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MEMORANDUM

DATE: 6/3/2025

TO Adrienne Martinez, AMAFCA

FROM: Emma Adams, EI
Robert Salazar, PE
Stu Trabant, PE

SUBJECT: **Swinburne Dam Grade Control Structures – Design Memo**

BACKGROUND

The Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) is planning the construction of a flow attenuation and sediment retention facility within the existing Swinburne Dam impoundment to manage dam outflows and reduce sediment loads discharged downstream to the Calabacillas Arroyo (**Figure 1**). Construction of the new facility would involve excavation of sediment from the dam pool and construction of a low-head dam to manage flow and retain sediment. The planned depth of excavation would exceed 20 feet at the upstream end of the dam pool, putting the existing grade control structures (GCSs) at risk. These GCSs are located at the head of the existing pool and include GCS NB1 in the Main Branch Calabacillas Arroyo and the GCS WB4 in the West Branch Calabacillas Arroyo (**Figure 1**). To protect these structures, AMAFCA contracted Bohannan Huston, Inc. (BHI) to prepare the design and construction plans for two additional GCSs at the upstream limits of the planned excavation. This design included hydraulic modeling to determine design parameters and establish proposed hydraulic conditions. These structures will be constructed and then reburied (finished conditions) so that future sediment removal can occur as necessary over time until the planned future conditions elevation is reached.

The modeling and design processes were iterative and co-dependent. Hydraulic modeling was performed to estimate hydraulic conditions. Hydraulic conditions were used as inputs to design calculations estimating hydraulic jump length/height, scour depth, and freeboard requirement at both proposed GCSs. Based on the calculated design parameters, the design was developed and modeled. This process was repeated to optimize the design and to ensure that the hydraulic jumps from the GCSs' drops occur within the armored portion of the structure.

This memorandum summarizes the work conducted to prepare 60% designs of these structures, identified as GCS NB0 in the Main Branch and GCS WB0 in the West Branch. This memorandum supersedes the conceptual design memorandum that was delivered to AMAFCA in October 2024.

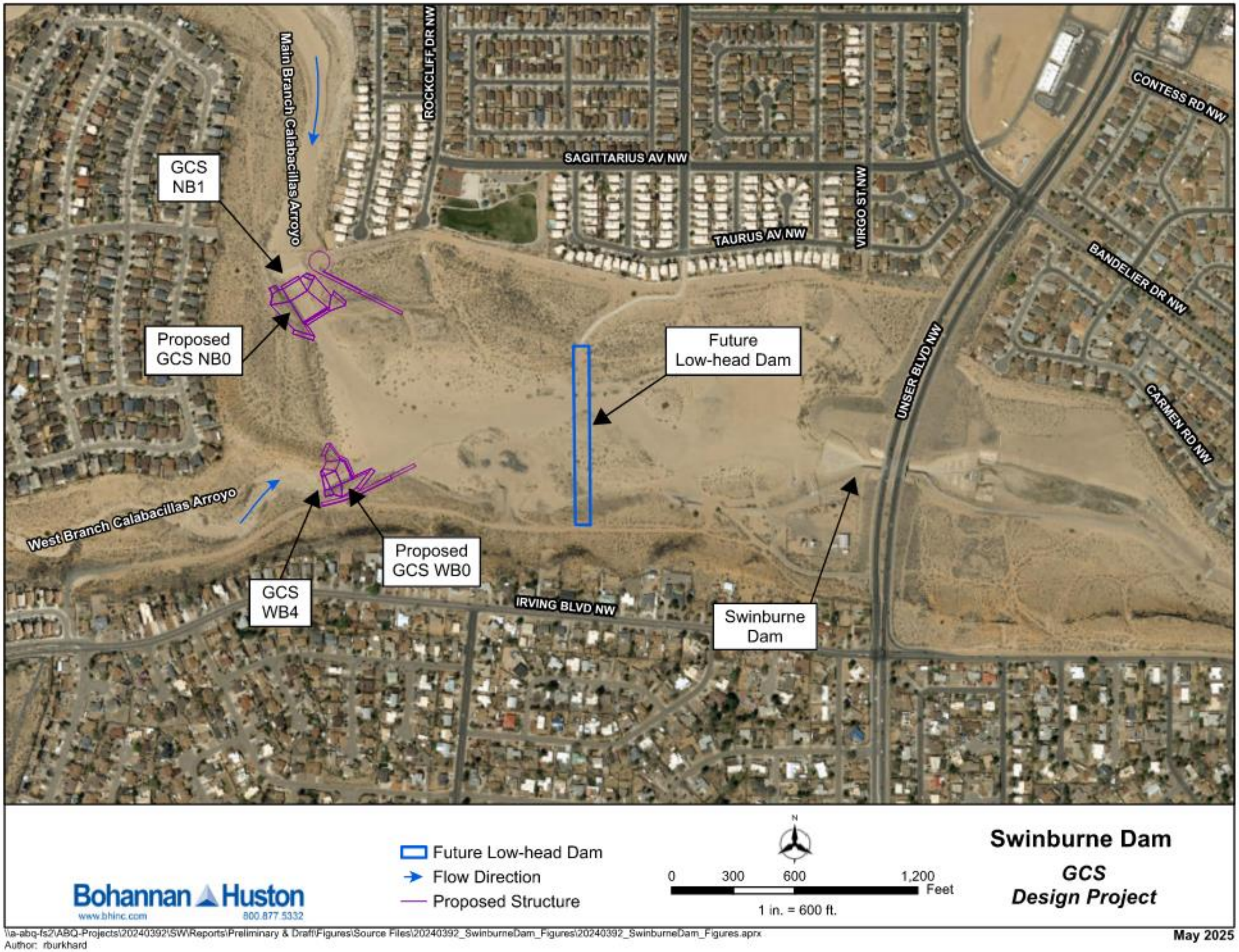

Figure 1. Key Elements Considered as Part of This GCS Design Project

Literature Review

A literature review was conducted in preparation for this analysis and design. The following background information was obtained and reviewed for this project:

1. The primary document that serves as the basis for this design project is the *Calabacillas Arroyo Facility Plan Above Swinburne Dam* [*CAFP*] (Tetra Tech and BHI, 2021). This document outlines the recommendations for the proposed sediment retention facility and identifies the planned depth of excavation. Based on those recommendations, AMAFCA retained Tetra Tech to prepare a preliminary grading plan for the facility and accompanying memorandum (Tetra Tech, 2022a and 2022b).
2. The preceding document to the *CAFP*, *Calabacillas Arroyo Facility Plan Field Reconnaissance Report* [*Field Report*] (Tetra Tech and BHI, 2020), was used to assess existing topographic data of the Swinburne Dam and pool.
3. Record drawings for AMAFCA's *Calabacillas Arroyo Grade Control Structures and Bank Protection – Phase 3*, which includes the design drawings for GCS NB1, GCS WB4, and the associated bank protection (BHI, 2001).
4. The *Calabacillas West Branch Arroyo Drainage and Storm Water Quality Management Plan – Phase 2, Task B Evaluation of Alternatives for Developed Conditions Options Final Report* [*CWB Management Plan*] (BHI and Tetra Tech, 2019) provided a peak flow rate at the proposed location of GCS WB0.

