness of these Prewitt Reservoir capacity increases. Since Prewitt Reservoir is used as a supplemental irrigation water supply source, the increase in yield in dry years is very important for maintaining crop yields when river flows diminish later in the irrigation season. Based on conversations with Mr. Yahn, an average annual demand of 12,000 ac-ft was simulated, with a dry-year demand of 42,000 ac-ft.

Initially, the model was run under the baseline condition, i.e., without any rehabilitation and the current storage limitations (24,224 ac-ft) so the increase in yield with the rehabilitation work could be assessed. For the baseline run, the estimated yield under average year conditions was 11,700 ac-ft, while the dry-year yield was 9,518 ac-ft (**Table 2-1**).

Three yield scenarios were then modelled, (1) storage restriction lifted but a remaining dead pool, (2) storage restriction in place but no dead pool, and (3) storage restriction lifted and no dead pool. The results of these simulations are summarized in **Table 2-1**. For the first scenario, it is estimated that increasing the reservoir capacity from solely lifting the storage restriction (3,455 ac-ft) will result in a dry-year yield increase of 1,001 ac-ft/yr. This is a 10.5 percent increase in the dry year yield of Prewitt Reservoir over the baseline condition, for a 14.3 percent increase in storage capacity.

For the second scenario, it is estimated that increasing the reservoir capacity from reconnecting the dead pool with the rest of the reservoir but not having the storage restriction lifted (1,604 ac-ft) will result in a dry-year yield increase of 476 ac-ft/yr (**Table 2-1**). This is a 5.0 percent increase in the dry-year yield of Prewitt Reservoir over the baseline condition, for a 6.6 percent increase in storage capacity.

For the third scenario, if the storage restriction is lifted and the dead pool is reconnected with the rest of the reservoir (5,059 ac-ft), it is estimated that this will increase the dry-year yield by 1,477 ac-ft/yr (**Table 2-1**). This is a 15.5 percent increase in the dry year yield of Prewitt Reservoir for a 20.9 percent increase in storage capacity. Average and wet year yield did not show consistent increases as the average and wet year demands are generally met using the current reservoir capacity.

As these model results show, the removal of the storage restriction and dredging the reservoir to eliminate the dead pool are both very efficient means to not only increase storage but increase the very important dry-year yield of the reservoir. The increase in yield is commensurate with the increase in storage, i.e., for the percent increase in storage there is a comparable increase in yield.

Table 2-1: Increase in Prewitt Reservoir Yield Based on Increase in Storage Capacity

	Inputs				Outputs				Increase Over Baseline						
Scenario	**Average Year Demand[1] (ac-ft)**	**Wet Year Demand (ac-ft)**	**Dry Year Demand (ac-ft)**	**Prewitt Capacity (ac-ft)**	**Average Year Yield (ac-ft)**	**Wet Year Yield (ac-ft)**	**Dry Year Yield (ac-ft)**	**Overall Average Yield (ac-ft)**	**Prewitt Capacity (ac-ft)**	**Average Year Yield (ac-ft)**	**Wet Year Yield (ac-ft)**	**Dry Year Yield (ac-ft)**	**Overall Averag e Yield (ac-ft)**	**Dry Year Yield (%)**	**Prewitt Capacity (%)**
Restricted, with Dead Pool[2]	12,000	9,000	42,000	24,224	11,700	9,000	9,518	10,723	-	-	-	-	-	-	-
No Restriction, with Dead Pool[3]	12,000	9,000	42,000	27,679	11,835	9,000	10,519	11,005	3,455	136	0	1,001	282	10.5%	14.3%
Restricted, No Dead Pool[3]	12,000	9,000	42,000	25,828	11,763	9,000	9,994	10,857	1,604	64	0	476	133	5.0%	6.6%
No Restriction, No Dead Pool[3]	12,000	9,000	42,000	29,283	11,895	9,000	10,995	11,136	5,059	195	0	1,477	413	15.5%	20.9%

1) Based on conversation with Prewitt Manager Jim Yahn and Prewitt Reservoir Accounting
2) Current condition of Prewitt Reservoir
3) Potential rehabilitation scenarios

3.0 Project Description

3.1 PROJECT ALTERNATIVES

The Project contemplates completion of both reservoir dredging and wave runup mitigation to maximize storage capacity in the Reservoir. Solutions to these issues differ, so they were analyzed separately below.

3.2 DREDGING ANALYSIS

As previously stated, dredging alternatives were analyzed to access dead storage within the existing pool. Reservoir bathymetry illustrating the existing dead pool is provided within **Drawing C-101** in **Appendix B**[1]. Dredging alternatives were evaluated for their technical merits including reliability, costs, robustness, multi-use potential and potential cooperation with wave runup mitigation efforts. Dredging analysis was essentially focused on answering three basic questions:

- How much material should be dredged?
- What should be done with it?
- What's the best way to dredge it?

3.2.1 Dredged Channel Dimensions

Channel dimensions will dictate the quantity of material to be dredged, so Wenck developed preliminary recommendations for channel geometry. The dredged channel should provide clear connectivity between the dead pool and outlet works. Channel grade shall be set by the base elevation of the dead pool (~4,076-ft) and the outlet gate invert (~4,070.4-ft). Dredge channel dimensions were established assuming near vertical side walls typical to hydraulic dredging operations. The minimum base width was set at 50-ft within sandy zones, and 100-ft within the softer clay zones near the outlet works. These widths should provide non-depositional flow velocities at the anticipated outlet works demand of 200-cfs. Non-depositional velocities were desired to prevent future channel blockage. The widths also provide a factor of safety against channel sloughing, which is anticipated with a box-cut section from hydraulic dredging.

A variable width channel design is recommended given the inferred affinity for sedimentation near the outlet works. The channel should widen as it approaches the outlet works to minimize the risk of blockage through additional sedimentation in that area. The proposed dredge channel alignment is shown within **Drawing C-102** within **Appendix B**. This minimum channel would generate about 162,500-CY of cut.

[1] Reservoir bathymetric survey completed by Others and presented here by Wenck without warranty as to its accuracy.

3.2.2 Sediment Analysis

Reservoir sediment samples were obtained from target locations using vibra-coring methods to characterize the material along the proposed dredging alignment.[2] Core locations are shown in **Drawing C-101** and **C-102** within **Appendix B**. Core depths varied from about 3.5 to 5.5-ft. Most of the material obtained was homogenized into bulk containers for geotechnical analysis while one core from each location was left intact within polycarbonate tubing and sent to LacCore for logging and initial core descriptions. High resolution photos of the cores and their initial core descriptions are provided within **Appendix C**. The material changed from highly plastic clays to non-plastic silty sands moving away from the outlet works between SP-1 and SP-2. Summary results from the sediment sampling program is provided in **Table 3-1** below:

Table 3-1: **Sediment Analysis**

Sample ID	USCS Description	% Passing No. 200 Sieve
SP-1	Clay (CH)	95%
SP-2	Silty Sand (SM)	27%
SP-3	Slightly Silty Sand (SP-SM)	11%

These results suggest that finer suspended solids entering the reservoir have an affinity for the outlet works. We believe the material transitions from fluvial clays to native sands moving away from the outlet structure. Any dredging program should anticipate this differentiation in the materials.

The initial core descriptions did not indicate the presence of contaminated sediments that would require special containment. Hydrogen sulfide odors were present at SP-1. Clearly precautions should be taken to limit human exposure to this dangerous gas, but conversations with dredging contractors indicate that this issue rarely impacts dredging projects. Rare occurrences of cyanobacterial pigments were potentially indicated within SP-1. No other contaminates were discovered. A photo of clayey material obtained from SP-1 is shown in **Figure 3-1** on the following page.

[2] Credit is due to both the Colorado Parks and Wildlife Service and LacCore for their assistance.

Figure 3-1: Clayey Sediment Core from SP-1

3.2.3 Dredging Alternatives

Various dredging alternatives were analyzed to determine the best means of removing and disposing of the materials.

3.2.3.1 Mechanical Dredging (Wet)

Wet Mechanical Dredging is the process by which an excavator or crane removes sediments while still storing water in the reservoir. Operations are conducted from shore or a barge within the reservoir. A clear benefit of this alternative is the facility Owner is allowed continued use of stored water during the dredging process. However, this alternative can cause increased turbidity levels which may have environmental permitting impacts. Clamshell buckets, or specially sealed environmental clamshell buckets, are often used to reduce turbidity, however these may impact disposal costs through increased dredged water contents. Removed sediments can vary from a fluid like slurry to a semi-plastic consistency. We anticipate a fluid like consistency for materials removed near the outlet works, becoming progressively more plastic moving towards the dead pool.

Wet Mechanical Dredging from Shore

Wet mechanical dredging from shore requires a long reach excavator or drag line. The material is placed into haul trucks for disposal. Enough staging area for these machines is required along the shorelines. Neither cranes nor excavators operating from shore would be able to reach the full dredge channel alignment, therefore this sub-alternative was removed from consideration.

Wet Mechanical Dredging from Reservoir

An excavator or crane is staged on a barge for this sub-alternative. The machine places excavated sediments onto another barge which ferries them to the disposal location. This incurs double handling of materials during unloading operations which increases costs. The draft of the loaded barge should be carefully considered for shoreline access due to Prewitt's relatively flat bathymetric shape. Machines should be able to unload the barges from the embankment or shoreline. This sub-alternative is considered feasible for the Project, although the fluvial sediments near the outlet works would be difficult to remove using this method.

3.2.3.2 Hydraulic (Cutterhead) Dredging

The cutterhead hydraulic dredge is the most widely used dredge in the United States, although the use of portable dredges for projects of this size and scale are less common. Hydraulic dredging is like a floating vacuum sucking sediment from the reservoir floor and pumping them to the disposal location. The hydraulic dredge consists of a floating barge with a pump house and forward cutterhead that loosens the material before sucking it into the delivery pipeline[3]. The delivery pipeline, often fused HDPE, may be submerged or float on pontoons on the water surface. The hydraulic slurry mobilizes the fines within the dredged material, so a segregation occurs as the material is disposed, with coarse grained materials settling first, near the outlet, and finer grained materials settling more slowly through time. Booster stations can be added to increase pumping rates and distances. Typical dredge pumping distances are up to 10,000-ft. A cutterhead dredge is shown in **Figure 3-2** below:

Figure 3-2: Cutterhead Hydraulic Dredge

Cutterhead hydraulic dredges work best with loosely consolidated fine-grained materials like those anticipated near the outlet works but can operate in the sandy soils anticipated elsewhere. This sub-alternative is considered feasible for the Project.

[3] Hopper and side casting type hydraulic dredges also exist but were excluded from analysis here as the former is a seagoing vessel and the latter would directly dispose the sediment back into the reservoir.

3.2.3.3 Mechanical Dredging (Dry)

This sub-alternative would simply be the excavation and removal of sediments after the reservoir has been drained. Typical equipment would include scrapers, dozers, excavators, loaders, and haul trucks. A challenge expected for this sub-alternative would be adequate dewatering of the reservoir base to allow equipment operations. Equipment can easily become bogged down or even buried within the fine, soft, sediments anticipated near the outlet works. Project sequencing must be carefully considered to allow enough time for dewatering the sediments. Winter conditions are considered desirable to provide a layer for supportive frost over the material.

An advantage of this sub-alternative would be a greater level of control over the material that is excavated, thus making it more reliable. Excavation dimensions could be easily verified, and materials sorted. This provides greater potential for cooperation with some of the wave runup mitigation alternatives discussed below.

This approach would create risks for water dependent users, as construction delays could disrupt fill cycles and water deliveries. This sub-alternative may also have permitting constraints due to the destruction of the fishery that would occur. Colorado Parks and Wildlife has leased recreational rights on the water's surface, however there is no mandated conservation pool or level. Therefore, this sub-alternative is considered feasible.

3.2.4 Sediment Disposal Alternatives

Dredged materials must be disposed of. Methods of disposal considered include:

- Sediment dewatering facilities
  - Confined disposal sites that allow for long term reclamation of the materials
- Reservoir placement
  - Direct placement of the material into another part of the reservoir
- Habitat enhancement sites
  - Habitat enhancement sites seek to obtain a beneficial ecological use from the dredged materials
- Project specific reuse
  - Project specific reuse opportunities for beaching or soil cement were considered

3.2.4.1 Sediment Dewatering Facilities

Sediment dewatering facilities consist of confined impoundments formed by berms or other means. Sediment dewatering may be required for hydraulically dredged materials and for the fine-grained materials anticipated near the outlet works. The slurry like sediment is placed within the impoundments which act as evaporation ponds removing the moisture. Geotextile dewatering tubes or mechanical separators such as belt presses can be used if adequate spacing is not available for the dewatering facility.

Mechanically removed coarse-grained sediments may not require dewatering facilities. These materials could potentially be placed and mounded at grade after stripping topsoil. Disposal areas should be at upland sites free of wetlands or other sensitive/protected plant or animal species and should be reclaimed after completion. Stormwater protection plans should be carefully crafted to avoid impacts to adjacent waterways from potential turbid run-off from these areas.

3.2.4.2 Reservoir Placement

Placement of fill within the reservoir may be permitted as part of the dam maintenance operations. Conversations with a dredging contractor indicate that there is precedence for this type of placement within Colorado. For hydraulic dredging, this simply includes directing the discharge pipe to a deep-water location. For mechanical dredging, it would simply be dumping/placing the material in a concentrated location within the reservoir. This could be the most cost-effective alternative but would require the appropriate permitting approvals. The permitting process for this could be lengthy, difficult, and costly.

3.2.4.3 Habitat Enhancement Sites

Habitat enhancement sites seek to create stable and biologically productive plant and animal habitats from dredged material. They include wetland, upland, aquatic and island habitat sites. Wetland and island habitat development were considered more valuable due to their relative scarcity to the area. Wetlands include a broad category of periodically inundated communities that survive in wet (hydric) soils (Corps, Dredging). They often support diverse plant and wildlife communities and are considered a highly valuable ecological resource.

Island habitats are defined as upland and or high zone wetland habitats distinguished by their isolation and uses and are surrounded by water or wetlands (Corps, Dredging). An advantage of islands is that they can develop or contain many types of attractive habitats. Aquatic habitats can develop around the permanently inundated portions of the structure, fringe wetlands can develop around island perimeter, and upland habitat, particularly avian nesting habitat, can develop on the island itself. This broad biodiversity is illustrated within **Figure 3-3:**

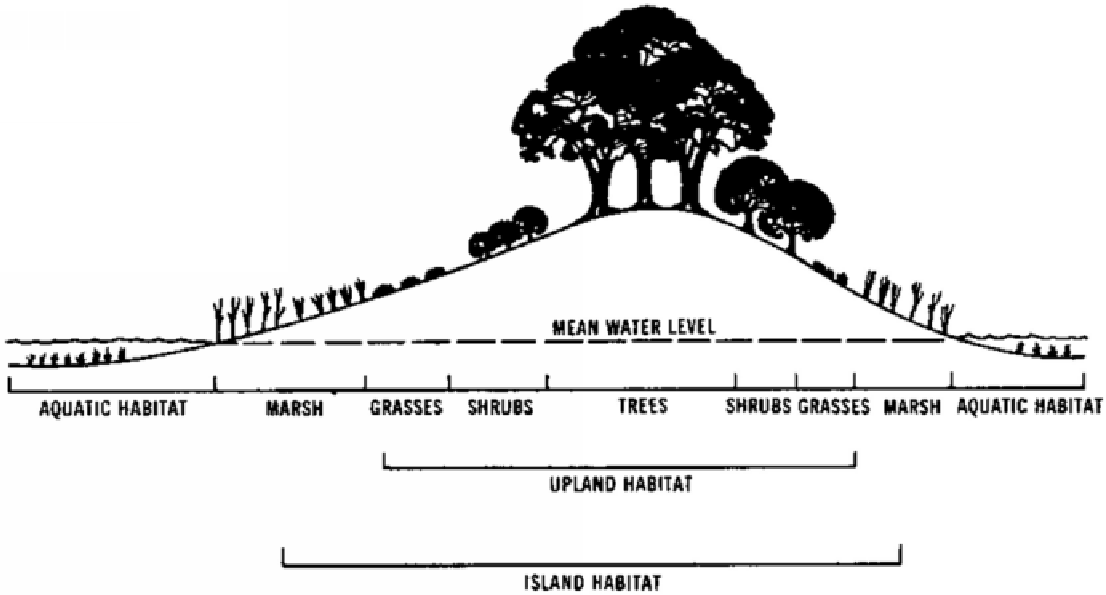


Figure 3-3: Island Habitat

(Corps, Dredging)

We believe that habitat enhancement sites should be considered because of the wide range of benefits they provide. Some of the general benefits include:

- Multiple agency participation or "buy-in" to the project
    - Our experience suggests that coalitions can be powerful, as they leverage expertise across entities and increase the probability of permitting and general project success
- Enhancement sites could be an asset to the Committee
    - Wetlands could potentially be banked, and credits sold for profit
- Increased public appeal for the project
    - Public support for these projects is almost universal
- Offsets to temporary project impacts
    - Habitat development could offset other potential impacts the project might have

Detailed analysis of these alternatives is beyond the scope of this study. However, some general considerations are offered now. The silty sand material could potentially be suitable for island development, although some bank stabilization may be required. Material classifying devices, such as those used in the mining industry for construction of tailings dams, could potentially be employed to ensure a sandy fill for the island. The US Army Corps of Engineers recommend that island habitats provide a minimum of 2-hectares, or about 5-acres of surface area. Given that hydraulically placed fill would settle at slopes from 10H:1V to 15H:1V, our estimate assumes the presence of containment dikes to minimize the total quantity required. A preliminary island site was developed to provide adequate surface area at elevation 4,105-ft, or 6.6-ft above the ordinary high-water line. Dikes could be formed from earthen materials or from dredged material placed within geotextile tubes This island (**Drawing C-103**, **Appendix B**) would require approximately 175,000 CY of sandy fill, which exceeds the proposed minimum dredging channel quantity. Therefore, participation from other project stakeholders may be required to offset those additional costs to the Committee. The National Audubon Society has previously participated in island habitat development projects and may be a potential project partner. Colorado Parks, Wildlife and Ducks Unlimited, and Delta Waterfowl are other potential project partners. This sub-alternative may potentially be feasible if such partnerships were developed. Additional detailed analysis of the island habitat is required to advance this concept.

A preliminary wetland habitat location was identified near the inlet to the reservoir, and is shown on **Drawing C-103**, **Appendix B**. This location was selected because:

- The presence of existing wetlands in the area demonstrates that appropriate conditions for establishment exist
- This area is generally shallow, allowing for efficient placement of dredged materials
- The proximity to the inlet canal would provide reliable access to irrigation water

The proposed wetland development would provide about 29-acres of surface area for wetland development assuming balanced cut/fill from dredging operations. The proposed wetland was sited to avoid impacting existing wetlands[4]. It's anticipated that spreader dikes and some supporting infrastructure would be required as part of the development. The site would be over 10,000 LF from portions of the dredge channel, so a second pumping station would likely be required to deliver hydraulically dredged materials, which is anticipated to increase pricing by 30-40% per CY.

A potential benefit of wetland development for the Committee would be wetland banking, whereby the banked credits could be sold to mitigate wetland impacts on other projects. Wetland banking requires approval from multiple agencies including the Corps, US Fish and Wildlife Service, and the Colorado State Engineer's Office. Available literature suggests that wetland development projects achieve about 65% wetland vegetation cover on average[5], placing the assumed bankable wetland area at about 18.85-acres. Wenck estimates the value of wetland credits in Colorado from $80 to $100-k per acre, equating to a likely potential bank value of $1.51 to $1.89-Mil.

3.2.4.4 Project Specific Beneficial Reuse

Opportunities for project specific reuse of dredged materials are available, particularly for the silty sands. These include wave run-up mitigation alternatives through beaching of dredged

[4] Pedestrian wetland delineations were not completed. Wetland shapes obtained from National Wetland Inventory: https://www.fws.gov/wetlands/Data/Data-Download.html

[5] *Design and Construction Considerations for Wetland Restoration Using Dredged Material.* Mohan, et al. Available at: https://westerndredging.org/index.php/woda-conference-presentations/category/56-session-2d-sediment-management?download=220:5-mohan-et-al-design-and-construction-considerations-for-wetland-restoration-using-dredged-materialpdf

materials at moderate slopes and use as soil-cement. Detailed discussion of these opportunities is provided in Section 3.3.

3.2.5 Dredging Cost Estimates

Cost estimates were developed for wet mechanical, dry mechanical, and hydraulic dredging alternatives. The price per cubic yard includes construction of containment and dewatering facilities. The estimates are provided within **Appendix E**.

3.3 WAVE MITIGATION ALTERNATIVE ANALYSIS

Alternatives for mitigating wave runup were analyzed on their technical merits. The development and analysis of these alternatives is described below.

3.3.1 Normal Freeboard Analysis

The Project seeks to restore Prewitt Reservoir to its normal operating pool at elevation 4,098.4-ft, which provides 7.5-ft of freeboard. Calculations were completed to first predict the extents of the existing embankment that would be subject to overtopping and therefore require rehabilitation. Different methods of rehabilitation were then evaluated for their ability to mitigate wave runup along those sections of the embankment.

3.3.1.1 Existing Embankment Condition

The USBR recommends considering the embankment condition when evaluating freeboard criteria for existing embankment dams (Bureau, Freeboard). A qualitative summary of the condition of Prewitt Dam embankment is provided below. Please note that detailed analysis of the embankment is beyond the scope of this study.

Table 3-2: **Recommended USBR Evaluation Criteria for Existing Embankments**

Criteria	Description
Crest Elevation, Width and Slope	Crest elevation=4,105.9-ft[6]; Width=16-ft; Slope=2H:1V (upstream). Crest is narrow with compacted surface from vehicular traffic.
Crest and Downstream Slope Face Materials	Crest Materials: Tightly compacted silty sands. The crest features a drain system (scuppers) consisting of stormwater catch-basins spaced along the embankment. Downstream Slope Materials: Sandy loess with well-established vegetation.
Vegetation	Vegetation is well established on downstream slope.
Permeability of Surface Materials	Surface materials considered highly permeable. Homogenous embankment composed of sands. Crest surface is dense.

[6] Additional survey is recommended to verify crest elevations and check for low spots.

Criteria	Description
Overall Condition of the Structure	Good. Structure has existed for over 100-yrs with no evidence of erosion.
Depth, Velocity, and Duration of Overtopping	Described below.
Wind	Described below.
Security	No available information suggests the dam is an attractive target for terrorism.

The narrow crest and permeable embankment materials adversely affect the embankment's ability to resist overtopping. The presence of a drain system called scuppers, tightly compacted crest surface, and well vegetated downstream slope add to the robustness of the embankment. These factors were considered when assigning wave overtopping exceedance probabilities described below.

3.3.1.2 Design Wind Storm

A design wind storm is required to calculate wave runup and setup. Local weather station wind data was obtained from official National Oceanic and Atmospheric Administration (NOAA) Climate Data Online (CDO) archives[7]. The Washington County Airport site in Akron, CO was selected. This site was considered highly representative of conditions at Prewitt due to its proximity and similar topography. The site features a robust 22-yr period of record and valuable data on sustained 2-min peak gust time. These data were used instead of the instantaneous peak gust data, as sustained winds are required for legitimate dam safety hazards.

The USBR recommends adjusting overland wind speeds to over water velocities when calculating wave heights (Bureau, Freeboard). Thus, overland values obtained from the Akron, CO station were adjusted by 0.90 using the following chart:

[7] Available at www.noaa.gov . Station Name: AKRON WASHINGTON CO AIRPORT, CO US. Station ID: USW00024015.

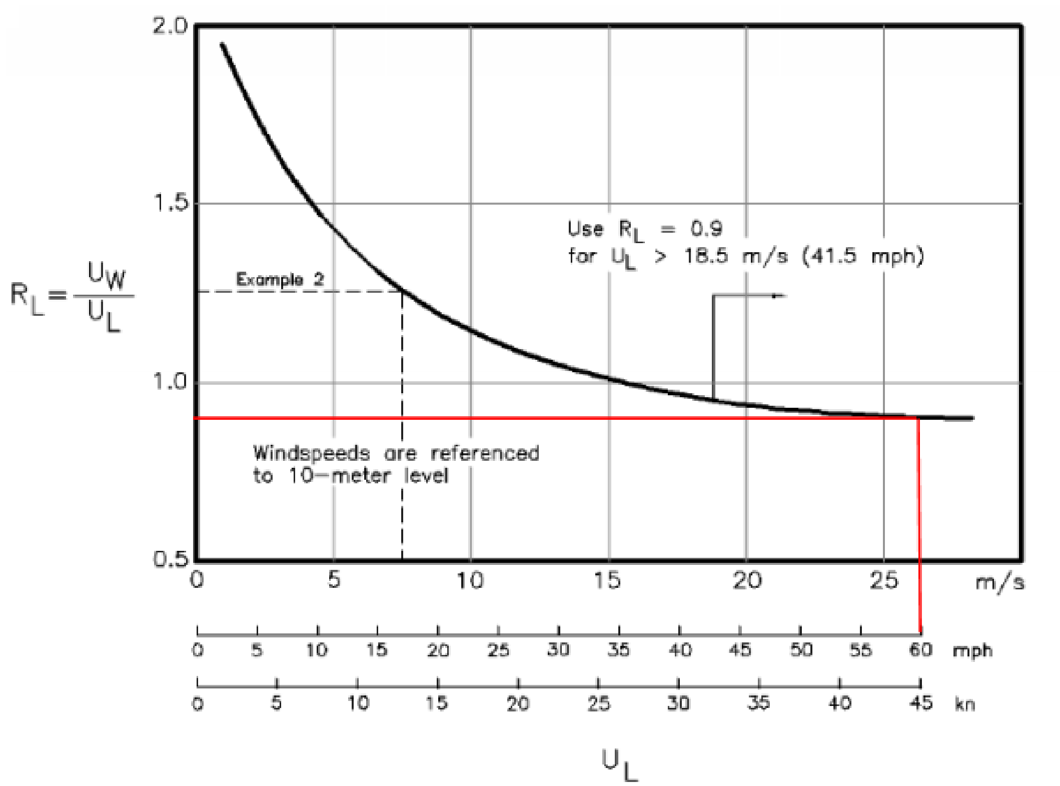


Figure 3-4: Over Land to Over Water Wind Speed Relations

Adjusted wind data for the Akron, CO site are presented in **Figure 3-5**:

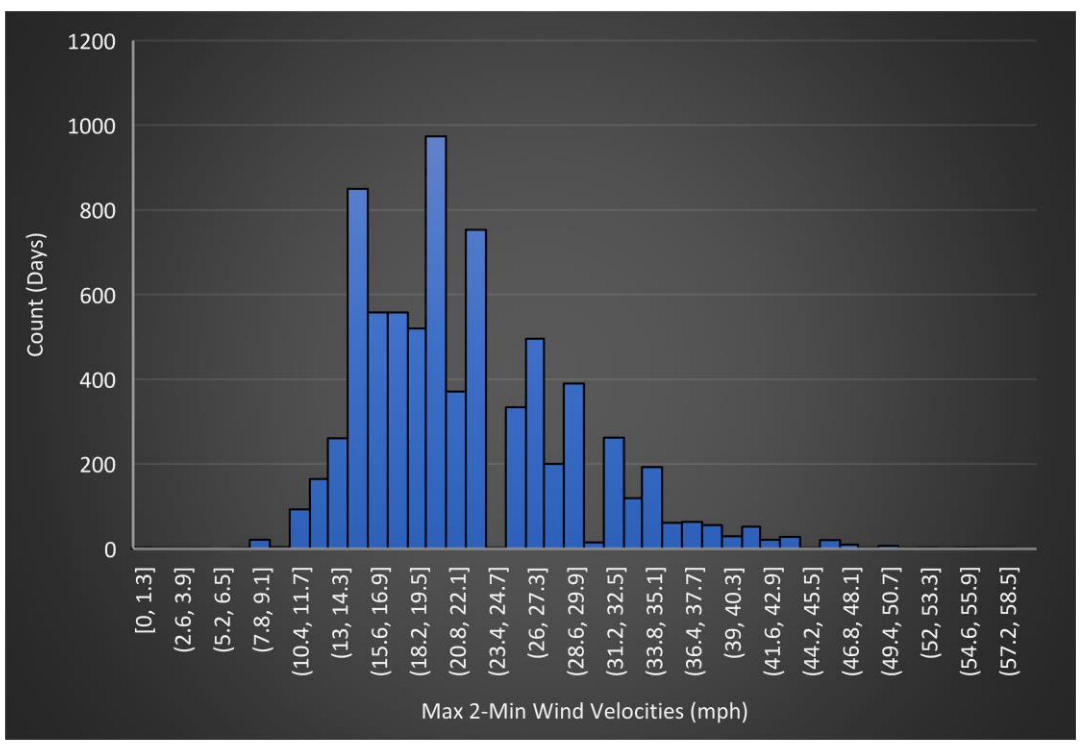


Figure 3-5: Akron, CO Adjusted Max 2-Min Wind Speed Histogram (1996-2018)

The data features an adjusted maximum 2-min wind speed of 59-mph[8]. This is significantly less than the 100-mph design storm recommended by the USBR. Therefore, a reduction to a 90-mph design wind storm was tested. A probabilistic approach was taken to determine the chances of meeting or exceeding the 90-mph speed. Wenck completed a regression analysis of the data using both Weibull and Rayleigh probability density functions, which are the most commonly accepted methods for modeling wind speeds over time (Journal of Modern Energy, 2016). The plots of the Weibull and Rayleigh synthesized wind data for the Akron, CO site are provided in **Figure 3-6**:

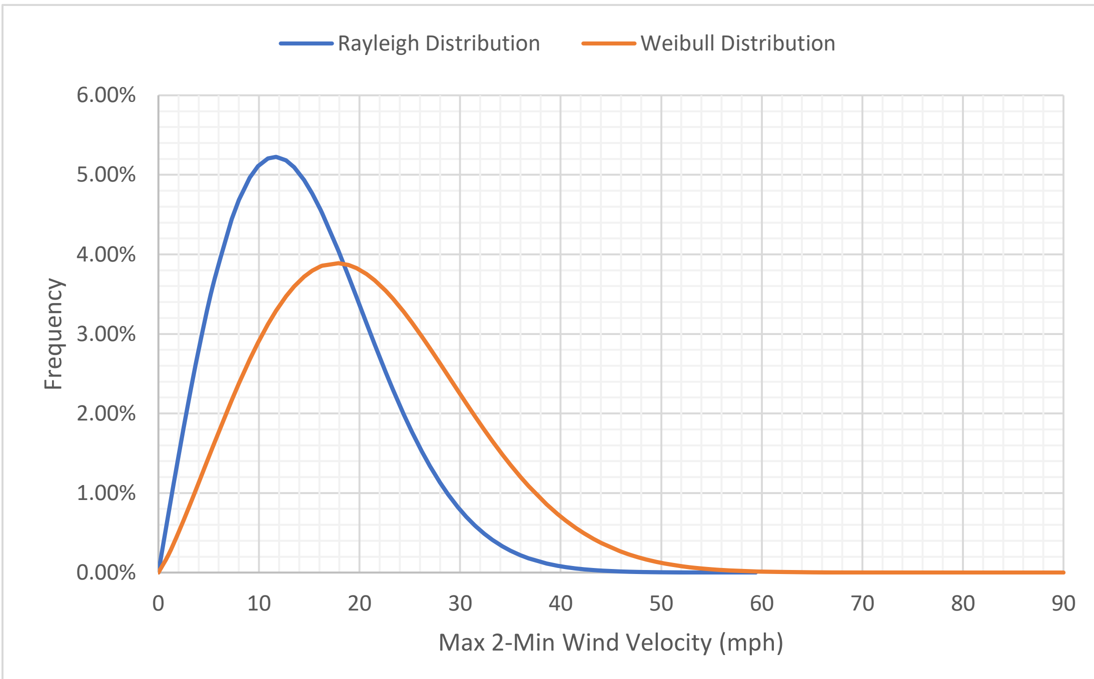


Figure 3-6: **Akron, CO 2-Min Synthesized Wind Speed Probability Distribution**

Using the Rayleigh probability function, the probability that 2-min wind will meet or exceed 90-mph on any day is $5.8x10^{-14}$. Assuming a 100-yr lifecycle, the number of days (36,500) times the daily probability ($5.8x10^{-14}$) equals $2.13x10^{-9}$. Using the Weibull distribution, the probability of a 90-mph wind velocity over the assumed lifecycle is $1.04X10^{-4}$. Thus, the 90-mph design wind storm is considered valid.

3.3.1.3 Reservoir Fetch Calculations

Reservoir fetch is a major factor in the development of waves. The great length (~3.43-Mi) and meandering orientation of Prewitt's embankment required an extensive analysis of reservoir fetch. Wenck organized the embankment into segments based upon their general

[8] A clear outlier suggesting a 160-mph wind was removed from the data set.

orientation to accomplish this. These segments are shown on **Drawing C-104** within **Appendix B**.

Fetch values were calculated for each segment using the traditional USBR method, which measures the average length of nine radials projected from a single point, with each radial offset 3-degress from the central radial. Additionally, radials were rotated to find the potential maximum fetch value for each location. These waves would strike the embankment surface at an angle, so reduction factors were later applied when calculating runup. The results of the fetch analysis are provided in **Table 3-3**:

Table 3-3: **Reservoir Fetch Values**

Embankment Sta Start.	Embankment Sta Stop	Max Fetch Value (Miles)
0+00	8+00	3.05
8+00	16+50	0.50
16+50	54+50	3.06
54+50	76+50	2.82
76+50	90+50	2.54
90+50	106+50	1.45
106+50	126+50	2.13
126+50	153+00	2.02
153+00	181+00	2.31

3.3.1.4 Wave Runup and Setup Calculations

Wave runup and wind setup were calculated for each reservoir segment and compared against the proposed freeboard levels. Segments whose predicted runup plus setup value exceeded available freeboard were identified for remediation.

Wave runup is the maximum vertical distance obtained by a wave traveling up an embankment. Wenck used standard USBR methodology to calculate wave run-up. Significant wave height, wave length, wave period, and surf similarity factors were completed. A design wave equal to 1.27*(Significant Wave Height) was selected based upon the embankment evaluation. Angular spread reduction factors were applied to segments whose maximum fetch was not normal to the embankment. No surface roughness reduction factor was applied due to the existing smooth concrete surface.

Wind setup, or "wind tide", is the rise in water surface elevation that occurs on the downwind side of the reservoir due to shear forces of wind on the water surface. Values for wind setup were calculated using standard USBR methods and added to wave runup for each segment to determine the final predicted value. A summary of these values is shown in **Table 3-4.** The embankment extents requiring mitigation are shown on **Drawing C-104** within **Appendix B**.

Table 3-4: **Existing Embankment Freeboard Analysis Results**

Embankment Sta Start.	Embankment Sta Stop	Predicted Wave Runup (ft)	Predicted Wind Setup (ft)	Total (ft)	Predicted Overtopping Height (ft)
0+00	8+00	11.79	0.88	12.67	5.17
8+00	16+50	1.22	0.14	1.36	0.0
16+50	54+50	11.83	0.88	12.71	5.21
54+50	76+50	8.18	0.81	8.99	1.49
76+50	90+50	7.65	0.73	8.38	0.88
90+50	106+50	4.87	0.42	5.29	0.0
106+50	126+50	7.31	0.62	7.93	0.43
126+50	153+00	6.47	0.58	7.05	0.0
153+00	181+00	7.40	0.67	8.07	0.57

3.3.2 Wave Runup Mitigation Alternatives

Wave runup mitigation alternatives were developed and analyzed for the segments identified above. Alternatives include beaching dredged material, soil cement facing, parapet walls, breakwaters, and pre-manufactured revetment systems.

3.3.2.1 Beaching Material

Fill materials would be directly placed on the existing dam face for this concept. The materials could be selectively borrowed or recycled from dredging operations. The silty sands are considered superior for this application. The fine-grained clays found near the outlet works were quickly removed from consideration due to their erosive potential[9].

Analysis of this sub-alternative presented several challenges, including:

- Identification of effective slopes for beaching the material
- Evaluation of dredged vs. mechanically placed methods
- Considerations for durability and long-term performance

Beaching Slopes. Beaching slopes were evaluated based upon both their ability to mitigate wave runup and geotechnical stability. Analysis indicates that the most problematic areas requiring major mitigation need minimum beaching slopes of 4H:1V, while a steeper 3H:1V would perform in areas requiring moderate mitigation. Slope stability analysis (Appendix D) completed at the minimum embankment slope of 2.5H:1V indicates that the silty sand materials would be stable. Slopes placed at 6H:1V would be easily accessible to maintenance equipment. Slopes flatter than 15H:1V should resist erosion and are recommended for placement (Corps, Dredging).

[9] Recommendation from Hollingsworth Associates, Inc.

<u>Placement</u>. Direct placement of hydraulically dredged fill on the slope would create natural segregation of the silt and sands and prevent compaction. Mechanically placed materials would retain a higher content of fines and could be placed in compacted lifts, thereby increasing their final densities and providing a more stable fill.

<u>Durability and Performance</u> The durability and performance of the material will be affected by particle grain size, final density, slope, vegetation, and variations in reservoir pool levels. A summary of these factors is provided below:

Table 3-5: **Factors Affecting Unprotected Embankment Durability and Performance.**

Factor	Description
Particle Grain Size	The material to be placed is known to be a silty to slightly silty sand. Those materials will be subject to erosion from wave action.
Density	Compaction in lifts is presumed to improve durability. Hydraulically placed fill is considered less durable.
Slope	<u>3H:1V to 4H:1V</u> Required to mitigate wave runup. <u>6H:1V</u> Required for maintenance equipment access. <u>15H:1V</u> Generally recommended for non-erosive conditions (Corps, Dredging).
Vegetation	Turf grasses can often provide protection against erosion. Turfs should be selected to tolerate the appropriate levels of inundation. A soil cap capable of supporting the vegetation would be required.
Pool Levels	Variable pool levels are anticipated, which requires adequate protection along the entire face of the embankment. Sustained pool levels may cause benching and backward erosion for fill placed at slopes steeper than 15H:1V.

This sub-alternative is considered technically feasible for slopes equal to or greater than 15H:1V. Slopes steeper than this would mitigate wave run-up up to a 4H:1V but would require routine maintenance.

3.3.2.2 Soil Cement Facing

Soil cement is a compacted mixture of soil, Portland cement, and water. It is widely recognized for use as embankment facing. The most desirable soils for soil-cement are silty sands (Bureau, Soil Cement). These materials are believed to be abundant in the area, so use of soil cement facing was analyzed.

Soil cement may be placed using either stair-stepped method or on a flat surface. The latter method is referred to as "plating". The stair-stepped method provides wave dissipation through additional surface roughness and/or slope moderation, while the plating methods relies entirely on slope moderation to mitigate wave runup. Examples of stair-stepped and plated soil cement applications are show below:

Figure 3-7: **Stair-Stepped Soil Cement**
Source: USBR

Figure 3-8: **Plating Method of Soil-Cement**
Source: USBR

Locally available sands were analyzed for their potential use as soil cement. Sands were obtained from a test pit (TP-1) located on the north east shore of the reservoir. The approximate location of TP-1 is shown on **Drawing C-101** in **Appendix B**. The material was tested using the USBR recommended "Bonny" criteria which include wet-dry and freeze-thaw durability, and 7-day and 28-day compressive strengths. The results of the Bonny testing program are provided in **Table 3-6**:

Table 3-6: **Bonny Criteria Test Results**

Bonny Criteria	7% Cement	9% Cement	11% Cement	USBR Criteria
Wet-Dry Durability (% Loss)	18.7	7.6	1.2	<6.0
Freeze-Thaw Durability (% Loss)	4.5	3.7	2.0	<8.0
7-day Compressive Strength	149	304	348	>600
28-day Compressive Strength	229	487	594	>875

The compressive strengths did not achieve recommended values; however, those tests are considered supplementary to the more important durability measures (Bureau, Dams). Additionally, the material used for the Bonny testing was obtained from a shoreline location that had been washed clean of most fines and featured a narrow sand gradation. We believe that the silty sands identified in the reservoir from SP-2 and SP-3 would provide a more suitable material. These data suggest that local materials are conducive to use as soil cement.

Calculations indicate that stair-step placement at a slope of 2.5H:1V would effectively mitigate wave run-up. Plated soil cement placed at 4H:1V would mitigate wave run-up, however constructability at those slopes could be difficult. 6H1:1V is considered a minimum for expedient placement and maintenance.

Various combinations of soil cement facing were considered, including:

Stair-Stepped Placement up the Entire Face of the Embankment

Stair-stepped placement up the entire dam face would require the greatest quantity of soil cement and thereby the greatest cost. However, this option is considered the most robust soil cement sub-alternative. A typical section was developed for this sub-alternative, and is shown on **Drawing D-101** within **Appendix B**. This sub-alternative is technically feasible.

Stair-Stepped Placement along Upper Half of the Embankment

This concept was considered to reduce the quantity of soil-cement. Stairs would be placed where they would provide the greatest benefit for mitigating wave runup. A continuous concrete anchor doweled into the existing panels would provide for a foundation. This sub-alternative is somewhat of a novelty that requires careful evaluation beyond the scope of this study.

Plated Placement

Plating soil-cement would significantly reduce the quantity of cement required. However, this method is much less robust than typical stair-stepped placement and may not perform well with ice loads, the large significant wave heights and lengths anticipated, variations in reservoir pool, and prolonged periods of freeze-thaw. Slopes of 5H:1V are recommended for constructability purposes. This sub-alternative is not considered technically feasible as the sole treatment method.

Plated/Stair-Stepped Composite

A composite section using both placement methods was developed for the purpose of utilizing plated soil-cement in such a way that would minimize its exposure to the elements. Operational model simulations suggest that reservoir water surface elevations are typically equal to or above 4,087.4-ft during the winter months, which are inferred to be the most damaging to soil-cement materials. A combination section could use plated soil-cement below this elevation and the more robust stair-stepped section above. Efficient use of materials would help to minimize cost of this sub-alternative. A typical section was developed for this sub-alternative and is shown in **Drawing D-101** within **Appendix B**. This sub-alternative is considered technically feasible.

It should be noted that proper foundation preparation and material compaction is critical to the success of any soil cement option. Differential settlement can cause failure of these elements and must be controlled.

3.3.2.3 Parapet Walls

Parapet walls are vertical walls placed into the top of a dam that increase its freeboard. They are typically cast-in-place concrete and should be tied into the impervious core of a zoned dam. They should only be used to provide freeboard for wave run-up, not setup or flood storage. (Bureau, Freeboard). Parapet walls are considered technically feasible for Prewitt because the dam is homogenous, so connection with a core would not be required, and they would not be used to retain set-up for flood storage[10], A typical parapet wall section was developed, and is shown on **Drawing D-101, Appendix B**. The section would be difficult to construct, requiring stripping and demolition of upper portions of the dam. The wall would also further limit access to the already narrow crest. Finally, achieving adequate contact with the existing parapet and facing may be difficult.

[10] The IDF was not modeled, however it was reasonably assumed that the embankment in its current configuration would not overtop.

3.3.2.4 Breakwaters

Detached Breakwaters (Permanent Structures)

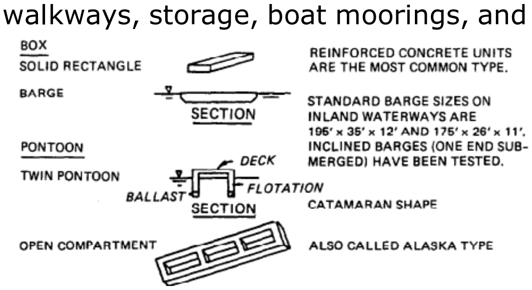
Detached breakwaters are structures constructed parallel to shore that reduce the amount of wave energy reaching the protected area by dissipating, reflecting, or diffracting incoming waves (Corps., Breakwaters). They are typically composed of rubble mound or vertical wall structures. These types of breakwaters were not considered feasible for this application because:

> *"…detached breakwaters include limited design guidance, high construction costs, and a limited ability to predict and compensate for structure-related phenomena such as adjacent beach erosion, rip currents, scour at the structures base, structure transmissibility, and effects of settlement on project performance."* (Corps., Breakwaters)

Floating Breakwaters

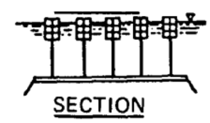
Floating breakwaters represent a special sub-class of breakwater. They are generally less effective at wave attenuation than fixed structures but are less expensive and movable. They function through reflection, dissipation, interference, or conversion of the incoming wave energy into oscillatory motion. Over 60 different configurations are recognized, but they are generally organized into the following groups (Corps, Breakwaters and Jetties):

- **Pontoon Floating Breakwaters**
  - The simplest form of floating breakwater which has been extensively studied. Their radius of gyration, in addition to their mass, contributes to wave attenuation. These structures also offer multi-use potential for walkways, storage, boat moorings, and fishing piers (Corps, Floating Breakwater Design). It's reasonably assumed that avian species would also utilize such systems at the site. These systems provide a potentially feasible alternative that should be investigated further if floating breakwater options are pursued.

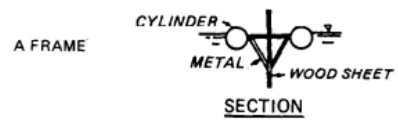


- **Sloping-Float (Inclined Pontoon) Breakwaters**
  - The sloping-float break-water consists of flat panels moored in such a way that, in still waters, the panels have one end resting on bottom and the other protruding above the water surface (Corps, Floating Breakwater Design). This type of breakwater was removed from consideration, as the total depth of the pool, and variations in reservoir water surface, would make this type of breakwater prohibitively large and difficult to manage.

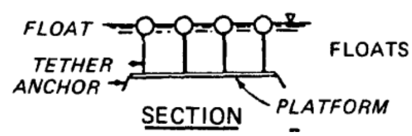
- **Scrap-Tire Floating Breakwaters**
    - Development of these systems was motivated by a need to utilize scrap tires which have been accumulating in mass throughout the United States. The resilient nature of the tires has made them suitable for use as breakwaters. These assemblies have a robust history of use and investigation. A configuration called the "Wave-Maze" has been patented, while the "Wave-Guard" assembly has been experimentally studied. These systems provide a potentially feasible alternative that should be investigated further if floating breakwater options are pursued, although special environmental considerations may exist. (Corps, Floating Breakwater Design)



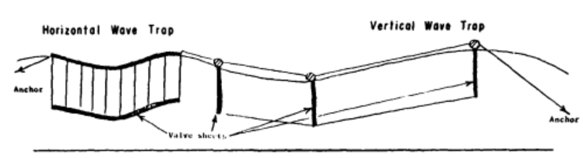
- **A-Frame Floating Breakwaters**
    - The A-Frame is a type of floating breakwater that was originally developed by the Canadian Department of Public Works. The structure consists of two floating cylinders joined together with a central vertical wall. The design concept utilizes a large moment of inertia, rather than large mass, to effectively attenuate incoming waves. These systems provide a potentially feasible alternative that should be investigated further if floating breakwater options are pursued. (Corps, Floating Breakwater Design)



- **Tethered-Float Breakwaters**
    - This type of floating breakwater consists of many buoyant floats independently tethered below the water surface. Their primary means of attenuation is drag, however the buoys can oscillate out of phase with the incoming waves, transforming wave energy into turbulence. These systems are not considered feasible for attenuating the large significant wave heights and periods anticipated at Prewitt. (Corps, Floating Breakwater Design)



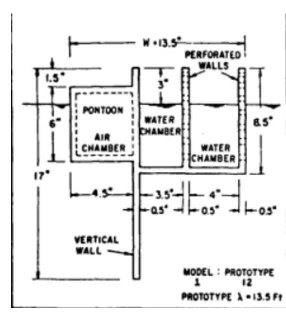
- **Porous-Walled Floating Breakwaters**
    - This breakwater consists of a vertical or included perforated wall. They principally work by reflecting incoming waves. As such, they require great structural strength and for that reason are not considered feasible for the Project. (Corps, Floating Breakwater Design)
- **Pneumatic and Hydraulic Breakwaters**
    - These are active systems that use air-bubbles or hydraulic jets to induce wave breaking, which dissipates the wave energy. These systems are not considered feasible for the project due to costs, operation and maintenance, and uncertainties regarding scalability. (Corps, Floating Breakwater Design)
- **Flexible Membranes**
    - This category consists of both blanket layer and bag type systems, of which the blanket type is considered superior. These systems have been experimentally tested, although results do not clearly demonstrate their usefulness as floating breakwaters. One patented system, the Wave Trap, appeared to be restricted to wave heights less



than 1-ft during testing. Limited testing also suggests that these structures must be very wide relative to wavelength to maintain effectiveness. These systems were therefore dismissed from further evaluation. (Corps, Floating Breakwater Design)

- **Turbulence-Generator Floating Breakwaters**
    - These structures are attractive as they allow for efficient dissipation of wave energy without undue stresses on the system. They typically operate by inducing wave breaks over the structure that dissipate the wave energy (Corps, Floating Breakwater Design). However, there are other ways of creating turbulence to reduce the wave energy. A type of manufactured system called Wave Eater should likely be placed under this category. The Wave Eater consists of baffled drums designed to rotate as the incoming wave strikes them. Wenck contacted the manufacturer of these systems regarding their viability for use at Prewitt and received positive initial feedback. Budgetary price proposals ranging from $1.07 to $1.45-Mil were received for 2-string and 3-string systems, respectively. The manufacture has provided initial warranty that the 3-string system would provide essentially zero wave transmission and the 2-string system would allow passage of 6-12-in waves with the maximum design wave height of about 6-ft. These systems provide a potentially feasible alternative that should be investigated further if floating breakwater options are pursued.