Site Visits

A preliminary site visit to the project area was conducted by AMAFCA and BHI on February 7, 2024. The purpose of the site visit was to generally assess the overall project area, identify design constraints and considerations, and discuss alternatives for the location and layout of the structures. Based on these discussions and observations during the site visit, options for locating the structures were identified, including setting the crest of the proposed structures adjacent to the basin end sill of the existing structures and locating the proposed structures some distance downstream from the existing structures. Discussions regarding the type of materials for construction included the options of shotcrete, grouted boulders, and a combination of the two, all of which were identified as reasonable.

A 60% plan-in-hand site visit to the project area was conducted by AMAFCA and BHI on April 16, 2025, and included AMAFCA's maintenance staff. The purpose of this site visit was to confirm the layout of the structures and maintenance access, evaluate options to connect the proposed design to the existing structures, and determine potential material salvage locations. An existing riprap stockpile was observed in the southern area of the existing Swinburne Dam above the proposed low head dam. The existing fencing around the NB0 access road was observed and noted as limits of proposed access. Additionally, a few design revisions and refinements resulted from these discussions. It was determined that the northern bank protection and key-in of WB0 would be rotated more to fit the existing topography of the dam. Lastly, shotcrete constructability was discussed, and it was concluded that the upper edge of the shotcrete should have a consistent elevation wherever practicable.

Potential Utility Conflicts

High Mesa, a Bowman company, performed subsurface utility identification services in the project area in January 2025, delivering drawings and field notes of what was identified. The project area includes an Albuquerque Bernalillo County Water Utility Authority (ABCWUA) sanitary sewer easement that crosses the West Branch just below existing GCS WB4. Communication with ABCWUA representatives indicated that the easement is not in use and there is no sanitary sewer crossing at this location.

An overhead electric line runs along the south side of the dam pool, but it is not within the anticipated construction activity footprint. Protection of the overhead electric line was considered during the design of the access road along the south side of the dam pool.

An ABCWUA waterline runs along the north side of the dam pool, crossing the Main Branch upstream of the existing GCS NB1, but is outside of the potential construction area for this project.

Potential Impacts on City of Albuquerque Open Space

BHI evaluated the potential for impacts on City of Albuquerque (COA) Open Space. Through discussions with COA Open Space, it was determined that there are no COA Open Space facilities in the project area and that there would be no impact from this project on COA Open Space facilities.

SITE SURVEY

As part of the *CAFP* (BHI and Tetra Tech, 2021), LiDAR topographic mapping data of the Swinburne Dam pool was collected by Geomni on February 5, 2020, and described in the *Field Report* (BHI and Tetra Tech, 2020). From this data, BHI produced a new surface model accurate to a 1.0-foot contour interval along with visible planimetric information. For this design, BHI obtained current arroyo survey check points in February 2024 to determine whether the 2020 LiDAR mapping is still representative of the existing Swinburne Dam terrain. Based on a comparison of the survey check points with the 2020 LiDAR mapping, the 2020 LiDAR mapping was deemed sufficient for use in this design. The survey information and check shots were presented to AMAFCA, and further use of the data was approved by AMAFCA.

A topographic and LiDAR check survey was performed in the region of the proposed GCSs in March 2024. At the time of that survey, the location of the proposed GCSs was at the end of the existing NB1 and WB4 structures, respectively, and portions of the existing GCSs were buried. Per AMAFCA direction, the proposed location for NB0 was moved upstream, such that its crest will begin within the existing NB1 basin.

As part of the survey efforts, drone-based ortho imagery was also collected to facilitate the design.

Additional topographic survey was performed in January 2025. This survey focused on capturing the top of existing soil cement elevations at the proposed junction of the two GCSs.

PERMITTING EVALUATION

Through coordination with the U.S. Army Corps of Engineers (USACE), an evaluation of the need for Clean Water Act (CWA) Section 404 permitting was carried out by Tetra Tech. Tetra Tech conducted field assessments to delineate ordinary high-water marks, collected information for application of the Streamflow Duration Assessment Method, and prepared a precipitation and flow data analysis. Tetra Tech also conducted threatened and endangered species surveys and concluded that no federal or state protected special status species or habitats, or burrowing owl habitat, are present in the project area.

As a result of this evaluation, USACE determined that the project would not result in the discharge of dredged/fill material into waters of the United States, and as such, a Section 404 permit is not required. The USACE determination letter is included as **Attachment 1**.

This project is in a floodplain and a floodplain development permit (FDP) will be required. AMAFCA is managing this permitting internally.

HYDROLOGY

Design flow rates were taken from the *CAFP* (BHI and Tetra Tech, 2021) for NB0 and from the developed conditions option #3 (as recommended in the *CWB Management Plan* (BHI and Tetra Tech, 2019)) for WB0. The bulked 100-year peak discharge at the location of the proposed structures is approximately 14,100 cubic feet per second (cfs) in the Main Branch and 2,840 cfs in the West Branch (hereinafter referred to as the “individual” flowrate). In the most conservative 100-year flooding scenario, both GCSs would experience their 100-year peak discharge at the same time, making for a flowrate of 16,940 cfs in the dam pool (hereinafter referred to as the “coincidental” flowrate).

HYDRAULIC ANALYSIS

The GCSs were designed through an iterative process of developing and updating Hydraulic Engineering Circular No. 14 (HEC-14) hydraulic jump calculations, Civil3D design grading, and one-dimensional (1-D) Hydrologic Engineering Center's River Analysis System (HEC-RAS) hydraulic models. Iterative calculations will continue throughout the design process as changes are made to any of these elements. Each element is discussed in detail below.

GCS Sizing Calculations

GCS dimensions were estimated using the design procedure outlined in Chapter 8 of HEC-14 for expansion and depression for stilling basins. These calculations require an input of conditions at the start of an expected hydraulic jump to estimate the length and height of the jump. The results of these calculations were used to establish the GCS basin length, sill location, and the required sill height. These design parameters were used to model the proposed structures in the 1-D HEC-RAS models. The subsequent flow characteristics at the jump location in the 1-D HEC-RAS model were used to refine the GCS dimensions. Iterations of modeling and CGS sizing calculation updates were performed until both showed the hydraulic jump occurs within the armored portion of the structure. GCS sizing calculations are included as **Attachment 2**.

Grading

The grading of the GCSs was dependent on the GCS sizing calculations, the geotechnical stability analysis performed as part of the *Swinburne Dam Grade Control* Report by GeoTest (received by BHI on January 27, 2024), and the existing Swinburne Dam bank geometry. As the grading was changed, the modeling and GCS sizing calculations were updated to confirm the graded GCS dimensions were adequate. The 60% GCS dimensions are listed in **Table 1**.