- **Peak Energy Dispersion Floating Breakwaters**
    - These systems essentially create a process of destructive wave interference through reflection of incoming waves. Both static, rigid wall, and dynamic, floating wave emitters, are known. Static systems can be offset to reduce forces on the structure, however these are still not considered feasible for the Project. The dynamic type creates extremely complicated non-linear systems that will respond differently to different wave lengths. Testing suggests that they become ineffective for ratios of wavelength to water depth over 2.5, which may be possible at Prewitt. They were removed from consideration. (Corps, Floating Breakwater Design)
- **Reservoir Application Floating Breakwaters**
    - As the name suggests, these types of breakwaters have been experimentally developed for reservoir applications, that is, they can easily accommodate large fluctuations in the water surface elevation. However, these systems consist of exotic configurations that would be difficult to implement on the scale required at Prewitt. It also appears that they have only been subjected to small scale laboratory testing, where the effects of scale, mooring forces, and mooring line elasticity were not evaluated.



There is no known precedent for use of floating breakwaters for embankment protection in Colorado, however preliminary conversations with CO Dam Safety suggests that, given enough supporting data, they could potentially be deployed in such a manner.

3.3.2.5 Pre-Manufactured Revetments

Articulated concrete block (ACB) mats and geosynthetic turfs were considered. Both products are used for erosion protection and energy dissipation. Both systems would require anchorage or embankment onto the existing sloped paving, which would be difficult to achieve, particularly for geosynthetic turfs. USBR surface roughness reduction factor for grass is only 0.9, so it's believed that these systems would only provide marginal protection against wave run-up. Geosynthetic turfs were therefore quickly removed from consideration.

The Committee expressed concerns over the potential detrimental effect that ice may have on ACB mat systems. Reservoir ice has been observed to displace the panels when close contact is present, so there is legitimate concern that the addition of anchored or embedded mats could aggravate this problem. Conversely, non-embedded mats would certainly be displaced and rendered ineffective. Additionally, recent bids opened by Wenck indicate that the systems can be expensive, with unit pricing reaching $35-40/SF. Pre-manufactured revetments were removed from consideration for these reasons.

3.3.3 Wave Mitigation Cost Estimates

Cost estimates for technically feasible wave mitigation alternatives and sub-alternatives were developed and are provided within **Appendix E**.

4.0 Financial Feasibility Analysis

4.1 COSTS OF STORAGE

Cost estimates for the preferred alternatives recommended in Section 5 (hydraulic dredging with environmental enhancements and floating breakwater) were used to estimate the cost per ac-ft of storage gained in Prewitt Reservoir. The cost of storage gained by reconnecting the dead pool is estimated to be $2,725/ac-ft, based on an increase in storage capacity of 1,604 ac-ft, for a total cost of $4.37 million. The cost per ac-ft of storage gained from lifting the storage restriction is estimated to be $578/ac-ft, based on an increase in storage capacity of 3,455 ac-ft, for a total cost of $2.0 million. For both stages combined, the cost per ac-ft of storage gained is estimated to be $1,259/ac-ft, based on an increase in storage capacity of 5,059 ac-ft, for a total cost of $6.37 million.

Although Prewitt Reservoir water rights cannot be stored in other locations, LWS compared the cost per ac-ft of storage gained in this project to some similarly-sized rehabilitation projects evaluated in the recently-completed South Platte Storage Study (SPSS). The cost per ac-ft gained in the rehabilitation of other storage reservoirs in the Lower South Platte region range from $3,700/ac-ft for 10,000 ac-ft of storage at Jackson Lake Reservoir to $5,400/ac-ft for 5,700 ac-ft of storage at Julesburg Reservoir, with the average project cost being $4,900/ac-ft of storage gained. By comparison, the proposed rehabilitation of Prewitt Reservoir would yield a similar volume of storage for about 1/3rd the cost per ac-ft of the next most cost-effective project evaluated in the SPSS.

5.0 Conclusions and Recommendations

5.1 ALTERNATIVE RANKINGS

Alternatives and sub-alternatives determined technically feasible were ranked using decision matrix. The purpose of each matrix is to objectively evaluate each alternative using multiple criteria for selection of the optimal alternative without bias. Likely project sequencing was contemplated because of the dual-purpose nature of the project.

5.1.1 Decision Matrix Results

Dredging and wave mitigation alternatives were scored using weighted decision matrices. Dredging projects were scored on capital costs, reliability, sequencing, and multi-use potential. Mitigation projects were evaluated based upon their reliability, robustness, capital costs, and expected operation and maintenance. Reliability is the likelihood that the alternative functions per the design intent. Robustness is the ability of the alternative to resist/endure unforeseen conditions. Items were scored on a scale of 1-5 with 5 being better, so the highest total score is the preferred alternative. The results are provided within **Table 5-1** and **5-2** on the following page.

Table 5-1: Dredging Alternative Decision Matrix

Option	Capital Costs	Reliability	Sequencing	Multi-Use Potential	Total
Factor Weight (1-5)	**5**	**5**	**4**	**5**	
1) Cutterhead Hydraulic Dredging	5	5	5	5	95
2) Wet Mechanical Dredging	2	2	5	5	65
3) Dry Mechanical Dredging	3	5	1	4	64

Table 5-2: Wave Mitigation Alternative Evaluation

Option	Capital Costs	Operation and Maintenance	Reliability	Robustness	Total
Factor Weight (1-5)	**5**	**3**	**5**	**4**	
4) Cutterhead Hydraulic Dredging	2	5	5	5	70
5) Wet Mechanical Dredging	2	4	5	4	63
6) Dry Mechanical Dredging	3	3	5	4	65
7) Parapet Wall	3	5	5	5	75
8) Floating Breakwater	5	4	4	4	73

The following rationale were used for scoring the dredging alternatives:

- Cutterhead hydraulic dredging
  - Likely the most cost-effective option. Pre and post dredge bathymetric surveys could reliably confirm the extents of dredging. This method of dredging also lends

itself to enhancement site development and would not significantly impact normal operation of the reservoir.

- Wet Mechanical Dredging
  - Would be the least cost effective alternative as it would require double handling of the materials. Loaded materials could have multi-use potential. Wet mechanical dredging could be effectively sequenced with wave mitigation alternatives.
- Dry Mechanical Dredging
  - Effective dewatering of the dead pool area to provide reliable equipment access is considered very difficult and would likely impact project costs. Multi-use potential for enhancement sites is somewhat limited due to haul distances. Dry mechanical dredging would impact the reservoir, and potentially impact project sequencing for wave mitigation alternatives.

The following rationale were used for scoring the wave mitigation alternatives:

- Embankment Beaching
  - This option is considered very reliable and robust. The effectiveness of this has been documented. The massive amounts of cut-fill make this option the most expensive alternative.
- Soil-Cement: Stair-Stepped Placement
  - Reliable and robust, this option would provide effective protection against wave runup. However, soil cement faces can require periodic maintenance, and this option would be capital cost intensive.
- Soil Cement: Composite Section
  - The composite section was developed to reduce capital costs while providing an equally robust and reliable soil cement alternative. Proper foundation preparation and material compaction would be required. Some maintenance work should be anticipated.
- Parapet Walls
  - Parapet walls would be extremely robust and require nominal maintenance. Their reliability is somewhat questionable due to the limited design guidance. They would also be costly to implement.
- Floating Breakwater
  - The most cost-effective option, these units would require periodic maintenance and may be less reliable and robust than other alternatives. Additional evaluation is required to confirm these criteria.

5.2 FINAL RECOMMENDATIONS

Wenck's final project recommendations for both dredging and wave mitigation are provided below. Projects are shown separately below. The recommended alternatives differ greatly, so we believe there would be little utility in implementing them concurrently, although they could potentially be completed under a single contract.

5.2.1 Phase-I Dredging Project

Wenck recommends that the Committee pursue a cutterhead hydraulic dredging project with ecological enhancements from the dredged material. Model simulations suggest there is robust need for the 1,604 ac-ft of storage this would provide. The immediate addition of that storage to Prewitt's active pool should provide 476 ac-ft of additional reservoir yield during dry years, when the water is most needed by irrigators.

Potential project partners and funding sources should be sought. These may include:

- Colorado Water Conservation Board (CWCB) Water Plan Grant Funding
- CWCB Loans
- Colorado Parks and Wildlife
- National Audubon Society
- Wetland Bank Sales (Potential Future Funding Stream from Wetland Development Sub-Alternative)
- Ducks Unlimited
- Delta Waterfowl

The wetland development sub-alternative seems particularly beneficial. The US Army Corps of Engineers, US Fish and Wildlife Service, and State Engineer should be engaged to determine feasibility of developing a wetland bank, for which there appears to be robust demand. This ecological enhancement also clearly increases the probability of securing grant funding.

5.2.2 Phase-II Wave Mitigation Project

Wenck recommends the Committee pursue wave mitigation alternatives after completion of the Phase-I dredging project. Lifting the restriction would provide 3,455 additional ac-ft of storage and should provide 1,001 ac-ft of additional yield during dry years. The floating breakwater alternative is recommended for further evaluation. The alternative graded comparatively well to others and is significantly more cost effective than all other known options. This alternative represents an uncommon application that would largely set precedent for implementation at the size and scale required at Prewitt. Therefore, additional evaluation is required prior to final implementation. This should include continued consultation with CO Safety of Dams Branch, who through initial correspondence, has indicated that the application may be allowable given adequate scientific justification.

6.0 References

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Appendix A

Prewitt Operating Committee Organizational Charter

The Prewitt Operating Committee

112 North 8th Avenue - P.O. Box 333 - Phone (970) 522-2025
STERLING, COLORADO 80751-0333

July 8, 2014

PREWITT RESERVOIR

<u>OPERATING UNDER THE NAME OF:</u>

Prewitt Operating Committee
P.O. Box 333 - 112 North 8th Avenue
Sterling, Colorado 80751

The Prewitt Operating Committee is a management organization used to perform the operation of the Prewitt Reservoir. It is composed of three entities - The Logan Irrigation District, The Iliff Irrigation District, and The Morgan Prewitt Reservoir Company. It is governed by a Governance Contact adopted January 31, 2006 (see attached). The ownership breakdown and their board members are provided below:

THE LOGAN IRRIGATION DISTRICT - 17/31 Interest

Organized under the Colorado Irrigation District Law of 1905 on December 19, 1910.
Elected Governing Board Members: Gerald Ruf - President, Bob Lingreen - Vice-President
Rod Mari - Director. James Yahn - Secretary/Manager - appointed by the board.

No. of Landowners:	128
District Acres:	12,818.9 acres
Current Levy:	$4.00/acre

THE ILIFF IRRIGATION DISTRICT - 8/31 Interest

Organized under the Colorado Irrigation District Law of 1905 on March 6, 1911.
Elected Governing Board Members: John Stieb - President, Bob Monheiser - Vice-President
Dave Breidenbach - Director. James Yahn - Secretary/Manager - appointed by the board.

No. of Landowners:	65
District Acres:	10,874.3 acres
Current Levy:	$2.50/acre

THE MORGAN PREWITT RESERVOIR COMPANY - 6/31 Interest

Organized as a corporation not for profit, under the General laws of the State of Colorado on February 10, 1923.
Elected Governing Board Members: Allyn Wind - President, Don Chapman - Vice-President
Wade Castor - Director, Robert Karg - Director, Brad Mortensen - Director.
Don Snider - Secretary - appointed by the board and James Yahn - Manager - appointed by the board.

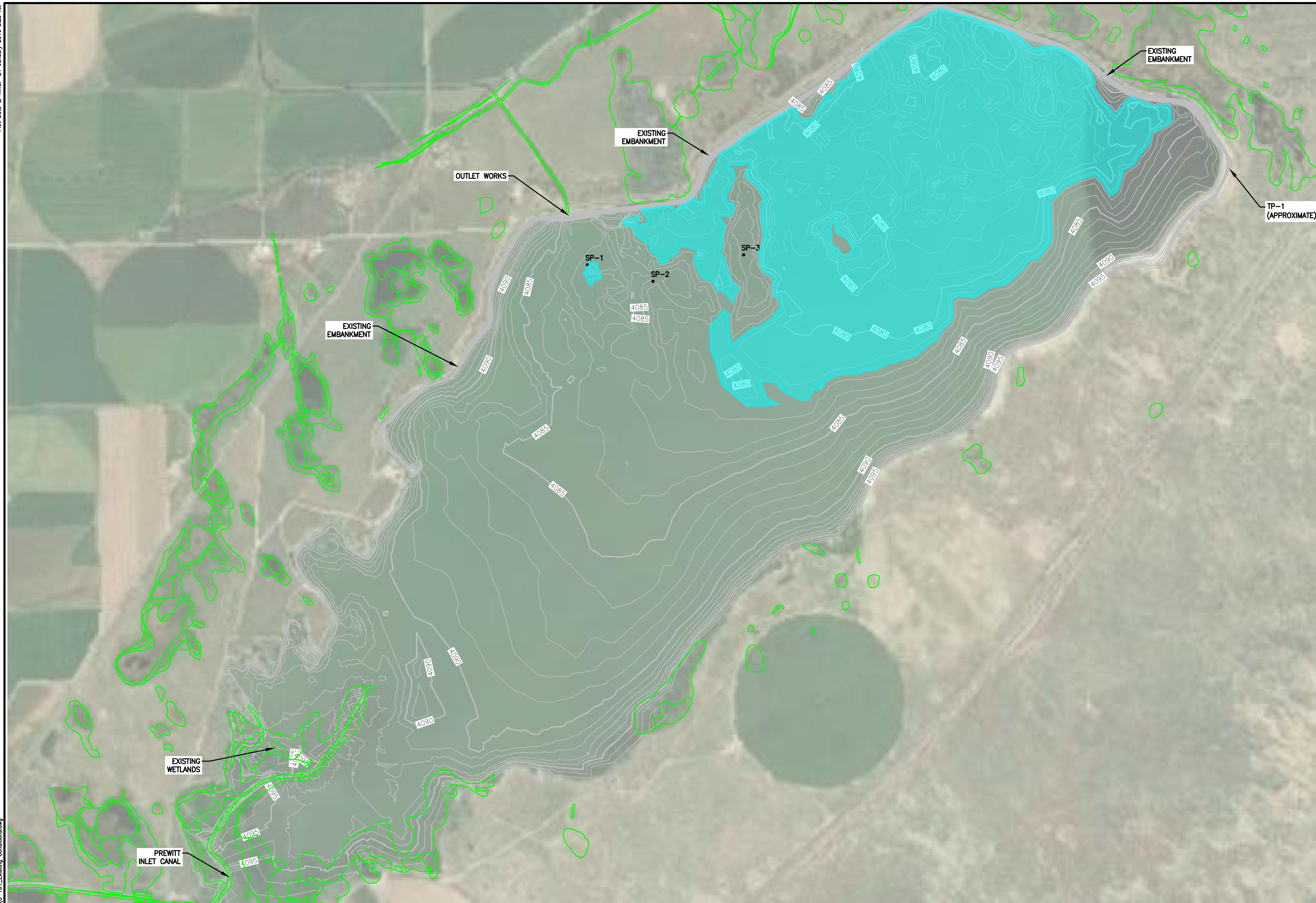
No. of Shareholders:	58
No. of Shares:	261
Current Assessment:	$125/share

<u>PREWITT RESERVOIR SPECIFICATIONS</u>

Water Rights:	Priority No. 75A - Storage of 32,300 acre feet at a rate of 695 cfs May 25, 1910
	Priority No. 75R(Refill) - Storage of 34,960 acre feet at a rate of 695 cfs Dec 31, 1929
Restricted Storage:	28,600 acre feet
Use of Water:	Supplemental Irrigation Supply for approximately 28,000 acres, augmentation, and recreation
Service Area:	Morgan and Logan Counties
Avg. Diversion:	40,160 acre feet
Length of Dam:	3 ½ miles
Height of Dam:	36 feet
Surface Area:	2,300 acres at restricted level

Appendix B

Drawings



RESERVOIR DEAD POOL ~1,604 AC-FT

GENERAL NOTES

1. RESERVOIR BATHYMETRIC SURVEY DATA COMPLETED BY OTHERS AND PRESENTED "AS-IS" WITHOUT WARRANTY TO ITS ACCURACY.
2. EXISTING WETLAND SHAPES OBTAINED FROM US FISH AND WILDLIFE NATIONAL WETLAND INVENTORY. PEDESTRIAN SURVEYS NOT COMPLETED.

N

0 800 1600
GRAPHIC SCALE IN FEET

#	DESCRIPTION	XXX	XXX	XX/XX/XX
REV	REVISION DESCRIPTION	DWN	APP	REV DATE

SEAL

SUB CONSULTANT

PRIME CONSULTANT
WENCK
Responsive partner. Exceptional outcomes.

PROJECT TITLE
PREWITT RESERVOIR REHABILITATION FEASIBILITY STUDY

PREWITT OPERATING COMMITTEE

SHEET TITLE
EXISTING CONDITONS

DWN BY	CHK'D	APP'D	DWG DATE	DEC. 2018
JVB	MJK	MJK	SCALE	AS NOTED

PROJECT NO.	SHEET NO.	REV NO.
7294-0003	C-101	0

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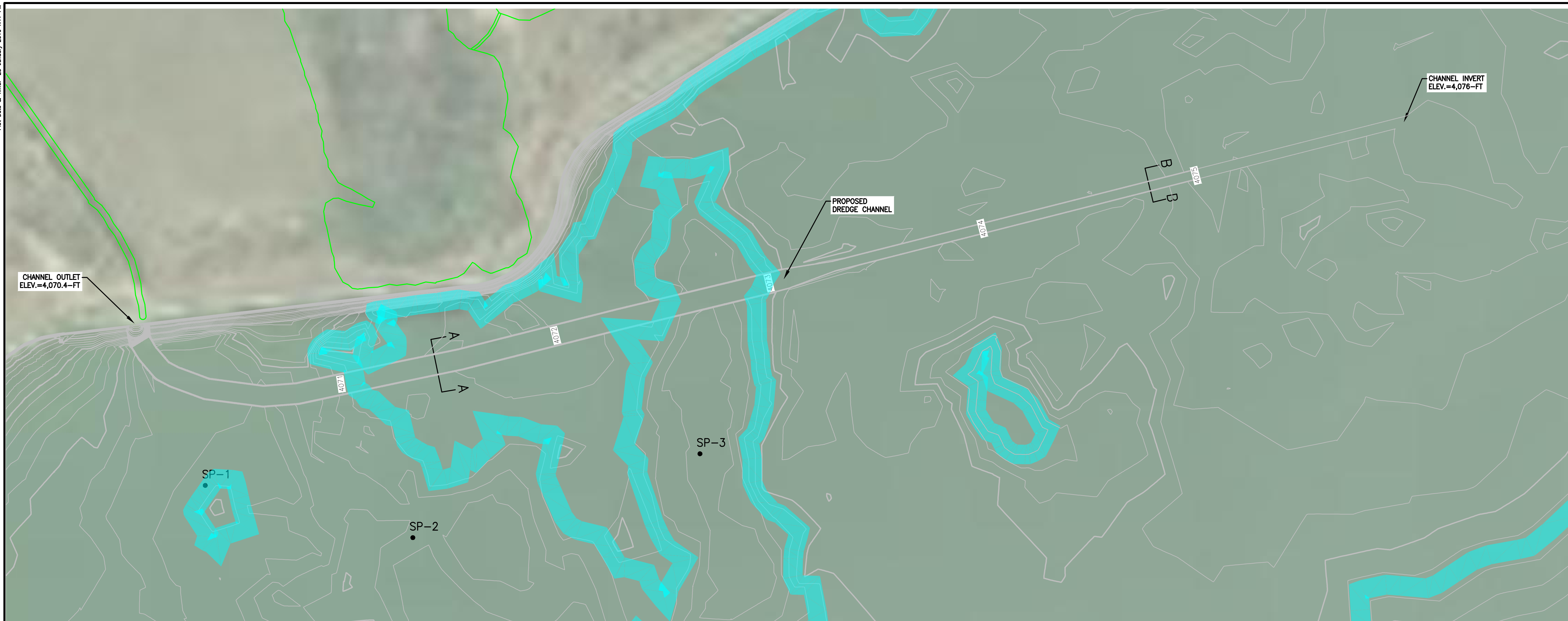
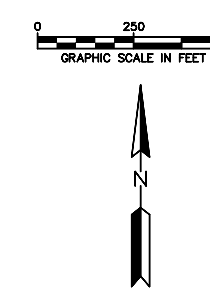
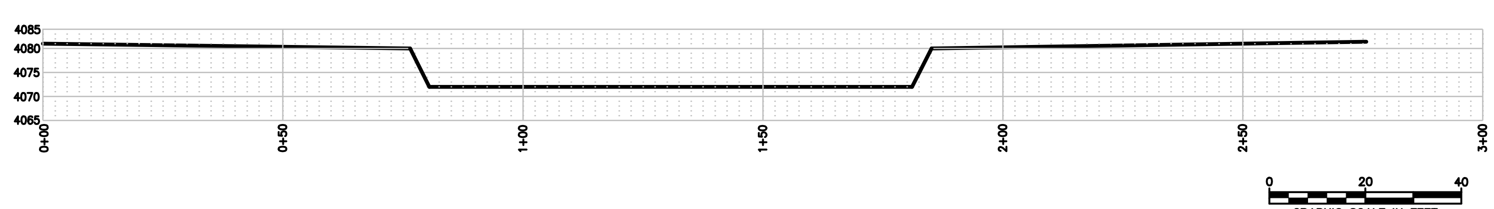
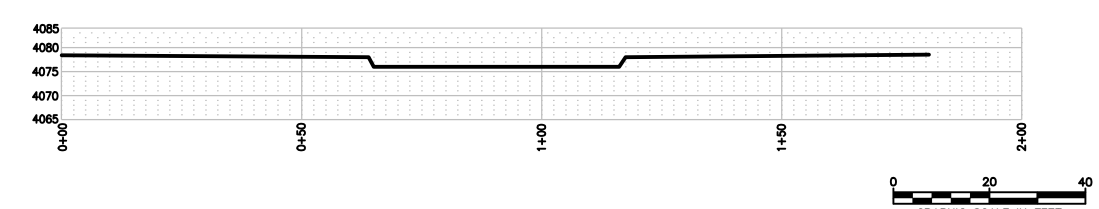




A–A



B–B



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REV	REVISION DESCRIPTION	DWN	APP	REV DATE

SEAL

SUB CONSULTANT

PRIME CONSULTANT

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PROJECT TITLE
PREWITT RESERVOIR REHABILITATION FEASIBILITY STUDY

PREWITT OPERATING COMMITTEE

SHEET TITLE
DREDGE CHANNEL PLAN

DWN BY	CHK'D	APP'D	DWG DATE	DEC. 2018
JVB	MJK	MJK	SCALE	AS NOTED

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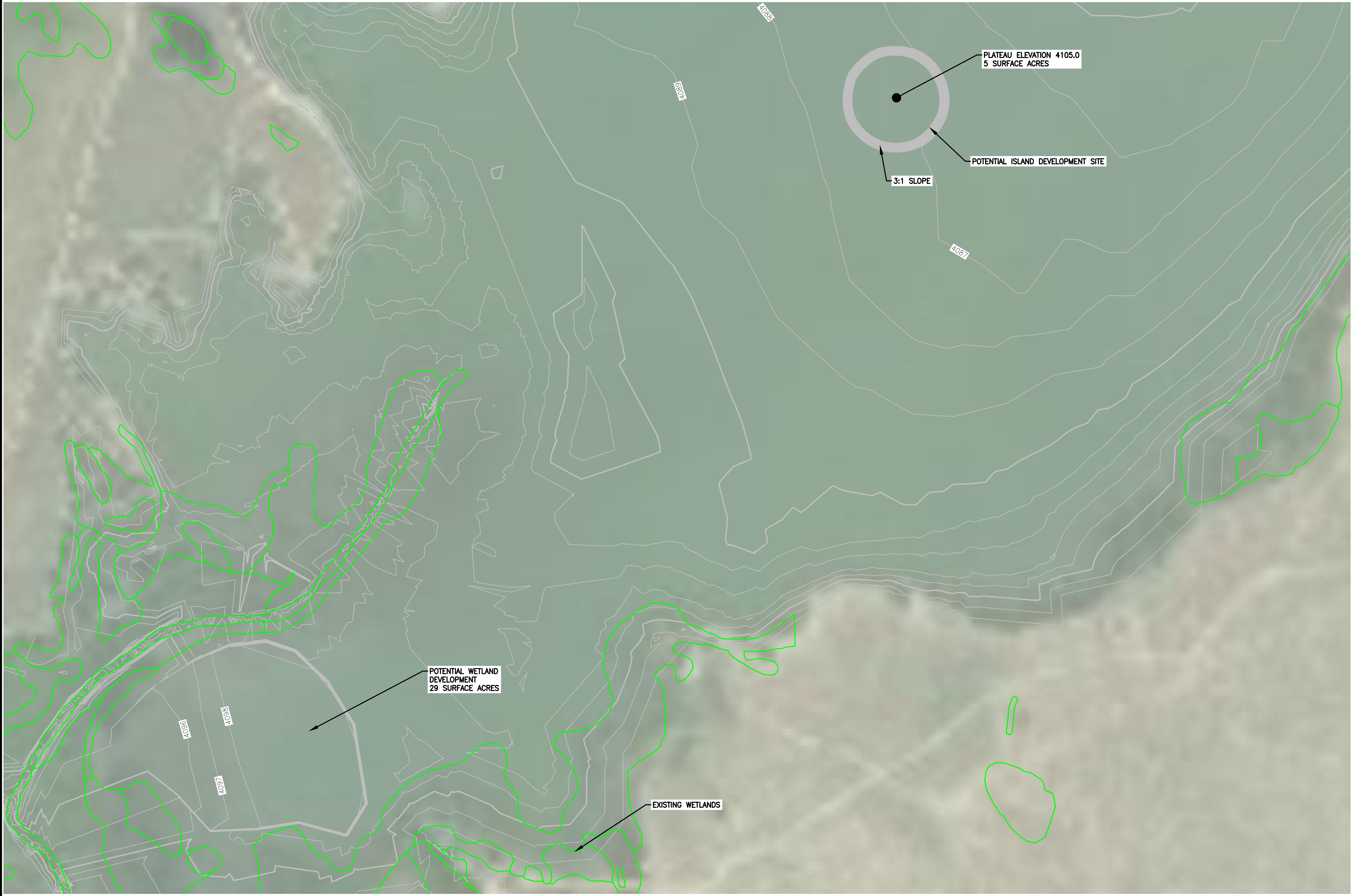

GENERAL NOTES

1. EXISTING WETLAND SHAPES OBTAINED FROM US FISH AND WILDLIFE SERVICE NATIONAL WETLAND INVENTORY. DELINEATIONS NOT COMPLETED.

N

0 300 600
GRAPHIC SCALE IN FEET

#	DESCRIPTION	XXX	XXX	XX/XX/XX
REV	REVISION DESCRIPTION	DWN	APP	REV DATE

SEAL

SUB CONSULTANT

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PROJECT TITLE
PREWITT RESERVOIR REHABILITATION FEASIBILITY STUDY

PREWITT OPERATING COMMITTEE

SHEET TITLE
POTENTIAL ISLAND AND WETLAND DEVELOPMENT SITES

DWN BY	CHK'D	APP'D	DWG DATE	DEC. 2018
JVB	MJK	MJK	SCALE	AS NOTED

PROJECT NO.	SHEET NO.	REV NO.
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Plot Date & Time:21 December 20181 PM

EXTENTS REQUIRING NO MITIGATION (0-FT PREDICTED OVERTOPPING)

EXTENTS REQUIRING NOMINAL MITIGATION (<1-FT PREDICTED OVERTOPPING)

EXTENTS REQUIRING MODERATE MITIGATION (1 TO 2-FT PREDICTED OVERTOPPING)

EXTENTS REQUIRING MAJOR MITIGATION (2+ FT PREDICTED OVERTOPPING)

N

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GRAPHIC SCALE IN FEET

REV	REVISION DESCRIPTION	DWN	APP	REV DATE

SEAL

SUB CONSULTANT

PRIME CONSULTANT

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PROJECT TITLE

PREWITT RESERVOIR REHABILITATION FEASIBILITY STUDY

PREWITT OPERATING COMMITTEE

SHEET TITLE

REMEDIAL EXTENTS

DWN BY	CHK'D	APP'D	DWG DATE	DEC. 2018
JVB	MJK	MJK	SCALE	AS NOTED

PROJECT NO.	SHEET NO.	REV NO.
7294-0003	C-104	0

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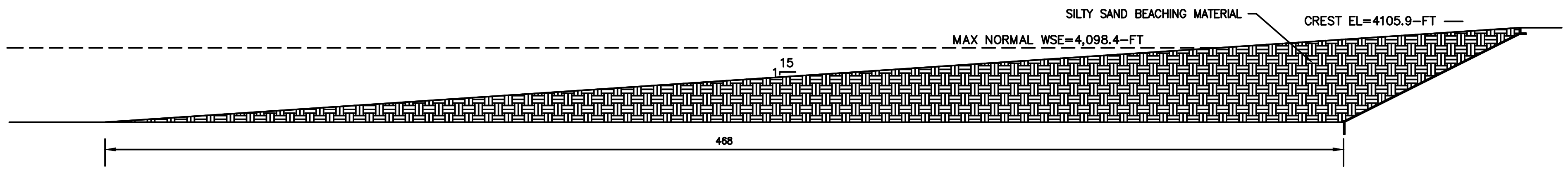
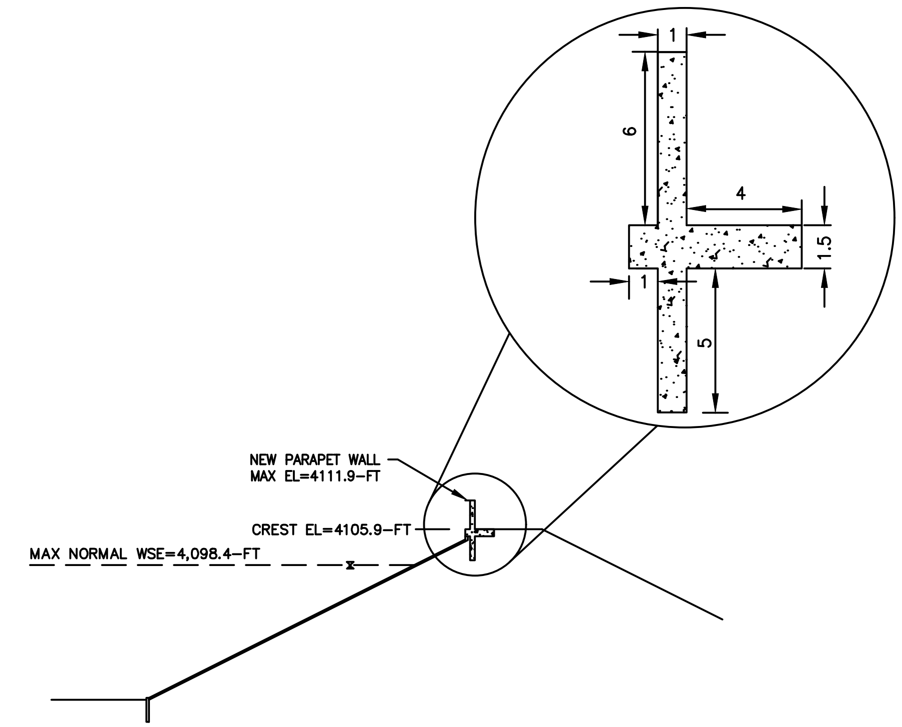
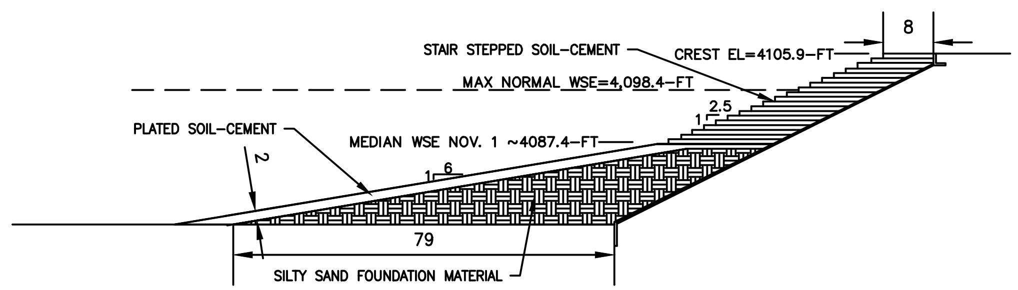


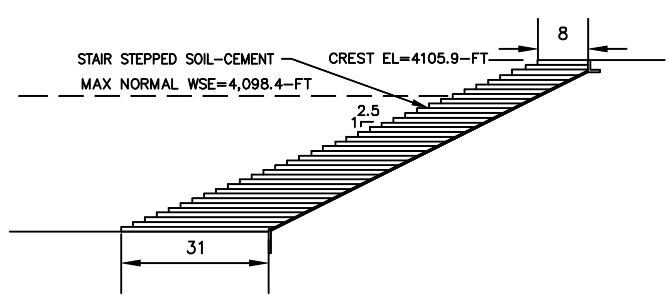
BEACHED MATERIAL SECTION 1 D-101



PARAPET EMBANKMENT SECTION 4 D-101



COMPOSITE SOIL CEMENT SECTION 2 D-101



STAIR-STEPPED SOIL CEMENT SECTION 3 D-101

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#	DESCRIPTION	XXX	XXX	XX/XX/XX

SEAL

SUB CONSULTANT

PRIME CONSULTANT

WENCK

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PROJECT TITLE
PREWITT RESERVOIR REHABILITATION FEASIBILITY STUDY

PREWITT OPERATING COMMITTEE

SHEET TITLE
WAVE MITIGATION ALTERNATIVES

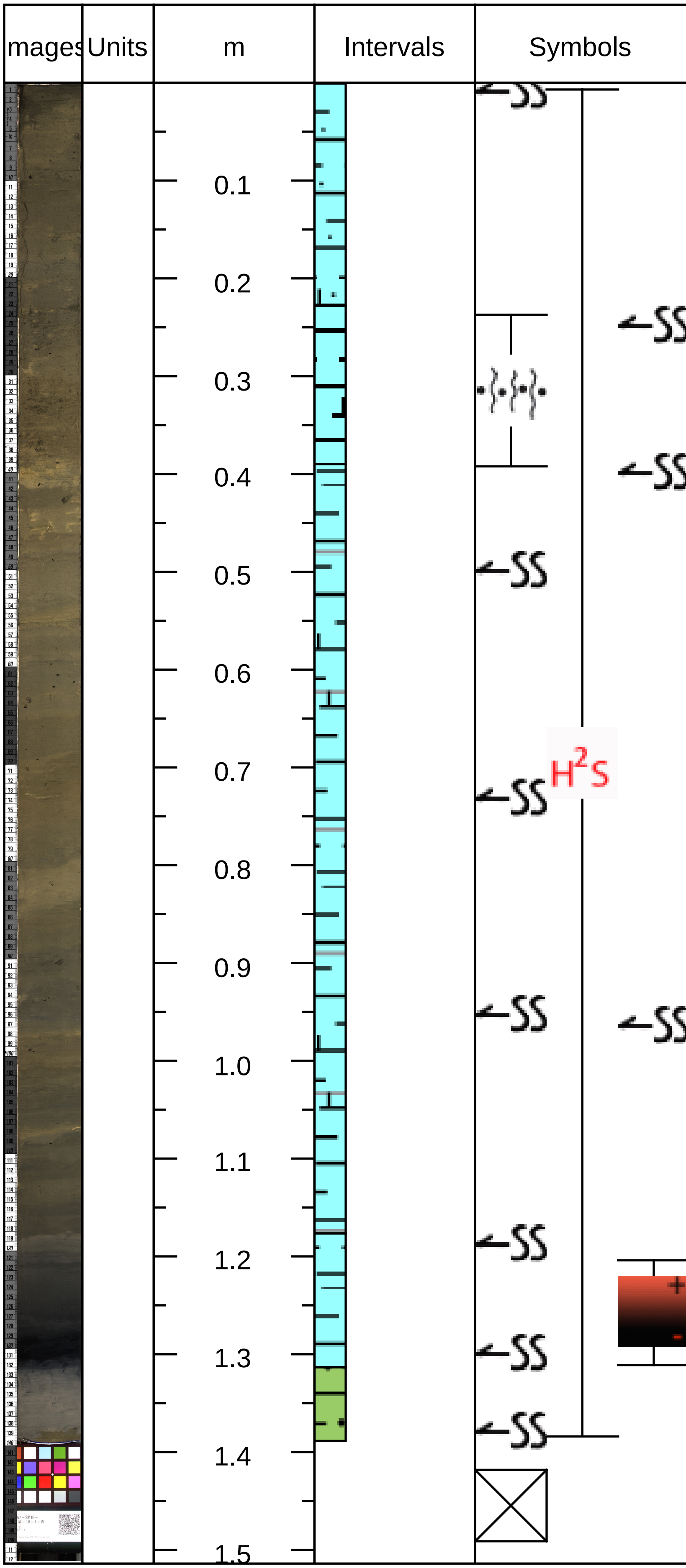
DWN BY	CHK'D	APP'D	DWG DATE	DEC. 2018
JVB	MJK	MJK	SCALE	AS NOTED

PROJECT NO.	SHEET NO.	REV NO.
7294-0003	D-101	0

Appendix C

LacCore Sediment Analysis

67-SP18-1A-1V-1-W



Description

0.0 m-0.23 m Lithostratigraphic Unit I. Slightly silty and diatomaceous carbonate mud with rare medium sand, rounded to sub-rounded. Mineral fraction is est 85% well-formed ellipsoidal and tabular carbonates, colorless to very faintly yellow, to 30 microns; twinning rare.

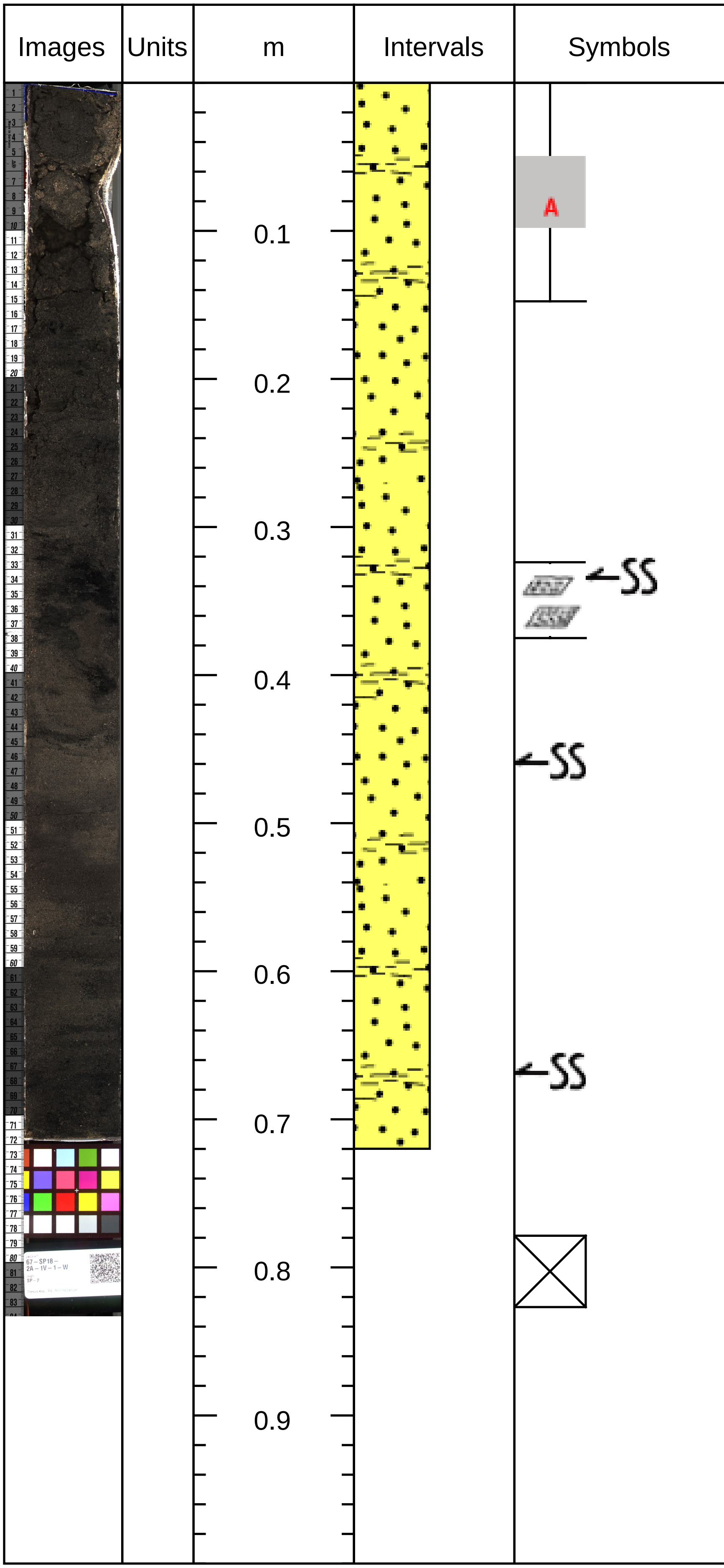
Pyrite framboids to 30 microns. Diverse diatom forms, rare Phacotus, and rare masses of cyanobacterial (?) pigments.

Sand fraction includes several rounded microcline grains (tartan twinning).

Mild to moderate coring disturbance in upper 30 cm, with core-wall liquefaction suggested.

0.008 m-1.386 m Moderately strong H2S pungency when unwrapped; core in a visibly

67-SP18-2A-1V-1-W



Description

0.0 m-0.72 m Crudely banded, silty, variably calcareous, rounded to sub-rounded medium sand with modal grain size 300-500 microns in smear slide (likely a component of coarse sand present). The smear slide from reduced material at 33.5 cm has abundant endogenic carbonate matrix in the 15-25 micron size range; the other two slides have sparse to rare carbonate component. There is a component of VERY well-rounded quartz and feldspar. Secondary components include a heavy mineral component: mafics, especially orthopyroxene (parallel extinction),probable amphiboles (hornblende), biotite, rare apatite, rare zircons, an isotropic high-relief unknown (possibly garnet), and possible magmatic Fe oxides (magnetite or titanomagnetite).

There is a rare but prominent component of conchoidal, sometimes vesilular, colorless volvcanic glass, very fresh-looking, up to 400 microns in dimension.

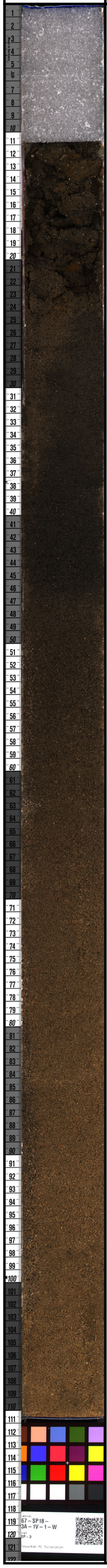
Biogenic material generally is very sparse. Very rare fragmentary plant debris; rare phytoliths. No diatoms observed.

The irregular, discontinuous, 'swirly' color variation here may possibly reflect disturbance from the vibracoring process.

0.001 m-0.15 m Sediment structure disturbed and liner distorted by coring process.
0.325 m-0.378 m Reduced sand in this

67-SP18-3A-1V-1-W

Images	Units	m	Intervals	Symbols	Description



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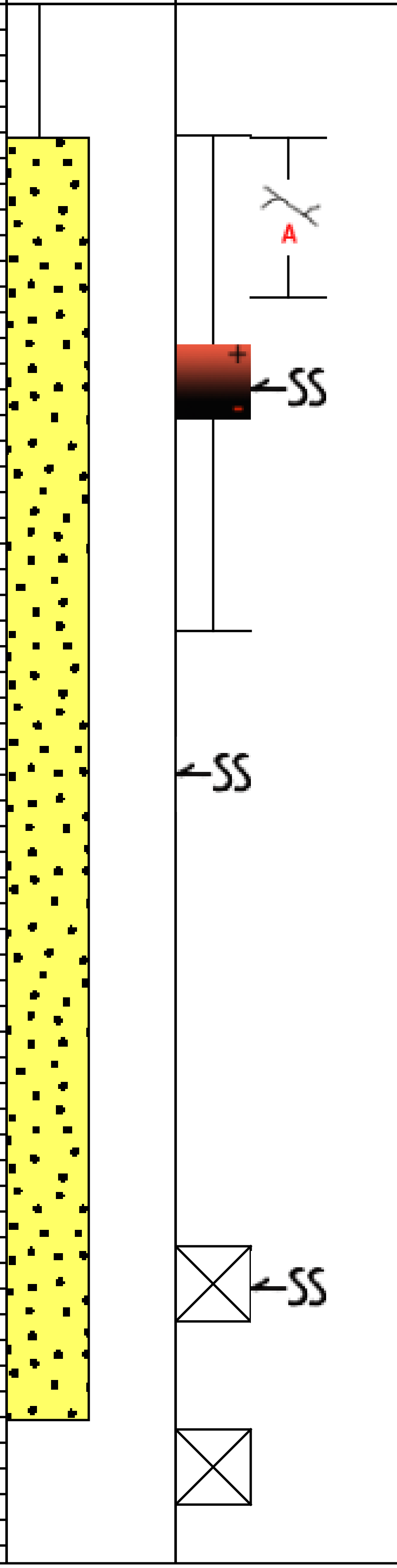


0.0 m-0.105 m

Foam stabilizer

0.105 m-0.23 m

Disturbed

0.105 m-1.102 m Massive, generally well-rounded, medium to coarse sand, to 600 microns. Carbonate-poor, with less silt-sized matrix overall than in 2A and carbonate a very subordinate component of the fines. As in 2A, sand fraction is well-rounded to sub-rounded, with a noticeable subset of quartz and feldspar sand grains with extreme spericity. Oxide coatings on sand grains are common, sometimes heavy. Some highly rutilated quartz grains.

Similar assemblage of heavy accessory minerals to 2A: mafics, especially orthopyroxene (parallel extinction),probable amphiboles (hornblende), biotite, rare zircons, isotropic unknown (garnet?) and possible magmatic Fe oxides (magnetite or titanomagnetite). Volcanic glass present, including one observed palagonitized fragment; otherwise looks fresh.

Biogenic components rare or very rare overall. No diatoms observed. Very rare fragmentary organic matter; rare phytoliths, including some apparently hematite-coated.