Table 1. GCS Design Dimensions

Structure	Bottom Width (ft)	Side Slope	Drop Slope	Drop Height (ft)	Manning's n-value (Drop / Basin)	Sill Height (ft)	Basin Length (ft)
NB0	99	2H:1V	3H:1V	23	0.017 / 0.051	1.0	120
WB0	70	2H:1V	3H:1V	19	0.017 / 0.051	1.0	56

Hydraulic Modeling

1-D hydraulic modeling of the proposed structures was prepared using the USACE HEC-RAS software, version 6.4.1. Digital copies of the HEC-RAS models are included as **Attachment 3**. To develop the hydraulic input to GCS sizing (discussed in the GCS Sizing Calculations section above) and the scour calculations (see Design section below), models were prepared for each of the proposed GCSs under existing, future, and finished conditions.

Cross sections representing the geometry of the proposed structures were cut from the grading associated with the GCS design. To accurately represent the supercritical inflowing hydraulic conditions at the ties to the existing structures, cross sections through the NB1 and WB4 structures were included in the models based on the existing structure record drawings. Cross sections representing the low-head dam and excavated pool were extracted from grading representing a future condition, when applicable. This grading was developed by using a constant slope of approximately 0.0013 ft/ft from the proposed low-head dam elevation of 5,269.0 feet (Tetra Tech and BHI, 2021), to the proposed basin elevation of 5,270.0 feet and 5,271.0 feet for NB0 and WB0, respectively. This cross-sectional geometry was updated and iterated until the model results and grading surface complied with the design requirements discussed in the Design section of this memorandum and the structure functioned as intended.

A normal-depth downstream boundary condition was assigned to both models, with a slope of 0.00725 ft/ft that is consistent with the slope of the proposed pool downstream of the future low-head dam in the *CAFP* (Tetra Tech and BHI, 2021). A normal-depth upstream boundary condition was used for both models. The NB0 model used an upstream boundary condition slope of 0.02 ft/ft, representing the approximate existing slope upstream of the existing structure (GCS NB1). The WB0 model used an upstream boundary condition slope of 0.01 ft/ft that is representative of the existing ground tie between the GCS WB4 crest and the basin of the adjacent upstream structure (GCS WB3).

Friction losses associated with hydraulic roughness were accounted for using Manning's "n" values. The Manning's "n" values ranged from 0.017 to 0.08. The Manning's "n" value for the proposed grouted boulders was determined using Figure 9-3 of the Mile High Flood District (MHFD) *Urban Storm Drainage Criteria Manual, Volume 2* (2016), under the assumption the boulders will be 30-inch diameter and grouted to one-half the height of the boulders. A summary of Manning's "n" values that were used in the modeling are presented in **Table 2**.

Table 2. Hydraulic Roughness Parameters (Manning's n-values) Used in the HEC-RAS Modeling

Model Regions	Manning's n	Source
Existing Soil Cement GCSs (NB1 and WB4)	0.022	*NMDOT Drainage Design Manual* (*DDM*) (Smith Engineering Company (Smith Engineering) and Occam Engineers Inc. (OEI),2018), Table 502-3 (soil cement, typical)
Existing Dumped Riprap	0.08	*NMDOT DDM* (Smith Engineering and OEI, 2018), Table 502-3 (1-ft rock riprap)
Proposed Shotcrete	0.017	*NMDOT DDM* (Smith Engineering and OEI, 2018), Table 502-4 (shotcrete)
Proposed Grouted Boulder	0.051	MHFD *Urban Storm Drainage Criteria Manual, Volume 2* (2016), Figure 9-3.
Excavated Pond Bottom & Slopes	0.035	*NMDOT DDM* (Smith Engineering & OEI, 2018), Table 502-1 (Man-made channels – Earth with vegetation)

Ineffective flow areas were added to the cross-sections beyond the expected zones of flow expansion below the proposed GCS basins, assuming expansion occurs at 3:1. Energy losses in the flow expansion zones were accounted for by increasing the contraction and expansion coefficients from 0.1 and 0.3, respectively, to 0.3 and 0.5, respectively. To improve the resolution of the predicted hydraulics through the structures, interpolated cross sections were added with a maximum spacing of 6 feet.

A model plan was created for all flow regimes (subcritical, supercritical, and mixed) under all design conditions (existing, finished, and future) for both NB0 and WB0. The model plan used a flow file populated with two flowrate profiles; one for the individual flowrate and one for the coincidental flowrate. For the latter profile, the coincidental flowrate was added to the first cross section spanning the entire dam pool bottom width.

As expected, results from the modeling indicated that a substantial amount of backwater occurs upstream from the low-head dam in both GCS models (see example water surface profile in **Figure 2**). Although the models are not capable of determining the precise length of the hydraulic jump, they indicate the location and hydraulic conditions of the start of the jump. The predicted hydraulic parameters at key locations are presented in **Table 3** and **Table 4.**

Flow characteristics at the start of the hydraulic jump were used in the GCS sizing calculations for hydraulic jump length and height. These flow characteristics extracted for GCS sizing calculations were from a future conditions run, without a GCS sill, with coincidental flow under a mixed flow regime. These flow conditions were used to identify the starting location of a free hydraulic jump.

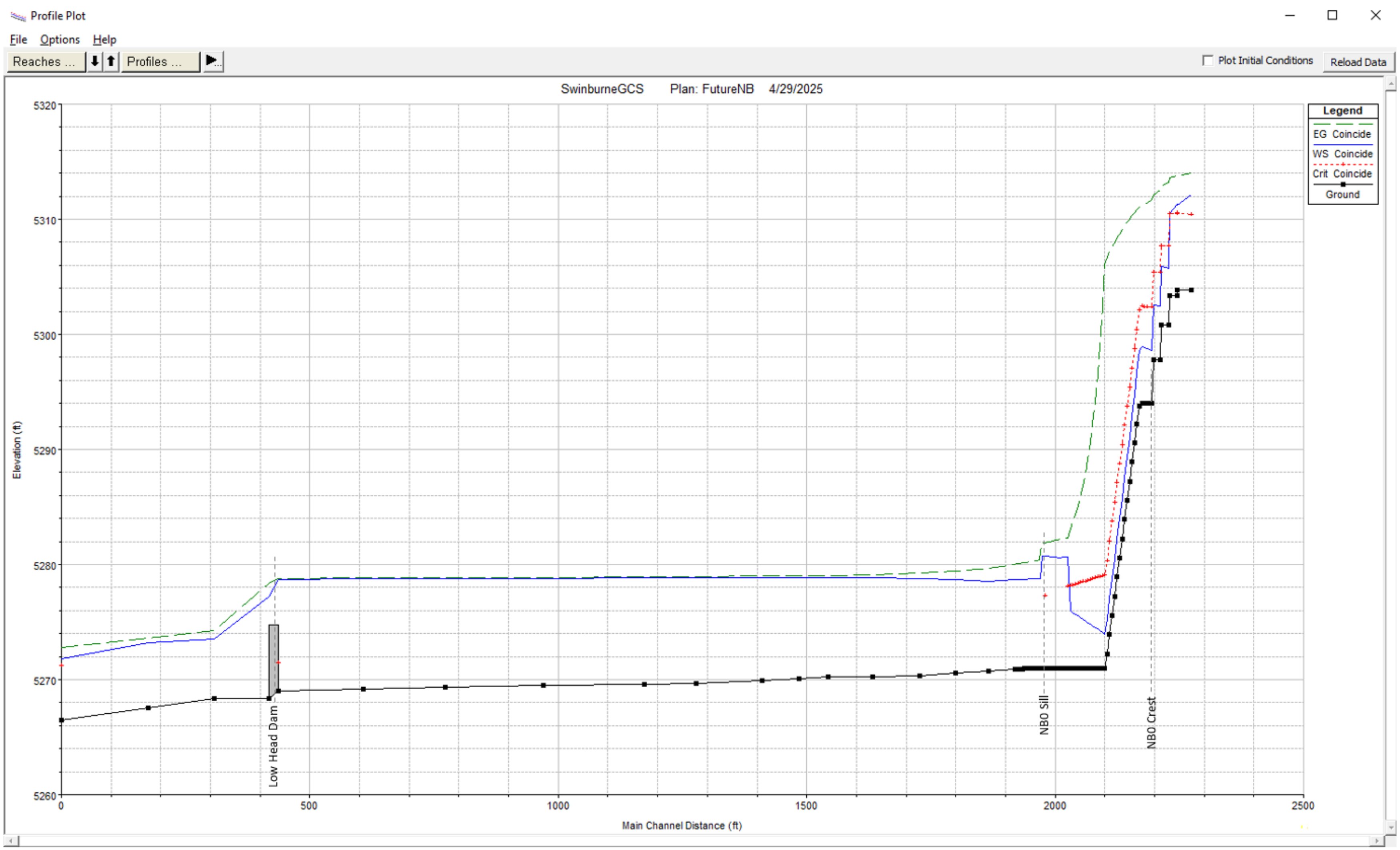

Figure 2. Computed Water Surface Elevation Profile from NB0 HEC-RAS Model

Table 3. Predicted Hydraulic Parameters at Key Locations in the NB0 HEC-RAS Model

Reach	River Station	Node Description	Flow Rate (cfs)	Minimum Channel Elevation (ft)	Water Surface Elevation (ft)	Critical Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Velocity (Channel) (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # (Channel)
NB	2273	Existing GCS Bank Protection	14,100	5,303.82	5,312.16	5,310.44	5,313.98	0.0040	10.99	1,320.63	178.49	0.67
NB	2244	Existing GCS NB1 Crest	14,100	5,303.32	5,311.30	5,310.49	5,313.76	0.0024	12.76	1,153.36	165.04	0.82
NB	2229	Existing GCS NB1 Drop Crest	14,100	5,303.32	5,310.50	5,310.50	5,313.65	0.0035	14.38	1,019.02	164.19	0.97
NB	2227	Existing GCS NB1	14,100	5,300.82	5,305.73	5,307.68	5,313.20	0.0130	22.23	656.99	140.2	1.77
NB	2213	Existing GCS NB1 Drop Crest	14,100	5,300.82	5,305.92	5,307.68	5,312.87	0.0115	21.44	686.16	148.55	1.67
NB	2211	Existing GCS NB1	14,100	5,297.82	5,302.45	5,305.43	5,312.52	0.0187	25.65	566.18	132.68	2.1
NB	2197	Existing GCS NB1 Drop Crest	14,100	5,297.82	5,302.54	5,305.41	5,312.12	0.0173	25.03	580.80	133.17	2.03
NB	2194	Existing GCS NB1 Basin	14,100	5,294.00	5,298.65	5,302.37	5,311.70	0.0242	29.29	499.17	117.14	2.4
NB	2184	Tie to NB1	14,100	5,294.00	5,298.80	5,302.42	5,311.40	0.0135	28.88	511.73	116.36	2.32
NB	2169	NB0 Crest	14,100	5,293.78	5,298.57	5,302.13	5,311.08	0.0136	28.8	507.13	115.1	2.33
NB	2100	Toe of NB0 Drop	14,100	5,271.00	5,273.97	5,279.12	5,306.05	0.5839	45.96	314.25	112.05	4.7
NB	1980	Above Sill of NB0 Basin	14,100	5,271.00	5,280.70	5,277.29	5,281.92	0.0040	8.45	1,640.62	208.33	0.48
NB	1968	Below Sill of NB0 Basin	14,100	5,271.00	5,278.78	-	5,280.45	0.0040	10.47	1,375.53	227.08	0.67
NB	1278	Inflow from West Branch	16,936	5,269.69	5,278.87	-	5,278.94	0.0001	2.06	8,273.22	958.27	0.12
NB	1173	-	16,936	5,269.62	5,278.85	-	5,278.92	0.0001	2.19	7,846.74	890.9	0.13
NB	970	-	16,936	5,269.53	5,278.81	-	5,278.89	0.0002	2.33	7,399.44	839.58	0.13
NB	772	-	16,936	5,269.31	5,278.78	-	5,278.86	0.0001	2.29	7,491.54	826.93	0.13
NB	607	-	16,936	5,269.16	5,278.76	-	5,278.84	0.0001	2.27	7,652.80	866.15	0.13
NB	436	Above Low-Head Dam	16,936	5,269.00	5,278.73	5,271.51	5,278.81	0.0001	2.31	7,434.91	820.28	0.13

Table 4. Predicted Hydraulic Parameters at Key Locations in the WB0 HEC-RAS Model