0.3 m Pyrite framboids present.

Appendix D

Geotechnical Analysis

GEOTECHNICAL ENGINEERING STUDY

FOR

PLACING DREDGED MATERIAL ON THE UPSTREAM SLOPE

OF

PREWITT RESERVOIR EMBANKMENT

STERLING, COLORADO

Job No.: 18-354A
December 14, 2018

Prepared for:

Wenck Associates
7000 Yellowstone Road, Suite 230
Cheyenne, Wyoming 82009

Hollingsworth Associates, Inc.
Geotechnical & Environmental Engineers 2875 W. Oxford Ave. #7 Sheridan, Colorado 80110
303-781-5188/fax 303-781-5224

TABLE OF CONTENTS

SUMMARY AND CONCLUSIONS

1. Three samples of the materials to be dredged from the reservoir were obtained in the field by Wenck personnel, designated as Samples SP-1, SP-2, and SP-3 and delivered to our laboratory for testing.

2. The initial laboratory testing indicated that Sample SP-1 is a clay (CH) with LL=83, PI=57, and minus #200 content=95%. Sample SP-2 is a silty sand(SM) with a minus #200 content =27% and is non-plastic. Sample SP-3 is a slightly silty sand(SP-SM) with a minus #200 content =11% and is non-plastic

3. It would be difficult to mix the clay with the sands; therefore, a composite sample of the potential dredged material was prepared by combining equal weights of Samples SP-2 and SP-3 and designated as Sample Composite A for the second step of laboratory testing. The results of the next step of laboratory testing indicated that Sample Composite A is a silty sand(SM), non-plastic, with a minus #200 content=17%, an optimum moisture content=12.1%, a maximum dry density=116.8 pcf, a cohesion=zero, and a phi=28 degrees.

4. The slope stability analyses conducted indicate that the dredged silty sands placed on the upstream slope of the existing embankment are stable at a slope of 2.5H:1V.

5. The dredged silty sands placed on the upstream slope of the existing embankment will be erodible when subjected to wave action and stormwater runoff. The Universal Soil Loss Equation may be used with an erodibility factor K=0.11 to quantify the amount of the soil loss.

PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical engineering study of placing silty sand materials dredged from the reservoir on the upstream slope of the existing Prewitt Reservoir Embankment in Sterling, Colorado. The geotechnical engineering study was conducted for the purpose of determining a stable slope for the material on the upstream slope and evaluating the erodibility of the placed material. The study was conducted in general accordance with our proposal P18-132A dated May 1, 2018. The task of evaluating the erodibility of the placed material was added after the study was started.

Three samples of the materials to be dredged from the reservoir were obtained in the field by Wenck personnel and delivered to our laboratory for testing. The results of the laboratory testing were analyzed to develop recommendations for a stable slope and an erodibility factor for the dredged soils when placed on the upstream slope of the embankment. The results of the laboratory testing are presented herein.

This report has been prepared to summarize the data obtained during this study and to present our conclusions and recommendations based on the proposed construction and the properties of the dredged materials. Design parameters and a discussion of geotechnical engineering considerations related to placement of the dredged materials on the upstream slope of the existing embankment are included in the report.

PROPOSED CONSTRUCTION

It is our understanding that the sediments in the reservoir will be dredged to increase the storage capacity of the reservoir. Beneficial use of the dredged materials is under study.

FIELD SAMPLING

Three large bulk samples considered representative of the range of materials expected to be dredged from the reservoir were obtained in the field Wenck personnel and delivered to our laboratory for laboratory testing. The samples were designated SP-1, SP-2, and SP-3.

LABORATORY TESTING

<u>Initial Laboratory Testing</u>: The submitted samples, SP-1, SP-2, and SP-3, were examined and visually classified in the laboratory by the principal engineer. Laboratory testing included standard property tests, such as natural moisture content (ASTM D-2216), grain size analysis (ASTM D-422),

and liquid and plastic limits (ASTM D-4318). Sample SP-1 is a clay (CH) with LL=83, PI=57 and minus #200 content=95%. Sample SP-2 is a silty sand(SM) with minus #200 content=27% and is non-plastic. Sample SP-3 is a slightly silty sand (SP-SM) with minus #200 content=11% and is non-plastic. Results of the laboratory testing are shown on Figs. 1 and 2 and summarized in Table I. The laboratory testing was conducted in general accordance with applicable ASTM standards.

Second Step in Laboratory Testing: We understand that it will be possible to select the material that will be placed on the embankment during dredging. It would be difficult to mix the clay with the sands; therefore, a composite sample of the potential dredged material was prepared by combining equal weights of Samples SP-2 and SP-3 and designated as Sample Composite A. The gradation(ASTM D-698), liquid and plastic limits (ASTM D-4318), Standard Proctor maximum dry density and optimum moisture content(ASTM D-698) and remolded direct shear strength(ASTM D-3080) were determined for the composite sample. Sample Composite A is a silty sand (SM), non-plastic, with a minus #200=17%, a minus #8=100%, an optimum moisture content=12.1%, a maximum dry density=116.8 pcf, a cohesion=zero and a phi=28 degrees. Results of the laboratory testing are shown on Figs. 1 through 4 and summarized in Table I. The laboratory testing was conducted in general accordance with applicable ASTM standards.

STABILITY OF PLACEMENT SLOPE

Stability analyses of the placement were performed using the Janbu method and Bishops modified method of slices. The existing embankment geometry was provided by the client. The upstream slope of the existing embankment is 2 horizontal to 1 vertical, the existing embankment crest width is 13.5 feet and the existing embankment height is 37 feet. The strength of the existing embankment was unknown and it was desired to force the failure surface through the dredged Composite A material so the existing embankment was modeled as homogeneous and impenetrable.

The dredged Composite A material was modeled as a layer at least 6 feet thick placed on the upstream face of the existing embankment at slopes of 2H:1V and 2.5H:1V. The dredged material is a cohesionless silty sand with a friction angle of 28° and a unit weight of 120 pcf. The upstream slope of the existing embankment with the layer of dredged material was analyzed under the end of construction condition using a total stress analysis. The material properties used in the analysis are shown on Fig. 5. The computer program Slope W was used with a search engine to find the failure planes with the lowest factor of safety. The 2.5H:1V slope had a minimum factor of safety of at least 1.3 which is considered stable. The factors of safety and the critical failure surfaces for the 2.5H:1V slope are shown on Fig. 5 and indicate that the layer of dredged Composite A material is stable at a 2.5H:1V slope and at least 6 feet in thickness.

ERODIBILITY OF THE SILTY SAND

Based on the gradation of the silty sand, sample Composite A, and experience, the dredged silty sands will be erodible when placed on the upstream slope of the Prewitt Reservoir Embankment and subjected to wave action and stormwater runoff. The Universal Soil Loss Equation may be used to quantify the soil loss. That equation uses a soil erodibility factor, K. A value for K was estimated for the silty sands of 0.11 from Figure 30.-The soil erodibility nomograph on page 36 of the Technical Publication SA-TP 11, A Guide for Predicting Sheet and Rill Erosion on Forest Land, United States Department of Agriculture.

LIMITATIONS

This report has been prepared in accordance with generally accepted geotechnical engineering practices in this area for use by the client for design purposes. The conclusions and recommendations submitted in this report are based upon the data obtained from the laboratory testing and the proposed type of construction. The nature and extent of the variations in the dredged materials may not become evident until construction is performed. If during construction, properties of the dredged materials appear to be different from those described herein, this office should be advised so reevaluation of the recommendations may be made.

Sincerely,
HOLLINGSWORTH ASSOCIATES, INC.

Casey McManus, Staff Geologist

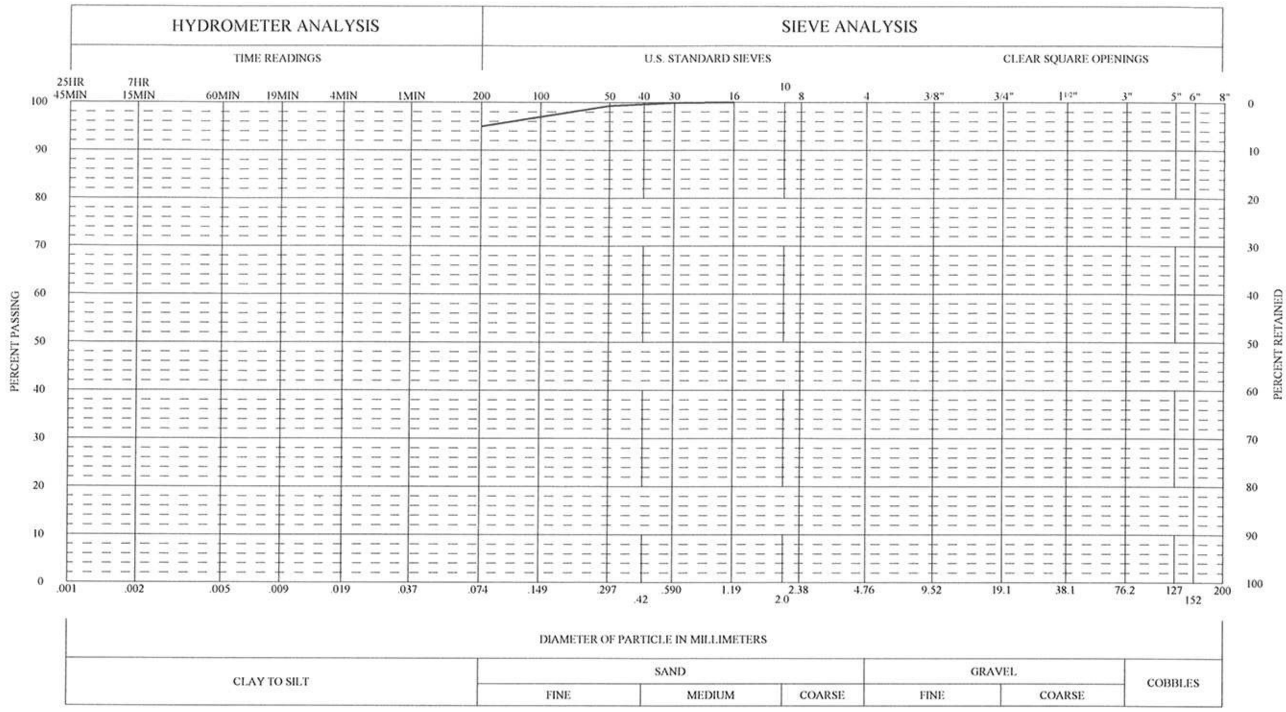
Harold Hollingsworth, P.E.

COLORADO REGISTERED PROFESSIONAL ENGINEER
HAROLD HOLLINGSWORTH JR.
9924
12/19/12

HH: cm

Reviewed by: TRH

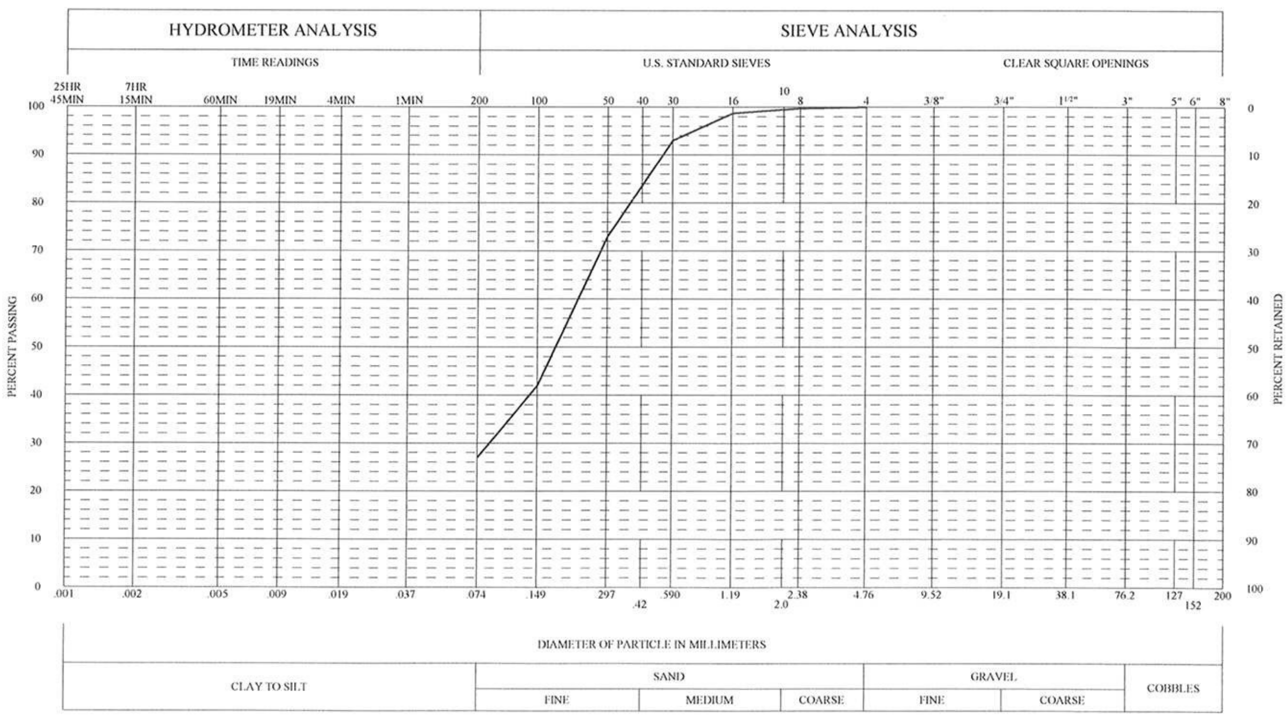
Attachments



GRAVEL 0 % SAND 5 % SILT AND CLAY 95 %

LIQUID LIMIT 83 % PLASTICITY INDEX 57 %

SAMPLE OF Clay FROM SP-1

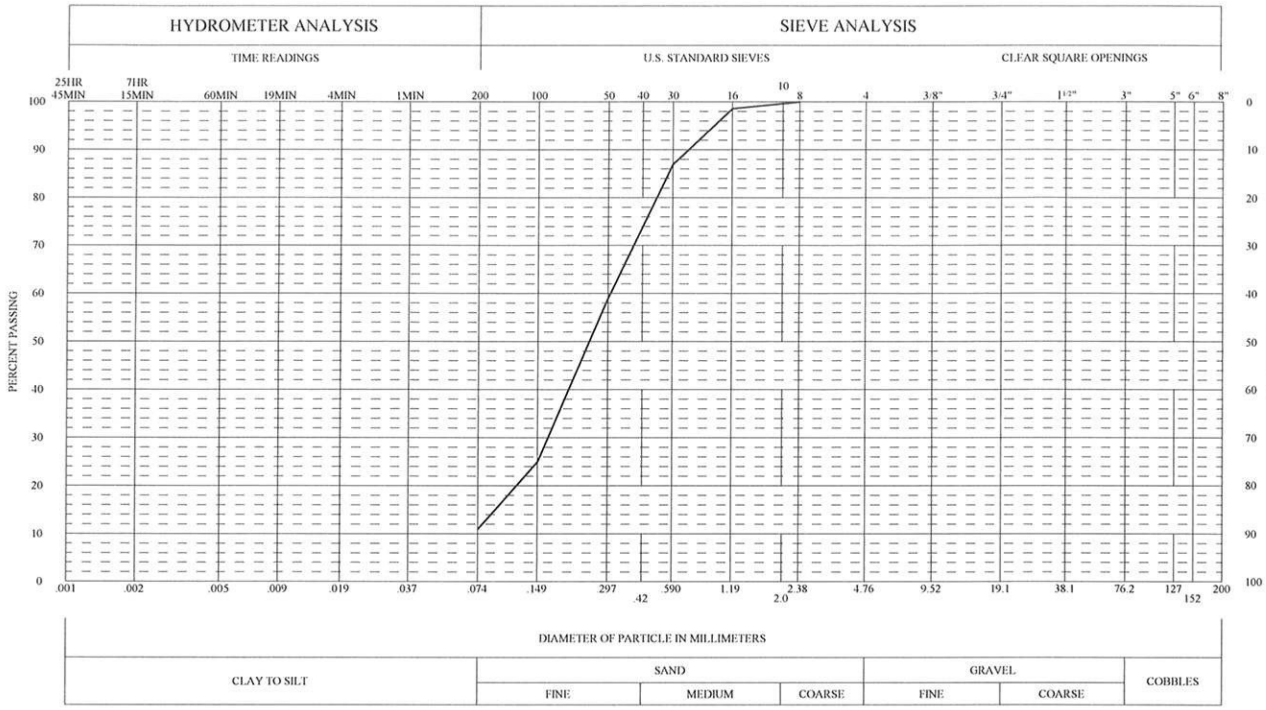


GRAVEL 0 % SAND 73 % SILT AND CLAY 27 %

LIQUID LIMIT % PLASTICITY INDEX NP %

SAMPLE OF Silty sand FROM SP-2

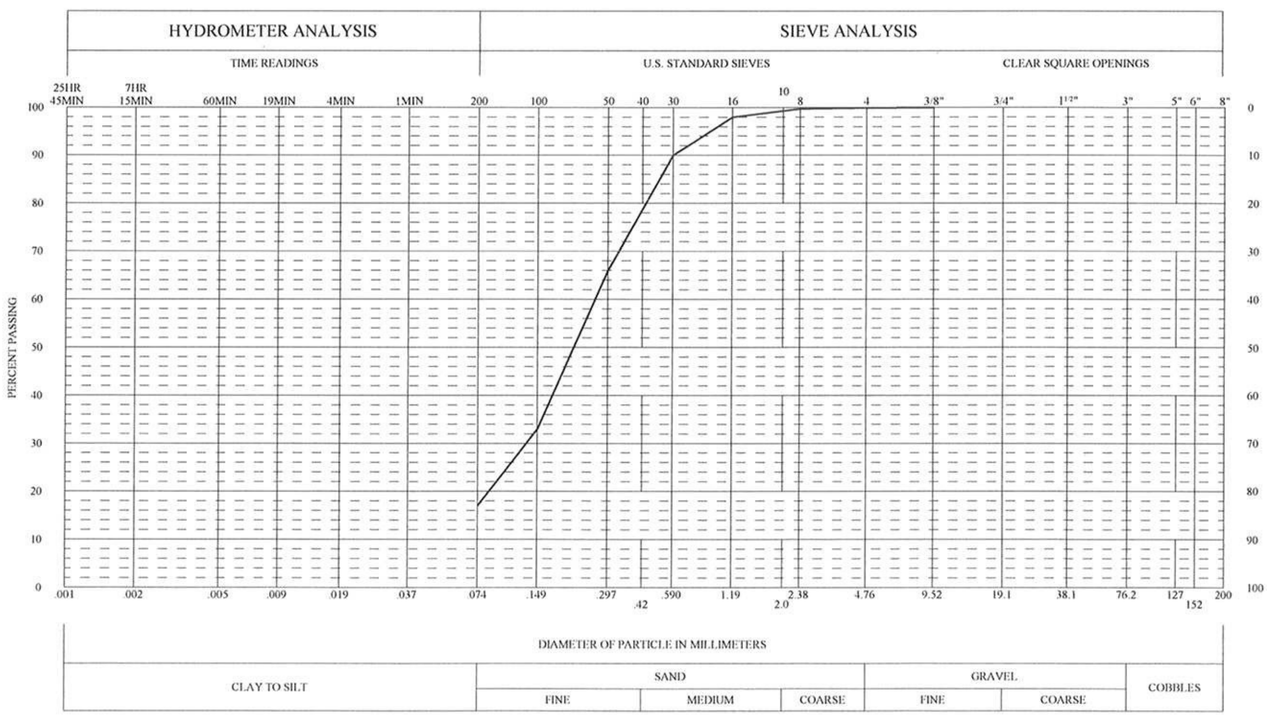
18-354A	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Gradation Test Results	Fig. 1



GRAVEL 0 % SAND 89 % SILT AND CLAY 11 %

LIQUID LIMIT % PLASTICITY INDEX NP %

SAMPLE OF Slightly silty sand FROM SP-3

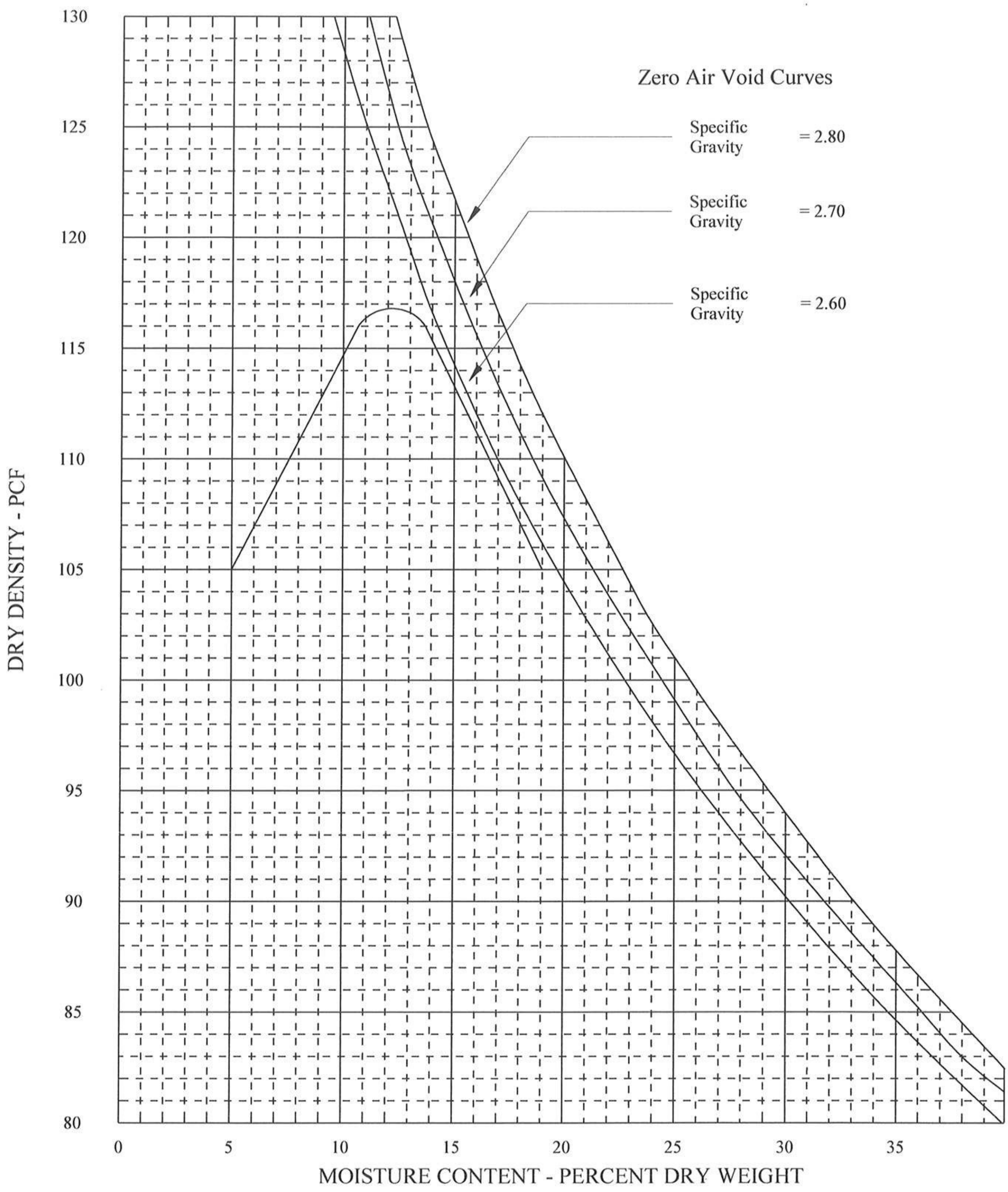


GRAVEL 0 % SAND 83 % SILT AND CLAY 17 %

LIQUID LIMIT % PLASTICITY INDEX NP %

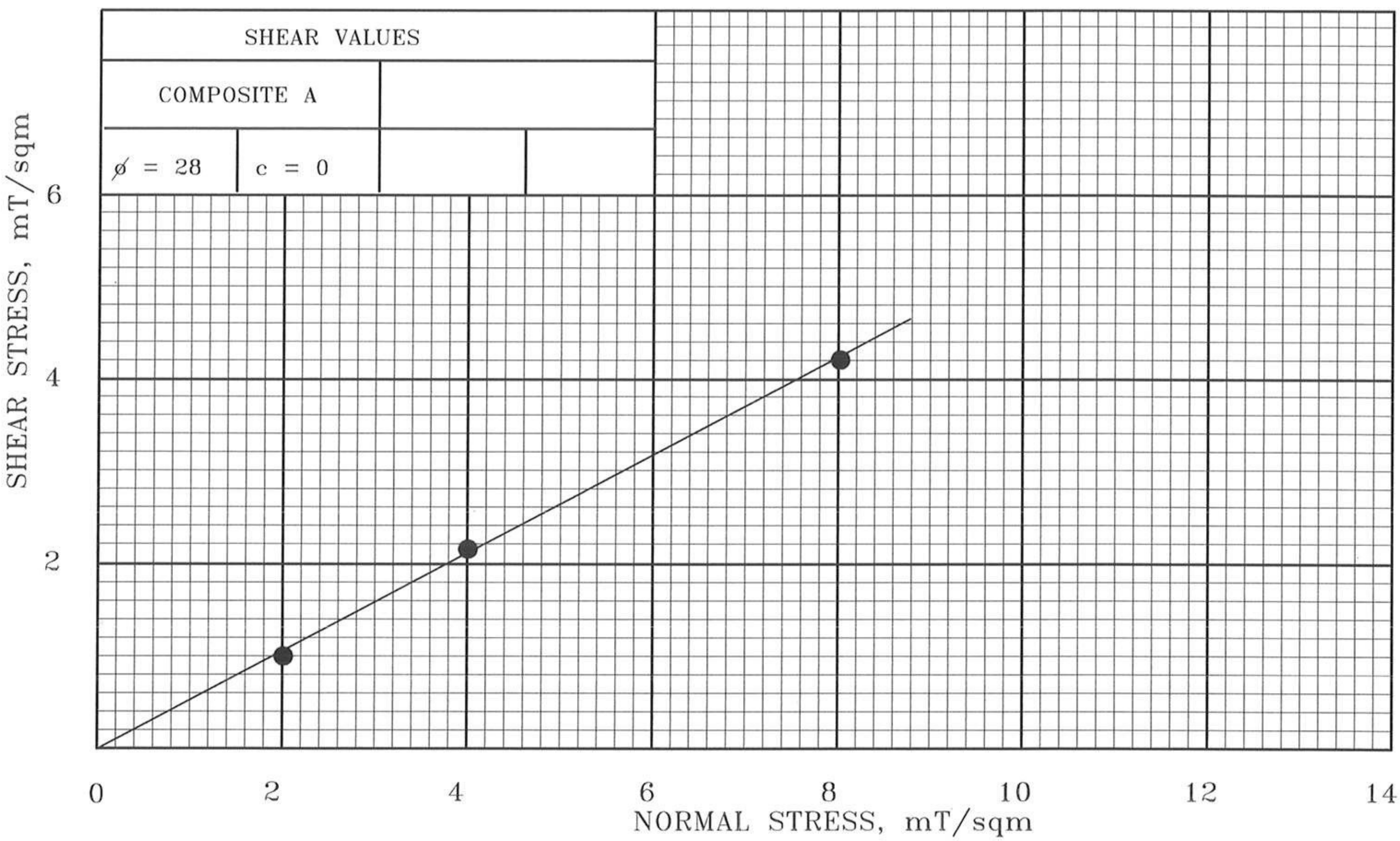
SAMPLE OF Silty sand FROM Composite A

18-354A	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Gradation Test Results	Fig. 2