Reach	River Station	Node Description	Flow Rate (cfs)	Minimum Channel Elevation (ft)	Water Surface Elevation (ft)	Critical Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Velocity (Channel) (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # (Channel)
WB	1830	Existing GCS WB4	2,836	5,295.35	5,298.68	5,299.60	5,301.81	0.0100	14.37	207.79	83.89	1.43
WB	1827	Existing GCS WB4 Drop Crest	2,836	5,295.35	5,298.55	5,299.50	5,301.78	0.0109	14.53	204.14	86.55	1.48
WB	1825	Existing GCS WB4	2,836	5,293.00	5,297.54	5,298.86	5,301.65	0.0181	16.28	174.35	67.37	1.78
WB	1812	Existing GCS WB4 Drop Crest	2,836	5,293.00	5,295.32	5,297.06	5,301.19	0.0272	19.54	148.62	68.26	2.26
WB	1810	Existing GCS WB4 Basin	2,836	5,290.00	5,294.47	5,296.41	5,301.06	0.0380	20.62	138.75	67.52	2.49
WB	1753	Existing GCS WB4 Sill	2,836	5,290.00	5,294.82	5,295.24	5,296.81	0.0081	11.47	254.67	91.38	1.17
WB	1752.7	Tie to WB4	2,836	5,290.00	5,292.63	5,293.82	5,296.61	0.0096	16.27	180.42	74.39	1.77
WB	1732	WB0 Crest	2,836	5,289.97	5,292.94	5,293.85	5,296.09	0.0066	14.53	203.40	75.97	1.49
WB	1672	Toe of WB0 Drop	2,836	5,270.00	5,280.06	5,273.87	5,280.26	0.0008	3.78	823.16	103.25	0.21
WB	1616	Above Sill of WB0 Basin	2,836	5,270.00	5,280.08	5,273.31	5,280.21	0.0005	3.04	1,018.69	122.47	0.17
WB	1604	Below Sill of WB0 Basin	2,836	5,270.20	5,280.02	-	5,280.15	0.0002	2.89	1,053.47	131.85	0.17
WB	1400	Inflow from Main Branch	16,936	5,269.88	5,278.42	-	5,279.43	0.0021	8.15	2,119.68	862.13	0.49
WB	1334	-	16,936	5,269.88	5,278.56	-	5,279.21	0.0013	6.5	2,634.41	861.75	0.39
WB	1268	-	16,936	5,269.81	5,278.79	-	5,279.04	0.0005	4.01	4,230.54	859.82	0.24
WB	1068	-	16,936	5,269.62	5,278.76	-	5,278.94	0.0003	3.42	4,957.56	845.07	0.2
WB	869	-	16,936	5,269.44	5,278.75	-	5,278.87	0.0002	2.81	6,033.69	822.79	0.16
WB	632	-	16,936	5,269.16	5,278.72	-	5,278.82	0.0002	2.47	6,916.91	819.8	0.14
WB	436	Above Low-Head Dam	16,936	5,269.00	5,278.70	5,271.53	5,278.78	0.0001	2.32	7,454.78	822.65	0.13

DESIGN

During the conceptual design phase, alternatives for the location of the proposed structures, drop configurations, and construction materials were evaluated.

Based on consultation with AMAFCA, the 60% design preferred alternative for GCS NB0 locates the 15-foot long crest of the new structure within the footprint of the existing basin for GCS NB1, about 60 feet upstream of the existing GCS NB1 end sill. The 15-foot long crest of the new GCS WB0 will tie to the existing end sill of GCS WB4. Both structures will have 3H:1V slope shotcrete drops with 2H:1V side slopes. Shotcrete was selected for the sloped drops by AMAFCA for ease of maintenance.

The basins were designed using methods presented in Chapter 8 of FHWA's HEC-14 with input from the HEC-RAS modeling discussed above. The proposed drop heights are listed in **Table 1**. A flared shape was used for the footprint of the basin floors to better conform with the existing topography of the dam pool slopes and to take advantage of the energy losses that would result from flow expansion through the flaring of the basin. The NB0 basin flares from 99 feet to 145 feet. The WB0 basin flares from 70 feet to 190 feet. Results from the basin sizing calculations indicate that a basin length of 110 feet (85 feet for the energy loss prior to the hydraulic jump and 25 feet for the hydraulic jump) would be required for GCS NB0, and a basin length of 48 feet (18 feet for the energy loss prior to the hydraulic jump and 30 feet for the hydraulic jump) would be required for GCS WB0.

The hydraulic jump calculations indicate that the expected jump height in each basin will be greater than the tailwater within the dam. To ensure that the jump occurs within the armored portion of the structure, 1-foot high end sills were included at the downstream limit of GCS NB0 and WB0 basins. 2H:1V sloped scour protection is included downstream of the end sills to mitigate the risk of scour per recommendations in the AMAFCA *Sediment and Erosion Design Guide* [*Design Guide*] (Resource Consultants & Engineers, Inc. (RCE), 1994) Section 3.5. The design toe-down depth at the end of each basin matches the depth of toe-down required for the bank protection downstream of each GCS, discussed in the following section.

Scour Calculations

Results from HEC-RAS hydraulic modeling were used as input to scour calculations to design the toe-down for the bank protection. Plunge scour along the downstream face of the basin sill will be mitigated by the sloped configuration of the proposed toe-down; thus, the estimated scour depths for the bank protection were used to set the toe-down depths. For both the NB0 and WB0 structures, the future conditions, subcritical, individual flowrate run was used to perform these scour depth calculations. This run gave the most realistic conditions in which scour would occur and resulted in the most conservative scour depths expected. Scour calculations were prepared to estimate the potential for revetment toe scour using Equation 4.3 of the *Design Guide* (RCE, 1994). Although this equation is intended to predict scour along a flood wall, the helical flow patterns represented by the equation would be expected to be similar to those along the toe of the bank protection and downstream of the sill. The results indicate that the bank protection and basin toe-down would need to be toed down to a depth of 6 feet and 4.5 feet for GCS NB0 and GCS WB0, respectively. As design progresses, these scour depths should be reevaluated based on refinements to the design and subsequent modeling updates. Scour calculations are included in **Attachment 4**.

Top of Bank Protection Calculations

To determine the top of bank protection elevation, the adequate height of freeboard was calculated at each cross section. Results from HEC-RAS hydraulic modeling were used as input to freeboard calculations. For both the NB0 and WB0 structures, the finished conditions, subcritical, coincidental flowrate run was used to perform the freeboard calculations. This run produces the most conservative (greatest) sum of water surface elevation and freeboard. Freeboard calculations were prepared using Equation 6.35 of the *Development Process Manual* (COA, 2020). This equation is applicable to trapezoidal channels with flowrates greater than 100 cfs, which is true of both the NB0 and WB0 structures. The results indicate that the necessary freeboard varies over the profile of the GCSs from 2.0 feet to 1.7 feet and 1.7 feet to 1.6 feet for NB0 and WB0, respectively. Adding the freeboard height to the water surface elevation at each respective cross section resulted in the minimum necessary top of bank protection elevation at each cross section. The minimum top of bank protection elevations vary over the profile of the GCSs from 5,304.1 feet to 5,298.4 feet and from 5,295.6 feet to 5,292.0 feet for NB0 and WB0, respectively. Freeboard calculations are included in **Attachment 5**.

Other Considerations

Maintenance Access

Maintenance access ramps near both GCSs are proposed to provide access to the GCSs during post construction and future excavation conditions. Current design plans show the ramps constructed to the finished conditions surface. The ramps can be extended at a maximum slope of 10% into the dam pool as it becomes fully excavated in future conditions.

Material Selection

The proposed NB0 and WB0 GCSs were designed with 8-inch shotcrete channel lining, structural concrete, and grouted boulders. The GCSs’ crests and drops were designed with 8-inch shotcrete channel lining. This shotcrete channel lining is robust enough to handle the expected velocities at the drop and is regularly used by AMAFCA for ease of maintenance. Structural concrete will be used at the toe of the shotcrete as a cutoff wall to be connected to the grouted boulders. Grouted boulders were selected to increase the channel roughness to force a hydraulic jump and dissipate energy within the basin. Additionally, grouted boulders were selected to handle the high velocities and shear stresses expected at the toe of a shotcrete drop.

Shotcrete Connection

The proposed NB0 and WB0 GCSs were designed to connect to designated faces of the existing soil cement structures NB1 and WB4, respectively. The proposed shotcrete follows the approximate existing structure slope by connecting to the toes of the steps. As would be expected, the edges of the existing structures are eroded to varying levels - concentrated typically in the top exposed layers. The design details will indicate to the contractor to use a combination of a partial-depth saw cut (at the surface) and hand (10 lb. hammer) removal for the lower levels to create a clean, durable connection to the new proposed shotcrete. To mitigate differential settlement of the existing and proposed structures, the design will indicate drilling and epoxying dowels into the existing structure at appropriate locations. Those dowels will then to be lapped to the proposed shotcrete reinforcing. The design and construction plans will determine the length, size, and depth of embedment for the dowels.

CONCLUSION

Several iterations of hydraulic modeling, calculations, and design were performed to develop the 60% design of the Swinburne Dam GCSs. As revisions are made to the design, additional analyses and iterations may be necessary.

EA/RS/ST/AE/ab

Attachments:

Attachment 1 – USACE Determination Letter
Attachment 2 – GCS Sizing Calculations
Attachment 3 – Hydraulic Models (HEC-RAS) (see digital files attached separately)
Attachment 4 – Scour Calculations
Attachment 5 – Bank Protection Calculations

Attachment 1 – USACE Determination Letter

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS ALBUQUERQUE DISTRICT REGULATORY DIVISION
NEW MEXICO & NW TEXAS BRANCH, ALBUQUERQUE OFFICE
4101 JEFFERSON PLAZA NE
ALBUQUERQUE, NEW MEXICO 87109-3435

May 9, 2024

Regulatory Division

SUBJECT: No Permit Required – (SPA-2024-00128)

AMAFCA
Attn: Adrienne Martinez
2600 Prospect Ave NE
Albuquerque, NM 87107
AMartinez@amafca.org

Dear Ms. Martinez:

We are responding to your request for a determination of Department of the Army permit requirements for the 0.46-acre review area located in the Calabacillas Arroyo Main Branch centered at latitude35.207689°, longitude -106.712595° and the Calabacillas Arroyo West Branch a 0.31-acre parcel approximate center point latitude 35.205684°, longitude -106.712003°, in the City of Albuquerque ,Bernalillo County, New Mexico. We have assigned Action No. SPA-2024-00128 to this project. Please reference this number in all future correspondence concerning the project.

Based on the information provided, we have determined that a Department of the Army permit is not required since the project would not result in the discharge of dredged/fill material into waters of the United States. However, it is incumbent upon you to remain informed of any changes in the U.S. Army Corps of Engineers (Corps) Regulatory Program regulations and policy as they relate to your project. If your plans change such that waters of the United States could be impacted by the proposed project, please contact our office for a reevaluation of permit requirements.

We have determined that the Calabacillas Arroyo Main Branch and the Calabacillas Arroyo West Branch review areas are not waters of the United States because they do not meet the Definition (3) Tributary, that are relatively permanent, standing or continuously flowing bodies of water.

We are enclosing a copy of the Approved Jurisdictional Determination Form for your review area (Enclosure 2). A copy of this JD is also available at http://www.spa.usace.army.mil/reg/JD. This approved JD is valid for 5 years from the date of this letter unless new information warrants revision of the determination before the expiration date. If you intend to conduct work that could result in a discharge of dredged or fill material into waters of the United States, please contact this office for a determination of Department of the Army permit requirement.

The delineation included herein has been conducted to identify the location and extent of the aquatic resources for purposes of the Clean Water Act for the particular site identified in this request. This delineation may not be valid for the Wetland Conservation Provisions of the Food Security Act of 1985, as amended. If you or your tenant are USDA program participants, or anticipate participation in USDA programs, you should discuss the applicability of an NRCS Certified Wetland Determination with the local USDA service center, prior to starting work.

You may accept or appeal this approved JD or provide new information in accordance with the attached Notification of Administration Appeal Options and Process and Request for Appeal (Enclosure 3). If you elect to appeal this approved JD, you must complete Section II of the form and return it to the Army Engineer Division, South Pacific, CESPD-PDS-O, Attn: Travis Morse, Administrative Appeal Review Officer, by email at w.travis.morse@usace.army.mil within 60 days of the date of this notice. Failure to notify the Corps within 60 days of the date of this notice means that you accept the approved JD in its entirety and waive all rights to appeal the approved JD.

Please refer to identification number SPA-2024-00128 in any correspondence concerning this project. If you have any questions, please contact me by email at Forrest.Luna@usace.army.mil or telephone at (505) 342-3678.

Sincerely,

Forrest Luna
Regulatory Specialist
New Mexico / West Texas Branch
Albuquerque District

Enclosures

cc: Ondrea Hummel, Tetra Tech, ondrea.hummel@tetratech.com

Attachment 2 – GCS Sizing Calculations

Sloped Grade Control Structure Sizing

Project Name: **Swinburn Dam GCSs**
Project Number: **20240392**
Prepared By: **Emma Adams**
Date: **2/10/2025**

Color Key				
Input	Linked Cell	Calculation	Chk OK	Chk Not OK

HYDRAULIC COMPUTATIONS TO DETERMINE SLOPED GRADE CONTROL STRUCTURE DIMENSIONS, BASED ON CHAPTERS 8 HEC-14

HEC-14: http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf

Drop Structure ID (1)	Flow Profile (2)	Flow Rate Q (cfs) (3)	Initial TW Ratio C (4)	Tailwater XS (Sta ft) (5)	Tailwater TW (ft) (6)	Supercritical XS (Sta ft) (7)	Supercritical Depth Y_1 (ft) (8)	Supercritical Velocity v_1 (fps) (9)	Supercritical Froude (10)	Drop Toe Froude # Fr_1 (11)	Jump Height Y_2 (ft) (12)	L_b/Y_2 Fig. 8.2 Pg. 8-4 (13)	Jump Length L_b (ft) (14)	Is TW > Y2? If yes, no sill wall required C (15)	Sill Wall Height Z_3 (ft) (16)	End Sta of Basin (ft) (17)	Toe of Drop Slope (Sta ft) (18)	Total Necessary Basin Length (ft) (19)	Design Basin Length (ft) (20)	RAS River Sta for Start of Slope (ft) (21)	RAS River Sta for US Top of Sill (ft) (22)	RAS River Sta for DS Top of Sill (ft) (23)	RAS River Sta for Bottom of Slope (ft) (24)
Excavated NB_NoSill	NB Only	14100	1.00	1799	7.60	2010	6.2	16.68	1.18	1.18	7.65	3.25	24.9	Sill Wall	0.1	1985.13	2095.00	109.87	115.00	1980.00	1980.00	1976.00	1976.00
Excavated WB_NoSill	WB Only	2836	1.00	1400	6.02	1654	2.0	20.14	2.49	2.49	6.21	4.80	29.8	Sill Wall	0.2	1624.22	1672.00	47.78	50.00	1622.00	1622.00	1618.00	1618.00