Location: Prewitt Reservoir	Max Dry Density: 116.8 pcf
Sample: Composite A	Opt. Moist. Content: 12.1 %
Sample Description: Silty sand	Gravel 0 % Sand 83 % Silt and Clay 17 %
Procedure: ASTM D698, A	Liquid Limit: % Plasticity Index: NP %

18-354A	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Moisture-Density Relationships	Fig. 3

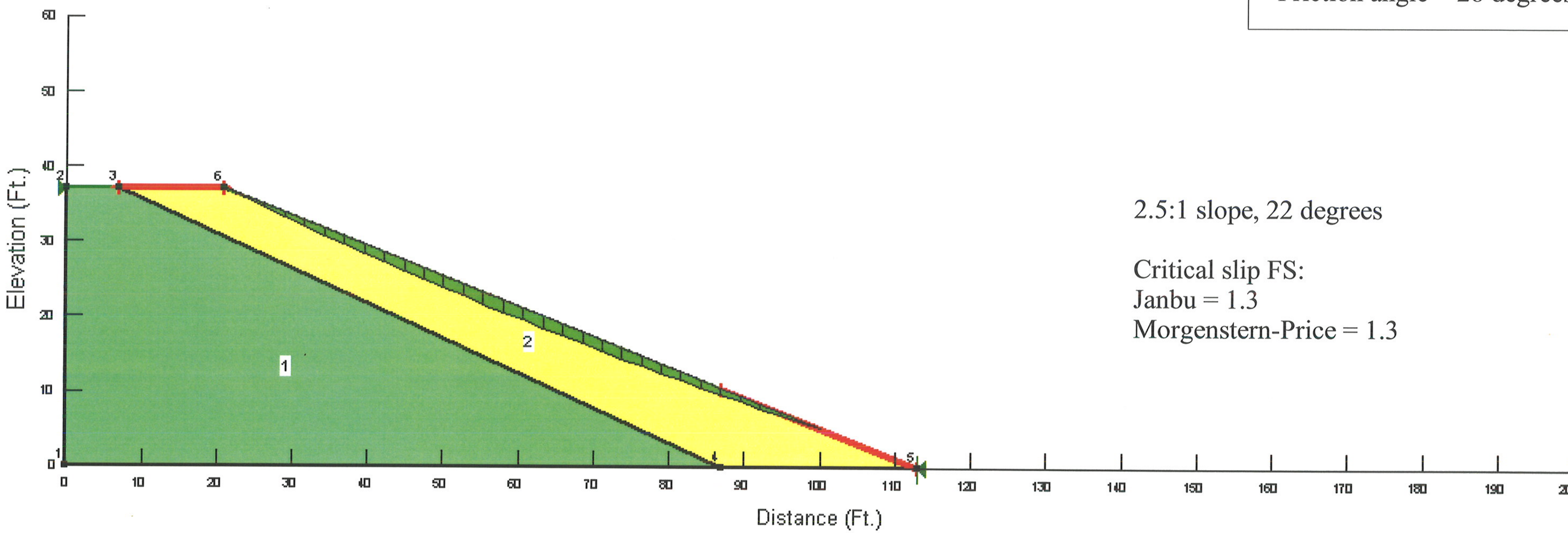


18-354A	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Direct Shear Test Results	Fig. 4

End of Construction

Soil 1, Existing embankment:
Modeled as impenetrable

Soil 2, Silty sand:
Unit weight = 120 pcf
Cohesionless
Friction angle = 28 degrees



Not to Scale

Hollingsworth Associates, Inc.

Job No.: 18-354A

TABLE I
SUMMARY OF LABORATORY TEST RESULTS

Sample Designation	Moisture Content (%)	Gradation			Atterberg Limits		Proctor		Direct Shear		Soil Erodibility Factor (K)	Soil Type
		Gravel + No. 4 (%)	Sand - No. 4 + No. 200 (%)	Minus No. 200 Sieve (%)	Liquid Limit (%)	Plasticity Index (%)	Optimum Moisture Content (%)	Maximum Dry Density (pcf)	Cohesion C (ksf)	Friction Angle (degrees)		
SP-1	93.0	0	5	95	95	83						Clay
SP-2	23.1	0	73	27		NP						Silty sand
SP-3	18.9	0	89	11		NP						Slightly silty sand
Composite A		0	83	17		NP	12.1	116.8	0	28	0.11	Silty sand
SP-2												
SP-3												

LABORATORY MIX DESIGN STUDY

FOR

THE PROPOSED SOIL-CEMENT UPSTREAM SLOPE PROTECTION

PREWITT RESERVOIR EMBANKMENT

STERLING, COLORADO

Job No.: 18-354B
December 6, 2018

Prepared for:

Wenck Associates
7000 Yellowtail Road, Suite 230
Cheyenne, Wyoming 82009

Hollingsworth Associates, Inc.

Geotechnical & Environmental Engineers 2875 W. Oxford Ave. #7 Sheridan, Colorado 80110
303-781-5188/fax 303-781-5224

TABLE OF CONTENTS

SUMMARY AND CONCLUSIONS

1. Although the Bonny criteria was not met for the minimum compressive strength with any of the cement contents tested, the cement content of 11% met the Bonny criteria for both wet-dry and freeze-thaw weight loss which are considered the durability factors and more significant than the compressive strength. A cement content of at least 11% is recommended with the TP-1 sand as the aggregate for the soil-cement upstream slope protection for the Prewitt Reservoir embankment

2. A bulk sample of the planned soil-cement aggregate for the soil-cement upstream slope protection for the Prewitt Reservoir embankment was obtained by Wenck personnel, identified TP-1, and delivered to our laboratory. The aggregate is a non-plastic sand with a minus No. 200 of 4%, an optimum moisture content of 11.2% and a maximum dry density of 105.0 pcf (ASTM D-698, A) and classifies as A-3 in accordance with the AASHTO soil classification system.

3. The average cement content, 9%, required for the soil-cement aggregate, TP-1, was selected using the procedures in the Portland Cement Association's Soil-Cement Laboratory Handbook. The aggregate mixed with 9% cement had an optimum moisture content of 11.1% and a maximum dry density of 111.4 pcf (ASTM D-558, A).

4. Soil-cement specimens with cement contents of 7%, 9%, and 11% by dry weight were prepared and tested.

5. For a cement content of 7%, the compressive strengths were 75 psi at 2 days, 149 psi at 7 days, 164 psi at 14 days, and 229 psi at 28 days. For a cement content of 9%, the compressive strengths were 111 psi at 2 days, 304 psi at 7 days, 390 psi at 14 days, and 487 psi at 28 days. For a cement content of 11%, the compressive strengths were 215 psi at 2 days, 348 psi at 7 days, 475 psi at 14 days, and 594 psi at 28 days.

6. For a cement content of 7%, the wet-dry weight loss was 18.7%. For a cement content of 9%, the wet-dry weight loss was 7.6%. For a cement content of 11%, the wet-dry weight loss was 1.2%, which meets the Bonny criteria for wet-dry weight loss.

7. For a cement content of 7%, the freeze-thaw weight loss was 4.5%. For a cement content of 9%, the freeze-thaw weight loss was 3.7%. For a cement content of 11%, the freeze-thaw weight loss was 2.0%. The Bonny criteria for freeze-thaw weight loss was met by all three cement contents.

PURPOSE AND SCOPE OF WORK

This report presents the results of a laboratory mix design study for the proposed soil-cement upstream protection for the Prewitt Reservoir embankment in Sterling, Colorado. The laboratory study was conducted for the purpose of developing recommendations for a cement content for the soil-cement slope protection. The study was conducted in accordance with our proposal P18-132 dated April 25, 2018.

A sample of the planned soil-cement aggregate was obtained in the field by Wenck personnel and delivered to our laboratory for testing. The laboratory testing program followed the procedures in the Portland Cement Association's Soil-Cement Laboratory Handbook and the applicable ASTM standards. The results of the laboratory testing were analyzed to develop a recommended cement content for the soil-cement slope protection. The results of the laboratory testing are presented herein.

This report has been prepared to summarize the data obtained during this study and to present our conclusions and recommendations including a cement content for the soil-cement.

PROPOSED CONSTRUCTION

It is our understanding that a soil-cement slope protection is being considered to protect the existing Prewitt Reservoir from erosion caused by wave action.

LABORATORY TESTING

Aggregate: A bulk sample of the planned soil-cement aggregate, identified as TP-1, was obtained by Wenck personnel and delivered to our laboratory. The sample was examined and visually classified in the laboratory by the principal engineer. Laboratory testing included standard property tests, such as natural moisture content (ASTM D-2216), dry unit weight, grain size analysis (ASTM D-422), and liquid and plastic limits (ASTM D-4318). The moisture-density relationships for the sample were determined in accordance standard Proctor (ASTM D-698, A) compaction procedures. The aggregate has a minus No. 200 of 4% and an optimum moisture content of 11.2% and a maximum dry density of 105.0 pcf (ASTM D-698). Results of the laboratory testing are shown on Figs. 1 and 2 and summarized in Table I. The aggregate classifies as A-3 in accordance with the AASHTO soil classification system.

<u>Selection of Average Cement Content</u>: The average cement content required for the soil-cement aggregate was selected by the STEP-BY-STEP PROCEDURE on page 11 of the <u>Portland Cement Association's Soil-Cement Laboratory Handbook</u>. Following that procedure, an average cement content of 9% was selected for further testing. The aggregate mixed with 9% cement had an optimum moisture content of 11.1% and a maximum dry density of 111.4 pcf (ASTM D-558, A). Results of the laboratory testing are shown on Fig. 3 and summarized in Table I.

<u>Cement Contents Evaluated</u>: Soil-cement specimens with cement contents of 7%, 9%, and 11% by dry weight were prepared and tested.

<u>Compressive Strength Test Specimens</u>: Twelve compressive test specimens were prepared using the sand from TP-1 and cement contents of 7%, 9%, and 11% by dry weight in 4-inch diameter molds. The four test specimens with a 7% cement content were identified as Cylinder Number 4A through 4D. The four test specimens with a 9% cement content were identified as Cylinder Number 5A through 5D. The four test specimens with an 11% cement content were identified as Cylinder Number 6A through 6D. The moisture content and dry density of the test specimens are given in Table II. The test specimens were then cured in a controlled moisture chamber at 100% humidity. The compressive strength of one test specimen at each cement content was determined at age 2 days, 7 days, 14 days, and 28 days, in accordance with the procedures of ASTM D-1633. The

compressive strengths of the test specimens are given on Figs. 4 and 5 and summarized in Table II.

Wet-Dry Test Specimens: Three wet-dry test specimens were prepared using the sand from TP-1 and cement contents of 7%, 9%, and 11% by dry weight in 4-inch diameter molds. The test specimen with a 7% cement content was identified as Cylinder Number 1. The test specimen with a 9% cement content was identified as Cylinder Number 2. The test specimen with an 11% cement content was identified as Cylinder Number 3. The moisture content and dry density of the test specimens are given in Table III. The test specimens were then cured in a controlled moisture chamber at 100% humidity for 7 days. The percent weight loss was determined for the test specimens in accordance with the procedures of ASTM D-559. The measurements made for the testing are given in Table IA in Appendix A. The percent weight loss of the test specimens are given on Figs. 6 and 7 and in Table III.

Freeze-Thaw Test Specimens: Three freeze-thaw test specimens were prepared using the sand from TP-1 and cement contents of 7%, 9%, and 11% by dry weight in 4-inch diameter molds. The test specimen with 7% cement content was identified as Cylinder Number 7. The test specimen with 9% cement content was identified as Cylinder Number 8. The test specimen with an 11% cement content was identified as Cylinder Number 9. The moisture content and dry density of the test specimens are given in Table IV. The test specimens were then cured in a controlled moisture chamber at 100% humidity for 7 days. The percent weight loss was determined for the test specimens in accordance with the procedures of ASTM D-560. The measurements made for the testing are given in Table IIA in Appendix A. The percent weight loss of the test specimens are given on Fig. 8 and in Table IV.

Discussion of Laboratory Test Results: The Bonny criteria for the cement content for soil-cement protection for wave action are: (1) a minimum compressive strength of 675 psi at 7 days, (2) a minimum compressive strength of 1,000 psi at 28 days, (3) a wet-dry weight loss of less than 6% after 12 cycles, and (4) a freeze-thaw weight loss of less than 6% after 12 cycles. The maximum compressive strengths achieved were a 7-day compressive strength of 348 psi and a 28-day compressive strength of 594 psi for the cement content of 11%. Thus, none of the cement contents tested met the Bonny criteria for minimum compressive strength. The minimum wet-dry weight loss was 1.2% for the cement content of 11%, which was the only cement content that met the Bonny criteria for wet-dry weight loss. The freeze-thaw weight loss for all three cement contents met the Bonny criteria.

CONCLUSIONS

Although the Bonny criteria was not met for the minimum compressive strength for any of the cement contents tested, the cement content of 11% met the Bonny criteria for both wet-dry and freeze-thaw weight loss which are considered the durability factors and more significant than the compressive strength. The cement content of at least 11% is recommended with the TP-1 sand for the soil-cement upstream slope protection for the Prewitt Reservoir embankment.

If we can provide additional information concerning the soil-cement, please call.

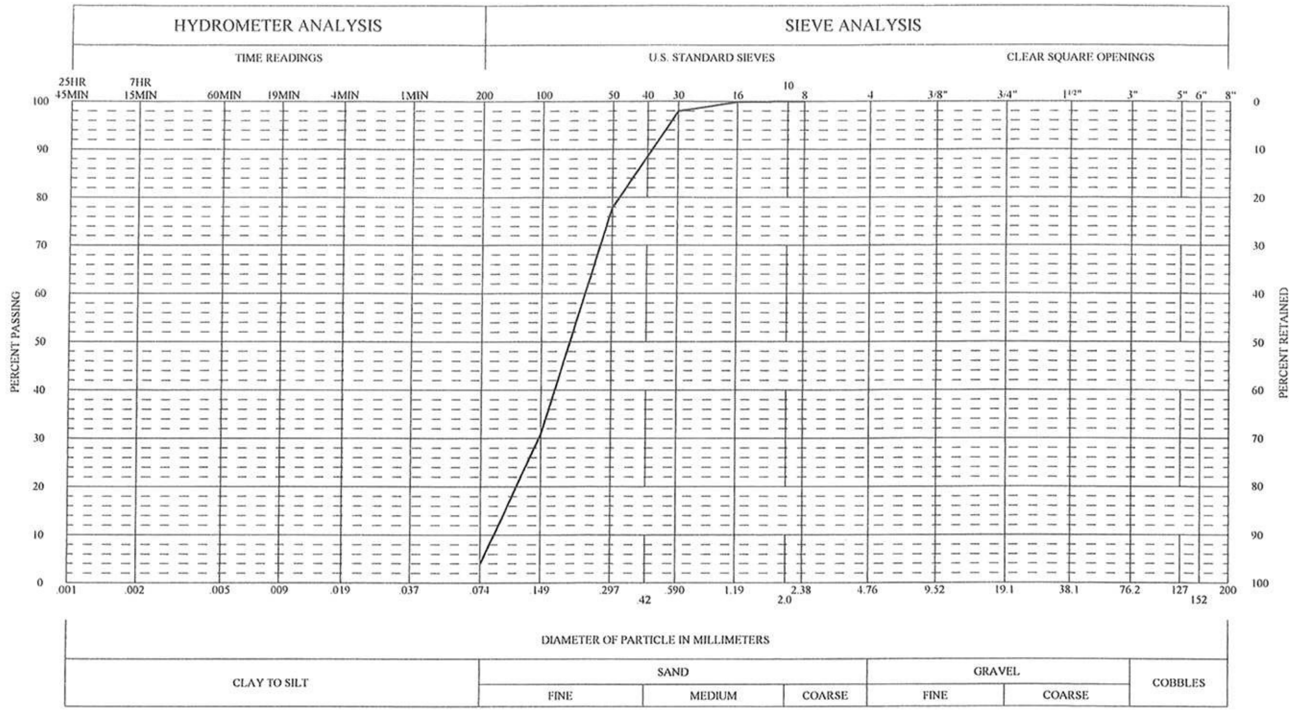
Sincerely,
HOLLINGSWORTH ASSOCIATES, INC.

Casey McManus, Staff Geologist

Harold Hollingsworth, P.E.