Notes:

1) HEC-RAS Plan
2) Flow profile from HEC-RAS Steady Flow
3) "NB Only" from Calabacillas Arroyo Facility Plan Above Swinburne Dam (Tetra Tech & BHI, 2021); "WB Only" from the Calabacillas West Branch Arroyo Drainage and Storm Water Quality Management Plan (Tetra Tech & BHI, 2019).
4) Tailwater ratio as defined by ratio of tailwater to conjugate depth = TW/y_2. C-1 for a free jump
5) Cross section used in tailwater determination taken as 2 cross sections past the end of post-sill interpolated sections
6) Tailwater depth (WSEL - Min. Ch Elev) at Tailwater XS
7) Supercritical XS taken as the last supercritical XS before the basin hydraulic jump
8) Supercritical depth (WSEL - Min. Ch Elev) at Supercritical XS
9) Supercritical velocity from HEC-RAS Results at Supercritical XS
10) Supercritical froude from HEC-RAS Results at Supercritical XS
11) Calculated Froude no. at upstream limit of hydraulic jump (used at a check that spreadsheet is pulling the right XS information. (Not used in remaining calculations)

12) Equation 8.4 (below) used to calculate congugate depth at end of jump (upstream from end sill)
(13) Ratio of hydraulic jump length to height (Figure 8.2 of HEC-14)
(14) Length of hydraulic jump = Y2 * Lb/Y2
(15) Determination of sill wall requirement (Required if Y2>TW)
(16) Height of end sill as measured from basin floor = Y2 - TW
(17) End RAS station of basin necessary = Supercritical XS - Lb
(18) Toe of drop slope as graded in Civil3D
(19) Total necessary basin length = toe of drop - end sta of basin
(20) Rounded basin length
(21) RAS River Sta for Start of Slope = toe of drop slope - necessary basin length
(22) RAS River Sta for US Top of Sill assumes a 6:1 slope on the sill US side (consistent with previous grading)
(23) RAS River Sta for DS Top of Sill assumes a 4' sill width (consistent with previous grading)
(24) RAS River Sta for Bottom of Slope assumes a 2:1 slope on the sill DS side (consistent with previous grading)

8.1 EXPANSION AND DEPRESSION FOR STILLING BASINS

As explained in Chapter 4, the higher the Froude number at the entrance to a basin, the more efficient the hydraulic jump and the shorter the resulting basin. To increase the Froude number as the water flows from the culvert to the basin, an expansion and depression is used as is shown in Figure 8.1. The expansion and depression converts depth, or potential energy, into kinetic energy by allowing the flow to expand, drop, or both. The result is that the depth decreases and the velocity and Froude number increase.

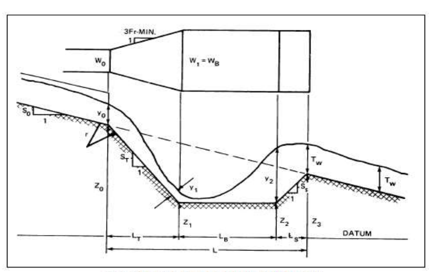

Figure 8.1. Definition Sketch for Stilling Basin

The Froude number used to determine jump efficiency and to evaluate the suitability of alternative stilling basins as described in Table 8.1 is defined in Equation 8.1.

$$Fr_1 = \frac{V_1}{\sqrt{gy_1}} \quad (8.1)$$

where,
Fr_1 = Froude number at the entrance to the basin
V_1 = velocity entering the basin, m/s (ft/s)
y_1 = depth entering the basin, m (ft)
g = acceleration due to gravity, m/s² (ft/s²)

To solve for the velocity and depth entering the basin, the energy balance is written from the culvert outlet to the basin. Substituting $Q/(y_1W_B)$ for V_1 and solving for Q results in:

8-2

$$Q = y_1W_B\left[2g(z_o - z_1 + y_o - y_1) + V_o^2\right]^{1/2} \quad (8.2)$$

where,
W_B = width of the basin, m/s (ft/s)
V_o = culvert outlet velocity, m/s (ft/s)
y_1 = depth entering the basin, m (ft)
y_o = culvert outlet depth, m (ft)
z_1 = ground elevation at the basin entrance, m (ft)
z_o = ground elevation at the culvert outlet, m (ft)

Equation 8.2 has three unknowns y_1, W_B, and z_1. The depth y_1 can be determined by trial and error if W_B and z_1 are assumed. W_B should be limited to the width that a jet would flare naturally in the slope distance L.

$$W_B \le W_o + \frac{2L_T\sqrt{S_T^2+1}}{3Fr_o} \quad (8.3)$$

where,
L_T = length of transition from culvert outlet to basin, m (ft)
S_T = slope of the transition, m/m (ft/ft)
Fr_o = outlet Froude number

Since the flow is supercritical, the trial y_1 value should start near zero and increase until the design Q is reached. This depth, y_1, is used to find the sequent (conjugate) depth, y_2, using the hydraulic jump equation:

$$y_2 = \frac{Cy_1}{2}\left(\sqrt{1+8Fr_1^2} - 1\right) \quad (8.4)$$

where,
y_2 = conjugate depth, m (ft)
y_1 = depth approaching the jump, m (ft)
C = ratio of tailwater to conjugate depth, TW/y_2
Fr_1 = approach Froude number

For a free hydraulic jump, C = 1.0. Later sections on the individual stilling basin types provide guidance on the value of C for those basins. For the jump to occur, the value of $y_2 + z_2$ must be equal to or less than $TW + z_3$ as shown in Figure 8.1. If $z_2 + y_2$ is greater than $z_3 + TW$, the basin must be lowered and the trial and error process repeated until sufficient tailwater exists to force the jump.

In order to perform this check, z_3 and the basin lengths must be determined. The length of the transition is calculated from:

$$L_T = \frac{z_o - z_1}{S_T} \quad (8.5)$$

8-3

where,
L_T = length of the transition from the culvert outlet to the bottom of the basin, m (ft)
S_T = slope of the transition entering the basin, m/m (ft/ft)

The length of the basin, L_B, depends on the type of basin, the entrance flow depth, y_1, and the entrance Froude number, Fr_1. Figure 8.2 describes these relationships for the free hydraulic jump as well as several USBR stilling basins.