COLORADO REGISTERED
HAROLD HOLLINGSWORTH, JR.
9921
12/6/17
PROFESSIONAL ENGINEER

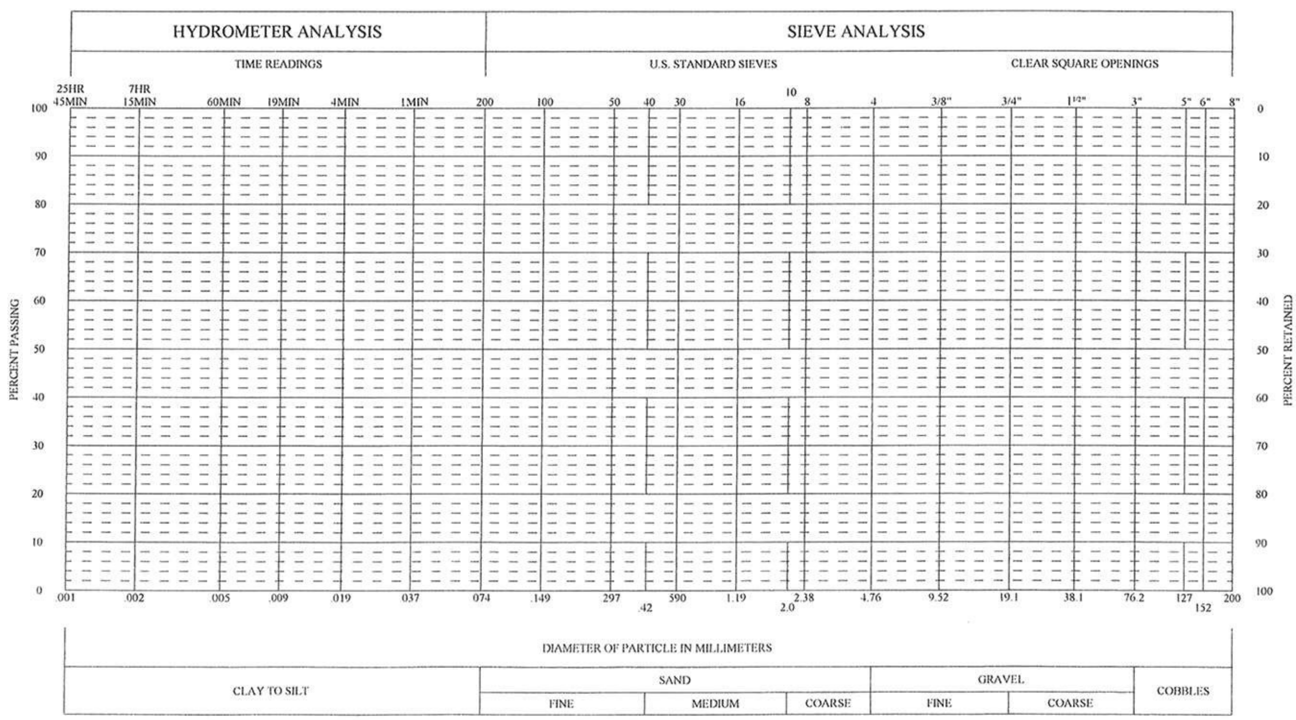
HH: cm
Reviewed by: TRH
Attachments



GRAVEL 0 % SAND 96 % SILT AND CLAY 4 %

LIQUID LIMIT % PLASTICITY INDEX NP %

SAMPLE OF Sand FROM Test Pit TP-1

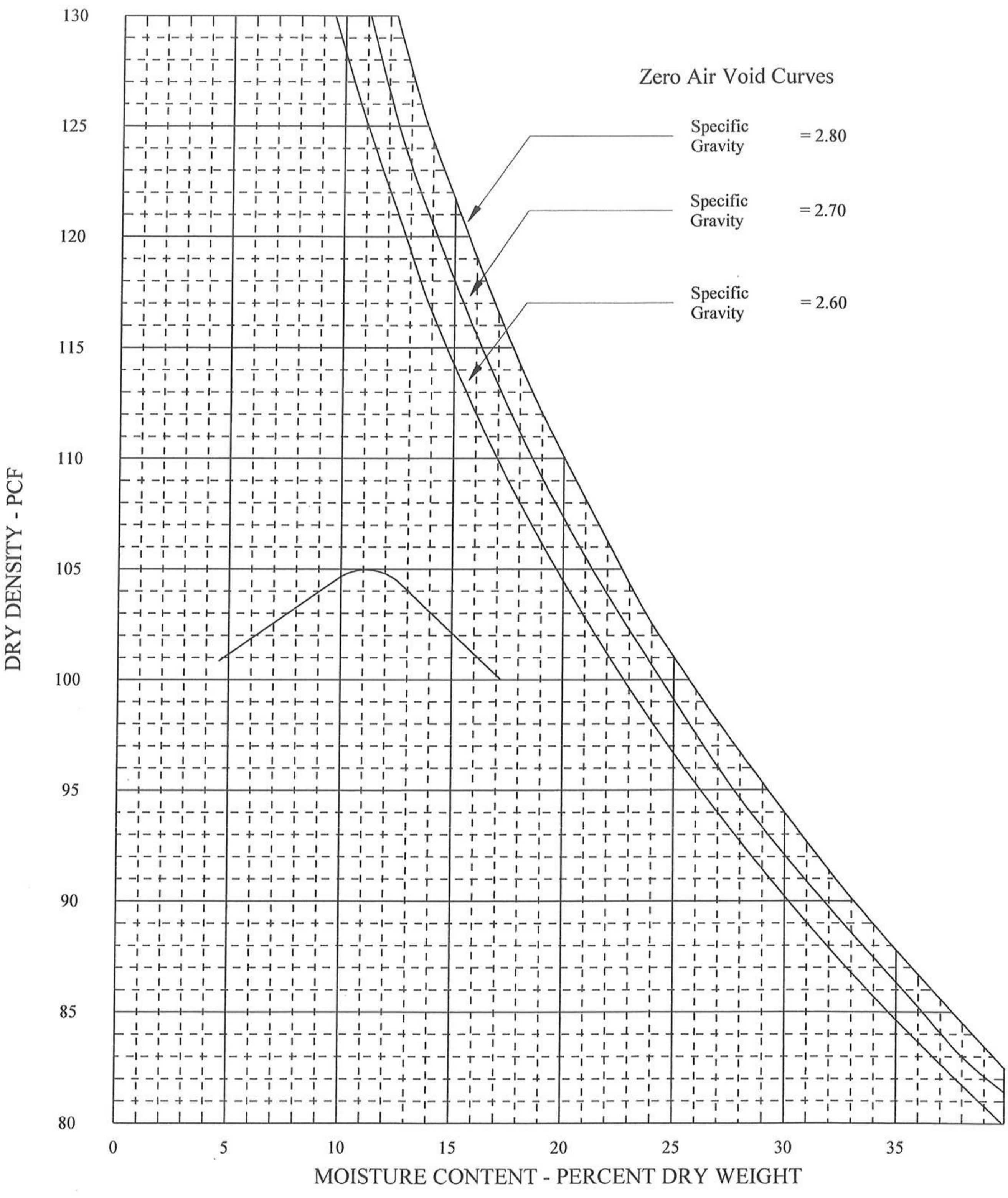


GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

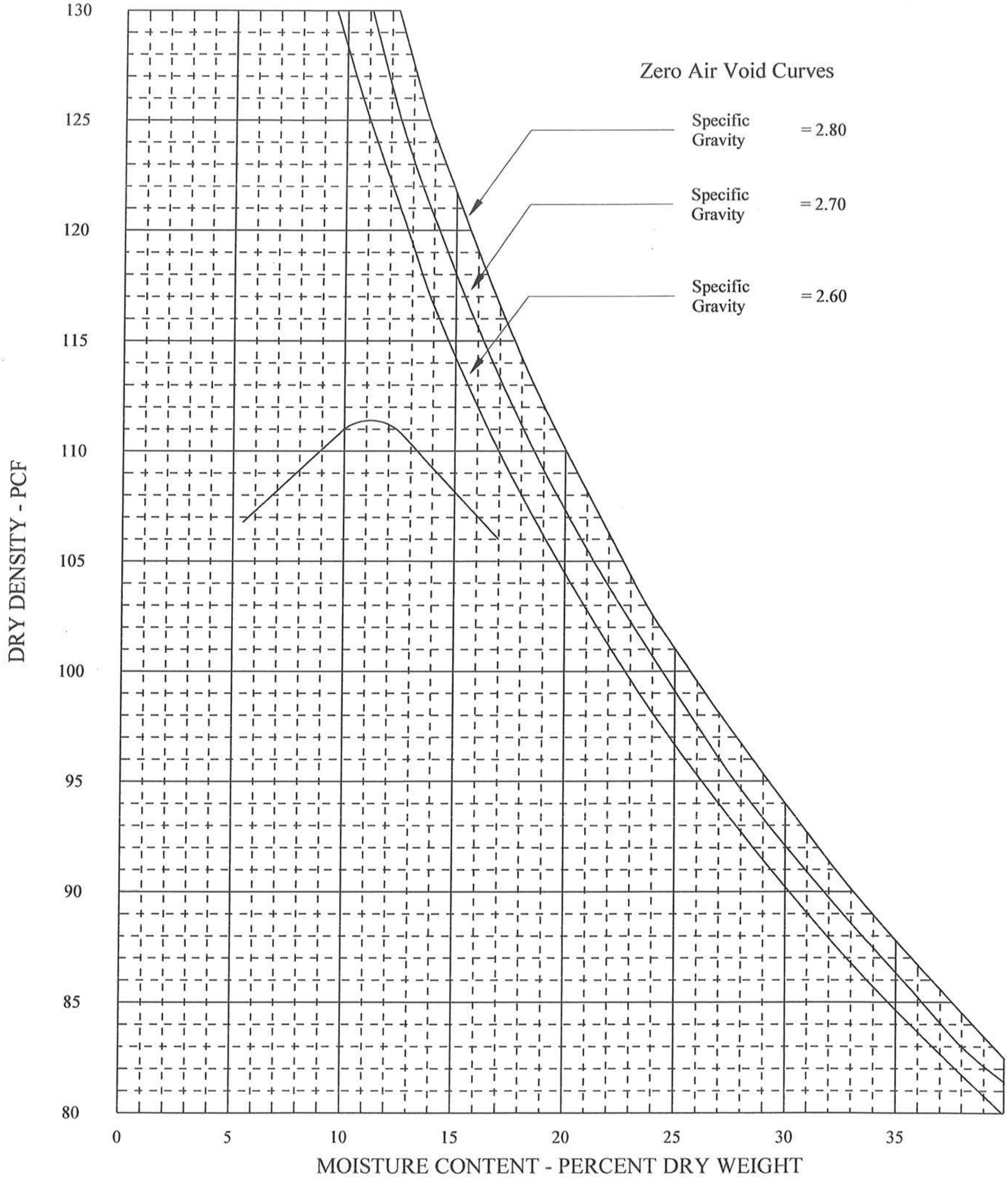
SAMPLE OF FROM

18-354B	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Gradation Test Results	Fig. 1



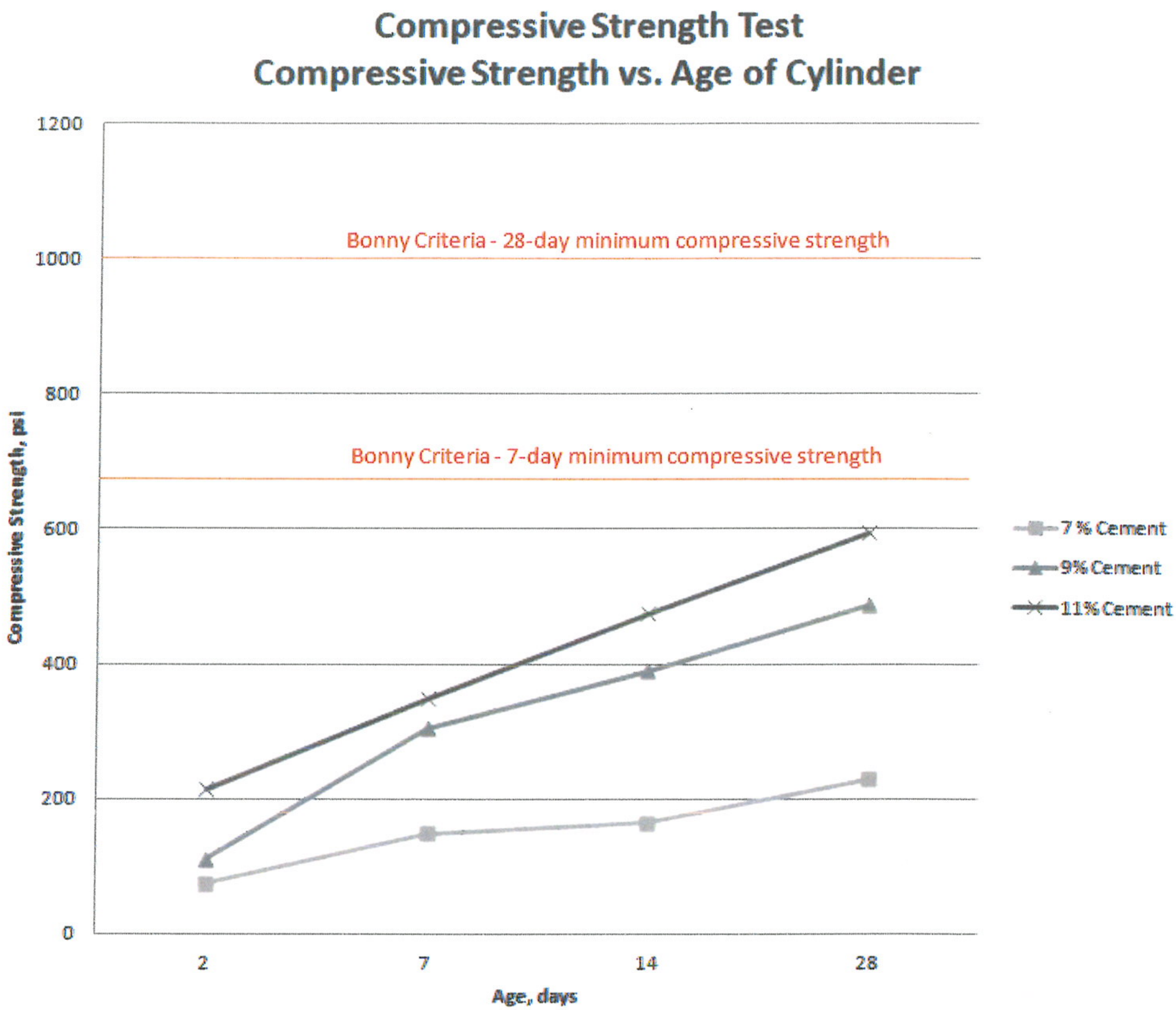
Location: Prewitt Reservoir Embankment	Max Dry Density: 105.0 pcf
Sample: Test pit TP-1	Opt. Moist. Content: 11.2 %
Sample Description: Sand	Gravel 0 % Sand 96 % Silt and Clay 4 %
Procedure: ASTM D698, A	Liquid Limit: % Plasticity Index: NP %

18-354B	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Moisture-Density Relationships	Fig. 2



Location: Prewitt Reservoir	Max Dry Density: 111.4 pcf
Sample: Test pit TP-1	Opt. Moist. Content: 11.1 %
Sample Description: Sand and 9% cement	
Procedure: ASTM D558, A	

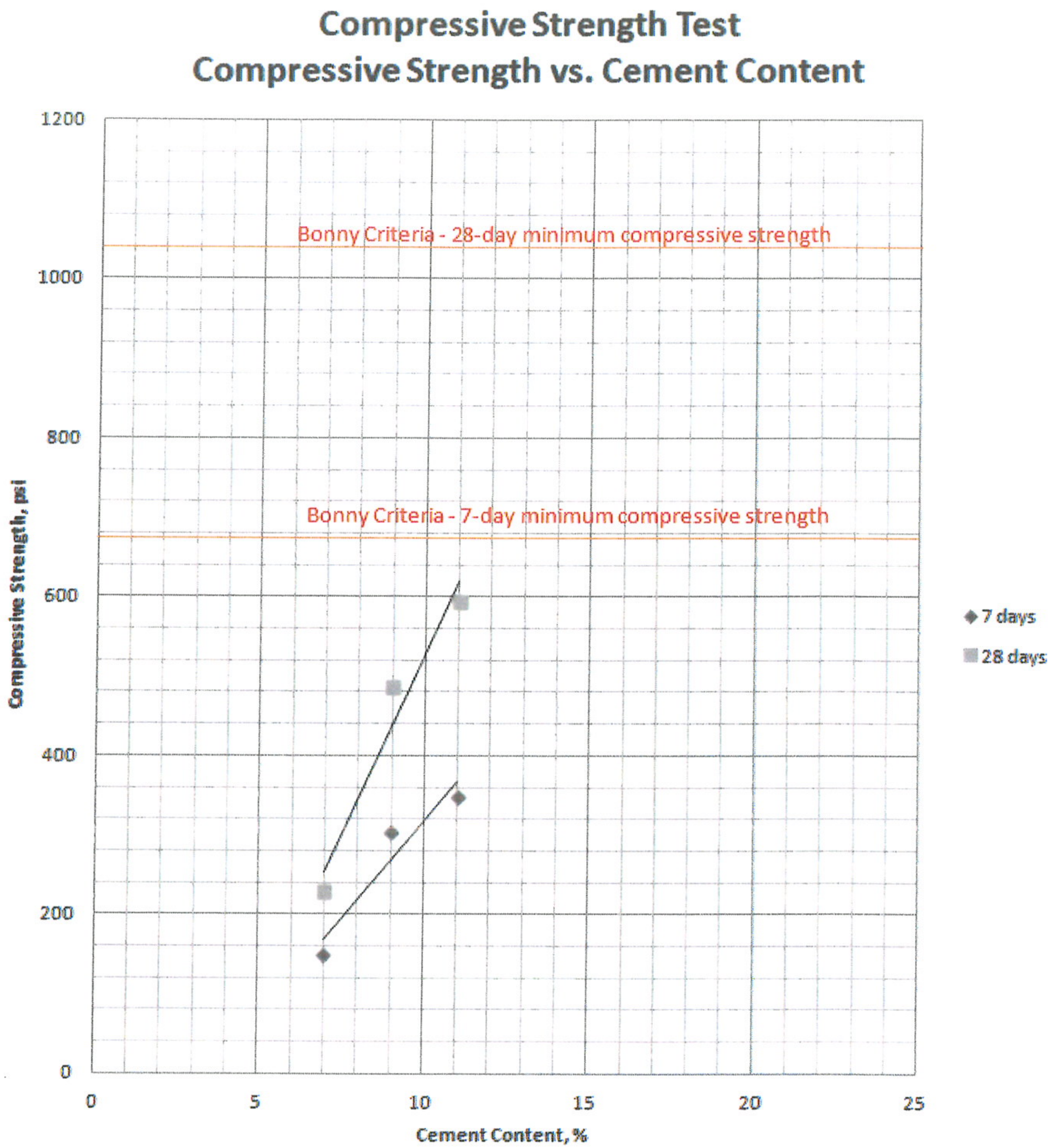
18-354B	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Moisture-Density Relationships	Fig. 3



Compressive Strength, psi

	Age, days			
	2	7	14	28
7 % Cement	75	149	164	229
9% Cement	111	304	390	487
11% Cement	215	348	475	594

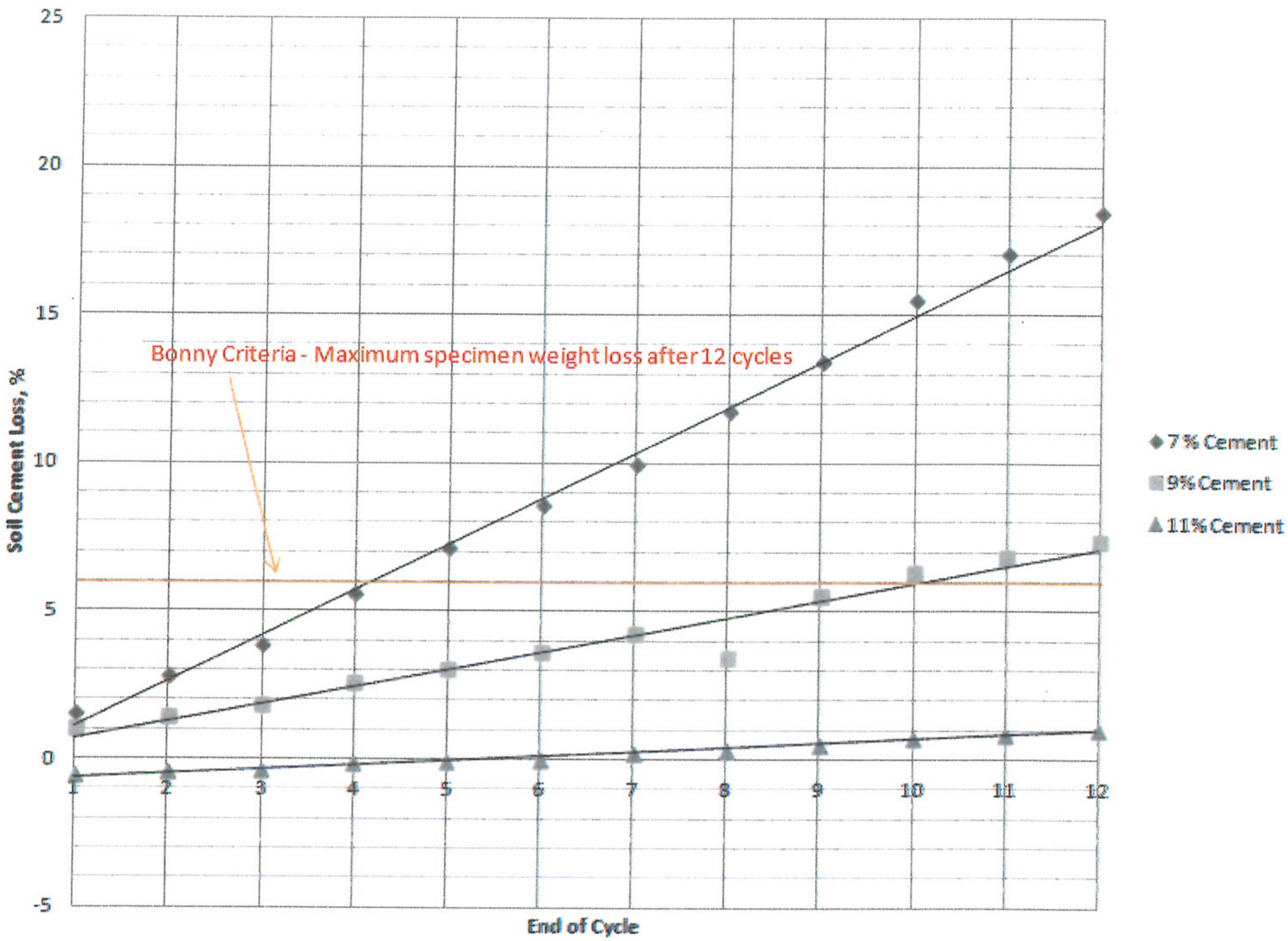
18-354B	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Compressive Strength vs. Age of Cylinder	Fig. 4



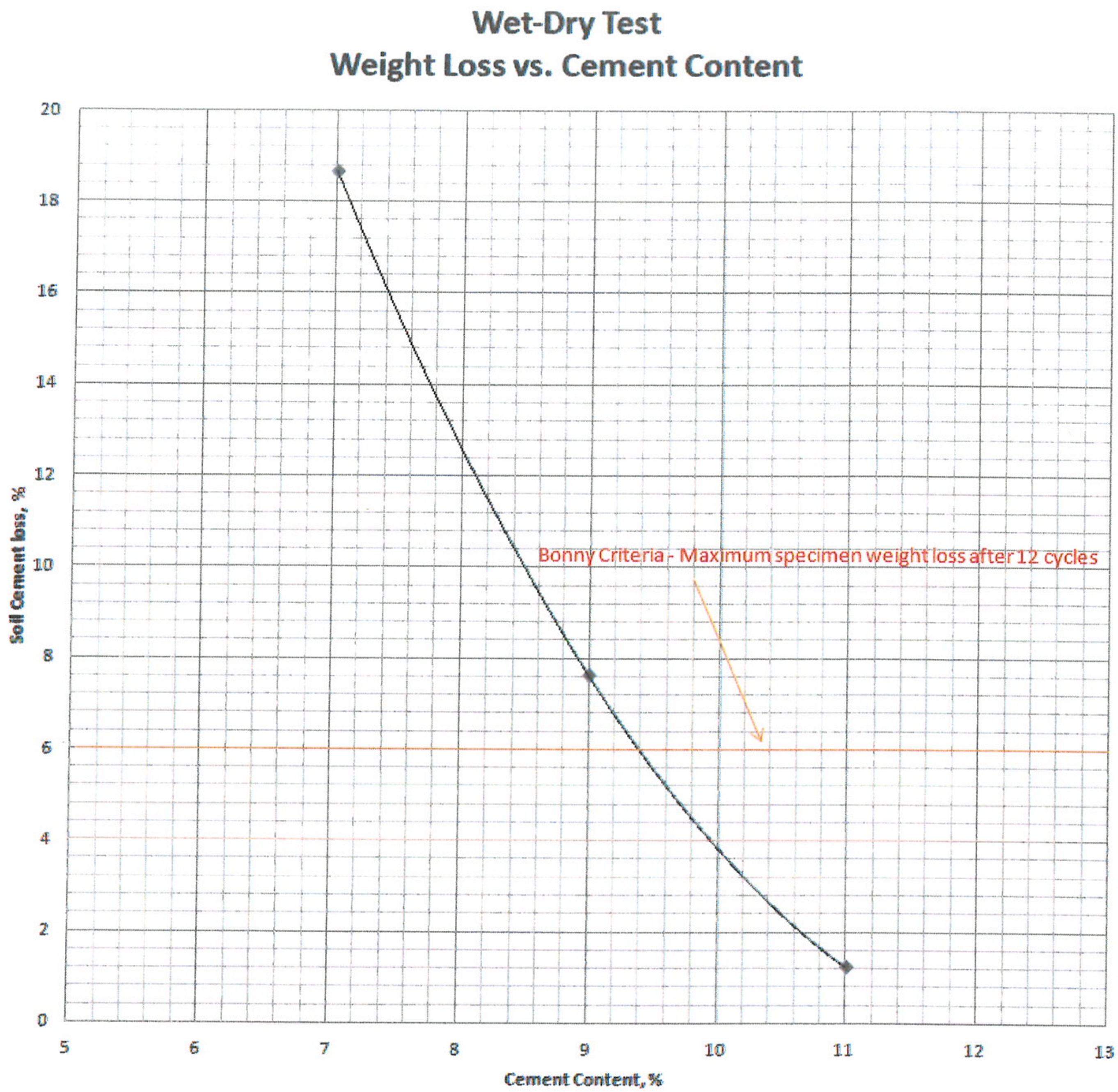
Compressive Strength, psi

	Age	
	7 days	28 days
7	149	229
9	304	487
11	348	594

Wet-Dry Test
Cumulative Weight Loss

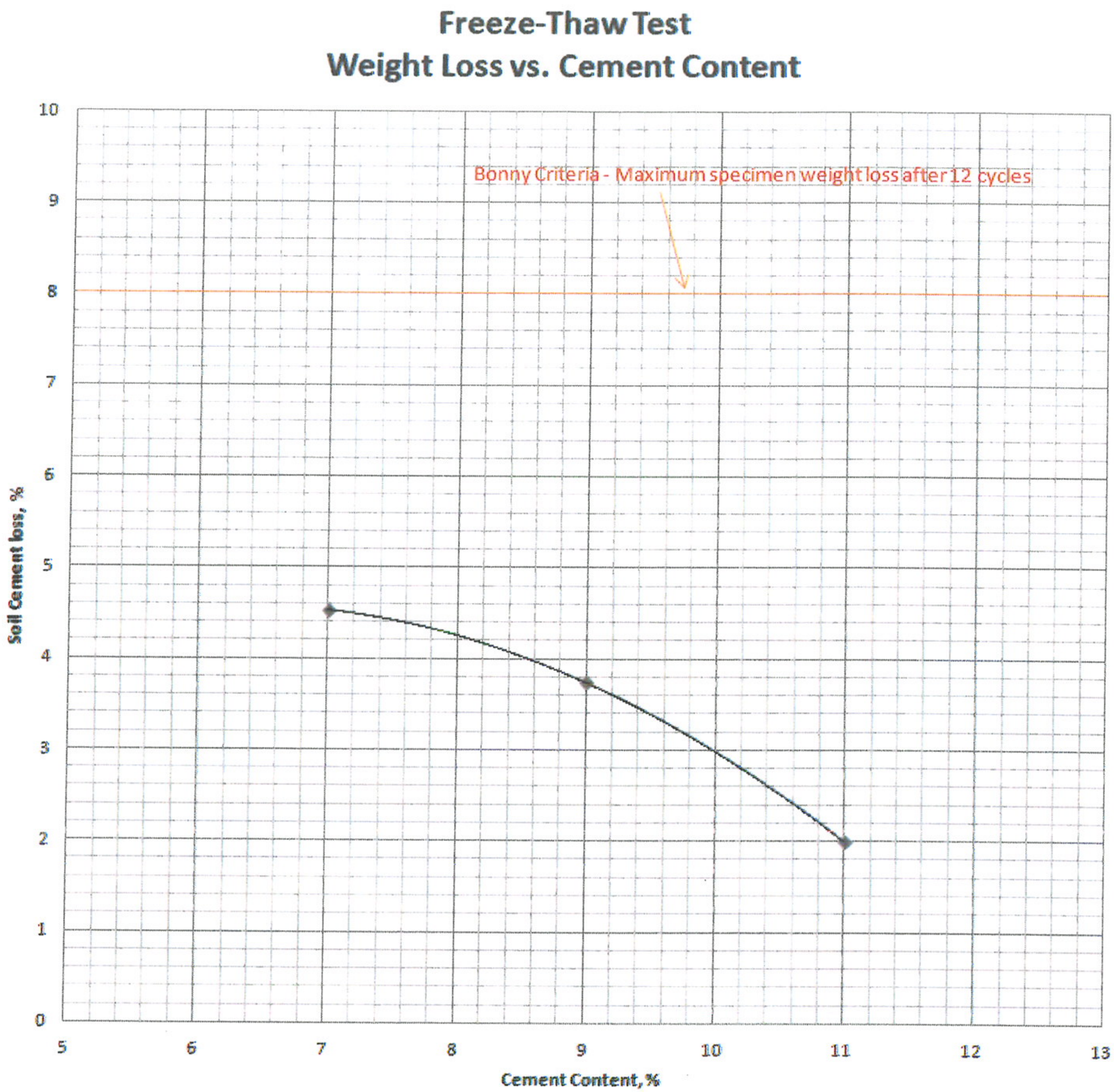


	Soil-Cement Loss, %					
	End of Cycle					
	1	2	3	4	5	6
7 % Cement	1.52	2.77	3.81	5.55	7.13	8.59
9% Cement	0.98	1.41	1.79	2.56	3.03	3.58
11% Cement	-0.55	-0.43	-0.38	-0.12	-0.09	-0.04
	7	8	9	10	11	12
7 % Cement	9.93	11.73	13.41	15.49	17.05	18.46
9% Cement	4.26	3.44	5.53	6.33	6.82	7.34
11% Cement	0.18	0.34	0.5	0.74	0.81	1.02



	Weight Loss, %
7	18.69
9	7.63
11	1.24

18-354B	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Wet-Dry Weight Loss vs. Cement Content	Fig. 7



	Weight Loss, %
7	4.53
9	3.74
11	2

18-354B	HOLLINGSWORTH ASSOCIATES Geotechnical/Environmental Engineers	Freeze-Thaw Weight Loss vs. Cement Content	Fig. 8

Hollingsworth Associates, Inc.

Job No.: 18-354B

TABLE I
SUMMARY OF TP-1 SAND LABORATORY TEST RESULTS

Sample Location		Gradation			Atterberg Limits		Proctor		
Boring	Natural Moisture Content (%)	Gravel + No. 4 (%)	Sand - No. 4 + No. 200 (%)	Minus No. 200 Sieve (%)	Liquid Limit (%)	Plasticity Index (%)	Optimum Moisture Content (%)	Maximum Dry Density (pcf)	Soil Type
TP-1	1.8	0	96	4		NP	11.2	105.0	Sand
TP-1 (9% cement)							11.1	111.4	Sand with 9% cement

Hollingsworth Associates, Inc.

Job No.: 18-354B

TABLE II
SUMMARY OF COMPRESSIVE STRENGTH TEST RESULTS

Cylinder No.	Cement Content (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength 2 days (psi)	7 days (psi)	14 days (psi)	28 days (psi)
4A	7	12.0	109.4	75			
4B	7	12.1	109.0		149		
4C	7	11.7	110.8			164	
4D	7	12.1	110.5				229
5A	9	11.5	111.6	111			
5B	9	12.1	113.9		304		
5C	9	10.9	111.1			390	
5D	9	12.1	112.6				487
6A	11	12.0	114.4	215			
6B	11	12.0	113.3		348		
6C	11	12.1	112.5			475	
6D	11	11.9	113.3				594

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Job No.: 18-354B

TABLE III
SUMMARY OF WET-DRY TEST SPECIMENS TEST RESULTS

Cylinder No.	Cement Content (%)	Moisture Content (%)	Dry Density (pcf)	12 Cycle Weight Loss (%)
1	7	12.1	108.4	18.69
2	9	12.1	111.0	7.63
3	11	11.3	111.1	1.24

Hollingsworth Associates, Inc.

Job No.: 18-354B

TABLE IV
SUMMARY OF FREEZE-THAW TEST SPECIMENS TEST RESULTS

Cylinder No.	Cement Content (%)	Moisture Content (%)	Dry Density (pcf)	12 Cycle Weight Loss (%)
7	7	11.8	109.7	4.53
8	9	12.1	108.9	3.74
9	11	12.0	111.3	2.00

APPENDIX A

Table IA – Wet-Dry Cycle Measurements
Table IIA – Freeze-Thaw Cycle Measurements

TABLE IA

Wet-Dry Cycle Measurements

Cylinder #	Cement, %	End of cycle	1	2	3	4	5	6	7	8	9	10	11	12	Final
1	7%	Cylinder height, in	4.2100	4.1800	4.1700	4.1700	4.1400	4.1200	4.1200	4.0800	4.0800	4.0400	4.0400	4.0000	3.9800
		Cylinder width, in	4.0000	3.9800	3.9800	3.9600	3.9400	3.9400	3.9200	3.9200	3.9200	3.8200	3.8000	3.7800	3.7800
		Pre-brush wt., lb	3.3650	3.3000	3.2670	3.2310	3.1730	3.1225	3.0690	3.0270	2.9670	2.9080	2.8405	2.7885	2.7300
		Post-brush wt., lb	3.3105	3.2685	3.2355	3.1750	3.1220	3.0730	3.0280	2.9675	2.9110	2.8410	2.7885	2.7410	

Cylinder #	Cement, %	End of cycle	1	2	3	4	5	6	7	8	9	10	11	12	Final
2	9%	Cylinder height, in	4.4400	4.4400	4.4400	4.4300	4.4200	4.4200	4.4200	4.4200	4.4200	4.3800	4.3800	4.3500	4.3500
		Cylinder width, in	4.0000	3.9700	3.9700	3.9700	3.9600	3.9600	3.9600	3.9600	3.9600	3.9600	3.9600	3.9400	3.9400
		Pre-brush wt., lb	3.6220	3.5850	3.5665	3.5505	3.5250	3.5075	3.4855	3.4620	3.4350	3.4130	3.3880	3.3715	3.3400
		Post-brush wt., lb	3.5805	3.5650	3.5115	3.5235	3.5065	3.4865	3.4620	3.4360	3.4160	3.3870	3.3695	3.3505	

Cylinder #	Cement, %	End of cycle	1	2	3	4	5	6	7	8	9	10	11	12	Final
3	11%	Cylinder height, in	4.4100	4.4000	4.3900	4.3900	4.3900	4.3900	4.3900	4.3900	4.3900	4.3900	4.3900	4.3900	4.3900
		Cylinder width, in	4.0000	3.9700	3.9700	3.9700	3.9700	3.9700	3.9700	3.9700	3.9700	3.9600	3.9600	3.9600	3.9600
		Pre-brush wt., lb	3.6470	3.6350	3.6260	3.6200	3.6155	3.6130	3.6085	3.6030	3.5950	3.5870	3.5835	3.5770	3.5600
		Post-brush wt., lb	3.6275	3.6230	3.6215	3.6120	3.6110	3.6090	3.6010	3.5955	3.5895	3.5810	3.5785	3.5710	

TABLE IIA

Freeze-Thaw Cycle Measurements

Cylinder #	Cement, %	End of cycle	1	2	3	4	5	6	7	8	9	10	11	12	Final
7	7%	Cylinder height, in	4.4300	4.4200	4.4200	4.3900	4.3900	4.3900	4.3900	4.3900	4.3900	4.3900	4.3900	4.3900	4.3900
		Cylinder width, in	3.9900	3.9900	3.9800	3.9800	3.9800	3.9800	3.9700	3.9700	3.9700	3.9700	3.9700	3.9700	3.9700
		Pre-brush wt., lb	3.9295	3.9430	3.9210	3.9065	3.8910	3.8735	3.8635	3.8560	3.8475	3.8375	3.8335	3.8260	3.4335
		Post-brush wt., lb	3.9155	3.9195	3.9055	3.8890	3.8725	3.8600	3.8520	3.8455	3.8370	3.8335	3.8235	3.8145	

Cylinder #	Cement, %	End of cycle	1	2	3	4	5	6	7	8	9	10	11	12	Final
8	9%	Cylinder height, in	4.5000	4.4400	4.4400	4.4400	4.4400	4.4300	4.4300	4.4300	4.4300	4.4300	4.4300	4.4200	4.4200
		Cylinder width, in	3.9900	3.9900	3.9900	3.9900	3.9700	3.9700	3.9700	3.9700	3.9700	3.9700	3.9700	3.9600	3.9600
		Pre-brush wt., lb	3.9790	3.9925	3.9730	3.9530	3.9370	3.9220	3.9135	3.8960	3.8980	3.8900	3.8845	3.8780	3.4820
		Post-brush wt., lb	3.9645	3.9720	3.9525	3.9360	3.9215	3.9100	3.9025	3.8910	3.8880	3.8860	3.8755	3.8680	

Cylinder #	Cement, %	End of cycle	1	2	3	4	5	6	7	8	9	10	11	12	Final
9	11%	Cylinder height, in	4.4000	4.4000	4.4000	4.4000	4.4000	4.4000	4.4000	4.4000	4.3900	4.3900	4.3700	4.3700	4.3700
		Cylinder width, in	3.9900	3.9900	3.9900	3.9900	3.9800	3.9800	3.9800	3.9600	3.9600	3.9600	3.9600	3.9600	3.9600
		Pre-brush wt., lb	3.9685	4.0095	4.0050	3.9970	3.9915	3.9850	3.9835	3.9740	3.9755	3.9720	3.9655	3.9635	3.5680
		Post-brush wt., lb	3.9610	4.0005	3.9960	3.9900	3.9540	3.9795	3.9785	3.9700	3.9695	3.9670	3.9600	3.9580	

Appendix E

Feasibility Level Opinions of Probable Construction Costs

Class IV Opinion of Probable Dredging Construction Costs

Hydraulic Dredging Opinion of Costs

Item	Unit	Estimated Quantity	Unit Price	Total Cost
Mobilization	LS	1	$ 208,000.00	$ 208,000.00
Hydraulic Dredging	CY	165,000	$ 12.00	$ 1,980,000.00
Site Reclamation	LS	1	$ 100,000.00	$ 100,000.00
			Estimated Construction Costs	$ 2,288,000.00
			Contingency (15%)	$ 343,200.00
			Sub-Total	$ 2,631,200.00
			Engineering , Permitting (7.5%)	$ 197,340.00
			Total Capital Costs	**$ 2,828,540.00**
			Use	**2.83-Mil**

Dry Mechanical Dredging Opinion of Costs

Item	Unit	Estimated Quantity	Unit Price	Total Cost
Mobilization	LS	1	$ 297,500.00	$ 297,500.00
Dewatering	LS	1	$ 500,000.00	$ 500,000.00
Mass Excavation	CY	165,000	$ 15.00	$ 2,475,000.00
			Estimated Construction Costs	$ 3,272,500.00
			Contingency (15%)	$ 490,875.00
			Sub-Total	$ 3,763,375.00
			Engineering , Permitting (15%)	$ 564,506.25
			Total Capital Costs	**$ 4,327,881.25**
			Use	**4.33-Mil**

Wet Mechanical Dredging Opinion of Costs

Item	Unit	Estimated Quantity	Unit Price	Total Cost
Mobilization	LS	1	$ 1,000,000.00	$ 1,000,000.00
Wet Mechanical Dredging	CY	165,000	$ 20.00	$ 3,300,000.00
Sediment Disposal	CY	165,000	$ 3.00	$ 495,000.00
Site Reclamation	LS	1	$ 100,000.00	$ 100,000.00
			Estimated Construction Costs	$ 4,895,000.00
			Contingency (15%)	$ 734,250.00
			Sub-Total	$ 5,629,250.00
			Engineering , Permitting (7.5%)	$ 422,193.75
			Total Capital Costs	**$ 6,051,443.75**
			Use	**6.05-Mil**

Class IV Opinion of Probable Dredging Construction Costs: Environmental Enhancement Sub-Alternatives

Island Via Hydraulic Dredging

Item	Unit	Estimated Quantity	Unit Price	Total Cost
Mobilization	LS	1	$ 242,500.00	$ 242,500.00
Hydraulic Dredging	CY	175,000	$ 13.00	$ 2,275,000.00
Site Reclamation	AC	5	$ 30,000.00	$ 150,000.00
Containment Dikes	LS	1	$ 250,000.00	$ 250,000.00
			Estimated Construction Costs	$ 2,917,500.00
			Contingency (15%)	$ 437,625.00
			Sub-Total	$ 3,355,125.00
			Engineering , Permitting (7.5%)	$ 251,634.38
			Total Capital Costs	**$ 3,606,759.38**
			Use	**3.61-Mil**

Wetlands Via Hydraulic Dredging

Item	Unit	Estimated Quantity	Unit Price	Total Cost
Mobilization	LS	1	$ 316,850.00	$ 316,850.00
Hydraulic Dredging	CY	165,000	$ 16.80	$ 2,772,000.00
Wetland Infrastructure	LS	1	$ 150,000.00	$ 150,000.00
Wetland Seeding	AC	29	$ 8,500.00	$ 246,500.00
Wetland Monitoring	YRS	10	$ 5,000.00	$ 50,000.00
			Estimated Construction Costs	$ 3,535,350.00
			Contingency (15%)	$ 530,302.50
			Sub-Total	$ 4,065,652.50
			Engineering , Permitting (7.5%)	$ 304,923.94
			Total Capital Costs	**$ 4,370,576.44**
			Use	**4.37-Mil**

Potential Wetland Bank Credits

Item	Unit	Estimated Quantity	Unit Price	Total Value
Wetland Bank Credits	AC	18.85	$ 100,000.00	$ 1,885,000.00

Class IV Opinion of Probable Wave Mitigation Construction Costs: Beaching Sub-Alternatives

Hydraulic Dredging Placement Estimate

Item	Unit	Estimated Quantity	Unit Price	Total Cost
Mobilization	LS	1	$ 1,679,400.00	$ 1,679,400.00
Hydraulic Dredging	CY	1,399,500	$ 12.00	$ 16,794,000.00
Estimated Construction Costs				$ 18,473,400.00
Contingency (15%)				$ 2,771,010.00
Sub-Total				$ 21,244,410.00
Engineering , Permitting (7.5%)				$ 1,593,330.75
Total Capital Costs				**$ 22,837,740.75**
Use				**22-Mil**

Mechanical Placement Estimate

Item	Unit	Estimated Quantity	Unit Price	Total Cost
Mobilization	LS	1	$ 839,700.00	$ 839,700.00
Dewatering	LS	1	$ 500,000.00	$ 500,000.00
Mass Fill	CY	1,399,500	$ 6.00	$ 8,397,000.00
Estimated Construction Costs				$ 9,736,700.00
Contingency (15%)				$ 1,460,505.00
Sub-Total				$ 11,197,205.00
Engineering , Permitting (15%)				$ 1,679,580.75
Total Capital Costs				**$ 12,876,785.75**
Use				**12.3-Mil**

Class IV Opinion of Probable Wave Mitigation Construction Costs: Soil-Cement Sub-Alternatives

Stair-Stepped Section

Item	Unit	Estimated Quantity	Unit Price	Total Cost
Mobilization	LS	1	$ 1,363,626.67	$ 1,363,626.67
Dewatering	LS	1	$ 500,000.00	$ 500,000.00
Foundation Prep	SY	23,500	$ 8.00	$ 188,000.00
Select Borrow	CY	213,067	$ 4.00	$ 852,266.67
Soil Cement-Stepped	CY	213,067	$ 60.00	$ 12,784,000.00
			Estimated Construction Costs	$ 15,687,893.33
			Contingency (15%)	$ 2,353,184.00
			Sub-Total	$ 18,041,077.33
			Engineering , Permitting (15%)	$ 2,706,161.60
			Total Capital Costs	**$ 20,747,238.93**
			Use	**20.75-Mil**

Composite Section

Item	Unit	Estimated Quantity	Unit Price	Total Cost
Mobilization	LS	1	$ 678,589.63	$ 678,589.63
Dewatering	LS	1	$ 500,000.00	$ 500,000.00
Foundation Prep	SY	23,500	$ 8.00	$ 188,000.00
Select Borrow	CY	332,696	$ 4.00	$ 1,330,785.19
Soil Cement-Stepped	CY	90,919	$ 60.00	$ 5,455,111.11
Soil Cement-Plated	CY	54,652	$ 50.00	$ 2,732,592.59
Embankment Fill	CY	187,126	$ 5.00	$ 935,629.63
			Estimated Construction Costs	$ 11,820,708.15
			Contingency (15%)	$ 1,773,106.22
			Sub-Total	$ 13,593,814.37
			Engineering , Permitting (15%)	$ 2,039,072.16
			Total Capital Costs	**$ 15,632,886.53**
			Use	**15.6-Mil**

Class IV Probable Wave Mitigation Construction Costs:
Parapet Alternative

Parapet Wall Estimate

Item	Unit	Estimated Quantity	Unit Price	Total Cost
Mobilization	LS	1	$ 485,000.00	$ 485,000.00
Demolition	LS	1	$ 250,000.00	$ 250,000.00
Parapet Concrete	CY	4,300	$ 1,000.00	$ 4,300,000.00
Site Reclamation	LS	1	$ 550,000.00	$ 550,000.00
			Estimated Construction Costs	$ 5,585,000.00
			Contingency (15%)	$ 837,750.00
			Sub-Total	$ 6,422,750.00
			Engineering , Permitting (15%)	$ 963,412.50
			Total Capital Costs	**$ 7,386,162.50**
			Use	**7.4-Mil**

Class IV Opinion of Probable Wave Mitigation Construction Costs:
Floating Breakwater "Wave Eater" Sub-Alternatives

Wave Eater Floating Breakwater-3-String

Item	Unit	Estimated Quantity	Unit Price	Total Cost
3-String Wave Eater	LF	3,750	$ 387.00	$ 1,451,250.00
Minor Parapets	LF	2,200	$ 75.00	$ 165,000.00
			Estimated Construction Costs	$ 1,616,250.00
			Contingency (15%)	$ 242,437.50
			Sub-Total	$ 1,858,687.50
			Engineering , Permitting (7.5%)	$ 139,401.56
			Total Capital Costs	**$ 1,998,089.06**
			Use	**2.0-Mil**

Wave Eater Floating Breakwater-2-String

Item	Unit	Estimated Quantity	Unit Price	Total Cost
3-String Wave Eater	LF	3,750	$ 284.00	$ 1,065,000.00
Minor Parapets	LF	2,200	$ 75.00	$ 165,000.00
			Estimated Construction Costs	$ 1,230,000.00
			Contingency (15%)	$ 184,500.00
			Sub-Total	$ 1,414,500.00
			Engineering , Permitting (7.5%)	$ 106,087.50
			Total Capital Costs	**$ 1,520,587.50**
			Use	**1.52-Mil**

WAVE EATER

December 12, 2018

Wenck
Attn: Marcus Krall
7000 Yellowtail Road, Ste. 230
Cheyenne, WY 82009

Re: WaveEater Proposal 181212

Marcus:

I am pleased to provide the following 2 proposals:

The first proposal is for a WaveEater 3-string attenuation system. This would be more than adequate to meet the wind and wave criteria. The second proposal is for a 2 – string attenuation system. I also believe this would meet the specifications. The fetch distance from SW – NE is 3.34 miles. I still believe that is not enough distance to generate a 6' high wave, but I am designing for it just the same.

Option 1. 3 string system - 3,750' long.

- Materials to include:
  - Model 3636 WaveEater 2,400 Units
  - ½" Stainless Steel Cable 7,700 Feet
  - 1 – ½" PolyPipe 7,700 Feet
  - Delrin Spacers 152
  - ½" SS Thimbles 160
  - ½" SS Wire Rope Clips 625
  - 7/16" SS Shackles 480
  - ½" SS Shackles 80
  - ½" Stainless eye-eye swivel 80
  - 3/8" Galvanized Chain 3,000 Feet
  - 2" Dia x 12' SS Anchor Rod 39
  - Anchor Buoy 24" 78
  - Regulatory solar lighted buoys 20
  - Submittals 1

Installation by WaveEater

- Scope of Work
    - Mobilization
    - Form and pour 78 – 2,000lb concrete anchors on site.
    - Form and pour 78 – 200lb 3000PSI concrete shock absorbers on site.
    - Form and pour 20 – 200lb 3000PSI concrete Buoy anchors on site.
    - Assemble and stage 176 anchor lines
    - Stage and launch anchor points including marker buoys.
    - Assemble and stage cable stringer assemblies including eye splices.
    - Assemble and stage 2400 WaveEater units onto cable stringer assemblies.
    - Launch and position WaveEater units.
    - Demobilization.

Option 1 – Price manufactured, delivered and installed	**$1,450,000.00**
Price per lineal foot	$ 387.00

- Terms: To be determined.
- Estimated lead-time 15 – 18 weeks ARO for materials to be on site.
- Estimated time for installation including mobilization and demobilization 60 days.

Option 2. 2 string system - 3,750' long.

- Materials to include:
  - Model 3636 WaveEater 1,800 Units
  - ½" Stainless Steel Cable 5,100 Feet
  - 1 – ½" PolyPipe 5,100 Feet
  - Delrin Spacers 102
  - ½" SS Thimbles 110
  - ½" SS Wire Rope Clips 440
  - 7/16" SS Shackles 480
  - ½" SS Shackles 54
  - ½" Stainless eye-eye swivel 80
  - 3/8" Galvanized Chain 3,000 Feet
  - 2" Dia x 12' SS Anchor Rod 39
  - Anchor Buoy 24" 78
  - Regulatory solar lighted buoys 20
  - Submittals 1

Installation by WaveEater

- Scope of Work
  - Mobilization
  - Form and pour 78 – 2,000lb concrete anchors on site.
  - Form and pour 78 – 200lb 3000PSI concrete shock absorbers on site.
  - Form and pour 20 – 200lb 3000PSI concrete Buoy anchors on site.
  - Assemble and stage 176 anchor lines
  - Stage and launch anchor points including marker buoys.
  - Assemble and stage cable stringer assemblies including eye splices.
  - Assemble and stage 2400 WaveEater units onto cable stringer assemblies.
  - Launch and position WaveEater units.
  - Demobilization.

Option 2 – Price manufactured, delivered and installed $1,065,000.00

Price per lineal foot $ 284.00

- Terms: To be determined.
- Estimated lead-time 12 – 15 weeks ARO for materials to be on site.
- Estimated time for installation including mobilization and demobilization 55 days.

This proposal is in effect for thirty days.

Thank you for this opportunity.

Sincerely,

Peter W. Odenbach
WaveEater LLC
4012 Maguire Blvd, Ste. 4209
Orlando, FL 32803
(407) 630-2692

peter@waveeater.com

WENCK

Responsive partner.
Exceptional outcomes.