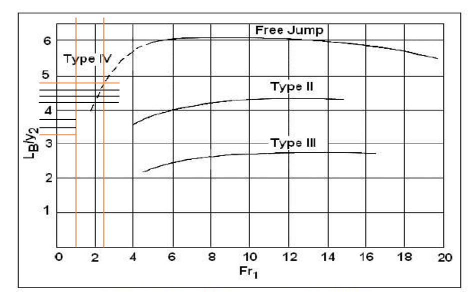

Figure 8.2. Length of Hydraulic Jump on a Horizontal Floor

The length of the basin from the floor to the sill is calculated from:

$$L_S = \frac{L_T(S_T - S_o) - L_BS_o}{S_S + S_o} \quad (8.6)$$

where,
L_S = length of the basin from the bottom of the basin to the basin exit (sill), m (ft)
S_S = slope leaving the basin, m/m (ft/ft)

The elevation at the entrance to the tailwater channel is then calculated from:

$$z_3 = L_SS_S + z_1 \quad (8.7)$$

8-4

Attachment 3 – Hydraulic Models (HEC-RAS)

(see digital files attached separately)

Attachment 4 – Scour Calculations

Scour Analysis - Flow Parallel to a Wall

Project Name: **Swinburne GCS**
Project Number: **20240392**
Prepared By: **ELA**
Date: **2/18/2025**

Color Key		
Input	Linked Cell	Calculation

Calculations reference Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Hydraulic Engineering Circular No. 23 (HEC-23)

HEC-RAS Plan (1)	Flow Profile (1)	Flow Rate (cfs)	Cross Section (Near end of sill)	Min. Channel (ft) (1)	W.S. Elev (ft) (1)	Froude # (Fr) (--) (1)	Flow Area (ft^2) (1)	Top Width (ft) (1)	Hydraulic Depth (Y) (ft) (2)	Scour Depth (Y_S) (ft) (3)
Excavated NB - supercritical	Coincide	14100	1968	5271.0	5277.0	1.00	1042.94	217.50	4.80	5.61
Excavated NB - supercritical	NB Only	14100	1968	5271.0	5277.0	1.00	1042.94	217.50	4.80	5.61
Excavated NB - subcritical	Coincide	14100	1968	5271.0	5278.8	0.67	1376.36	227.11	6.06	5.62
Excavated NB - subcritical	NB Only	14100	1968	5271.0	5278.4	0.73	1293.53	224.73	5.76	5.55
Excavated WB - supercritical	Coincide	2836	1599	5270.0	5272.5	1.31	247.15	104.62	2.36	3.51
Excavated WB - supercritical	WB Only	2836	1599	5270.0	5272.5	1.31	247.15	104.62	2.36	3.51
Excavated WB - subcritical	Coincide	2836	1599	5270.0	5280.0	0.15	1161.73	138.63	8.38	6.20
Excavated WB - subcritical	WB Only	2836	1599	5270.0	5275.9	0.34	635.33	120.75	5.26	4.11

Notes:

1) Excavated terrain to get the most conservative scour. Supercritical run used to calculation scour according to AMAFCA technical standards (below).

AMAFCA Technical Standards Manual FINAL 2024 1031_ELA

4.3.2.1 Flow Regimes

Subcritical Flows

Subcritical flow is required for the design of all arroyos with natural, riprap, geogrid, and other semi-permeable linings. Supercritical flows are not allowed for these lining types. Subcritical flows are used to determine flood depths within concrete channels. FEMA analysis must use subcritical flow for WSE and mapping.

Supercritical Flows

Supercritical flows are used for scour analysis in natural channels and for transitions from natural to lined channels (or vice versa). In addition, supercritical flows are necessary for the calculation of velocities and the determination of concrete erosivity to design thickened layers in concrete channels.

2) Hydraulic Depth calculated using HEC-RAS inputs. Y = Flow Area / Top Width
3) Scour depth computed using equation 4.3 in HEC-23
where, Y_s = Scour Depth; Y = Hydraulic Depth in Channel; Fr = Froude #

$$\boxed{Equation\ 4.3\ -\ Y_s = Y(0.73 + 0.14\pi F_r^2)}$$

Attachment 5 – Bank Protection Calculations

Bank Protection Sizing

Project Name: **Swinburn Dam GCSs**
Project Number: **20240392**
Prepared By: **Emma Adams**
Date: **2/17/2025**

Color Key				
Input	Linked Cell	Calculation	Chk OK	Chk Not OK

Location	Cross Section (1)	Description	Coincide Min Ch El (ft)	Coincide W.S. Elev (ft) (2)	Coincide Vel Chnl (ft/s)	Coincide Freeboard (ft) (3)	Coincide Top of BP Elevation (ft) (4)	NB Only Min Ch El (ft)	NB Only W.S. Elev (ft) (2)	NB Only Vel Chnl (ft/s)	NB Only Freeboard (ft) (3)	NB Only Top of BP Elevation (ft) (4)	Maximums Freeboard (ft)	Maximums Top of BP Elevation (ft) (5)
North Branch	2169	DS Edge of Crest	5293.8	5302.1	16.0	2.0	5304.1	5293.8	5302.1	16.0	2.0	5304.1	2.0	5304.1
	2164	-	5292.3	5300.4	16.0	2.0	5302.4	5292.3	5300.4	16.0	2.0	5302.4	2.0	5302.4
	2159	-	5291.9	5299.8	14.6	1.9	5301.7	5291.9	5299.8	14.6	1.9	5301.7	1.9	5301.7
	2154	-	5291.9	5299.4	14.4	1.9	5301.3	5291.9	5299.4	14.4	1.9	5301.3	1.9	5301.3
	2149	-	5291.8	5299.2	14.3	1.9	5301.0	5291.8	5299.2	14.3	1.9	5301.0	1.9	5301.0
	2144	-	5291.7	5298.8	14.1	1.9	5300.7	5291.7	5298.8	14.1	1.9	5300.7	1.9	5300.7
	2139	-	5291.7	5298.5	14.0	1.9	5300.4	5291.7	5298.5	14.0	1.9	5300.4	1.9	5300.4
	2134	-	5291.6	5298.3	14.0	1.9	5300.1	5291.6	5298.3	14.0	1.9	5300.1	1.9	5300.1
	2129	-	5291.5	5298.0	13.8	1.9	5299.9	5291.5	5298.0	13.8	1.9	5299.9	1.9	5299.9
	2124	-	5291.5	5297.8	13.8	1.8	5299.6	5291.5	5297.8	13.8	1.8	5299.6	1.8	5299.6
	2119	-	5291.4	5297.6	13.8	1.8	5299.5	5291.4	5297.6	13.8	1.8	5299.5	1.8	5299.5
	2114	-	5291.3	5297.4	13.7	1.8	5299.3	5291.3	5297.4	13.7	1.8	5299.3	1.8	5299.3
	2109	-	5291.3	5297.3	13.6	1.8	5299.2	5291.3	5297.3	13.6	1.8	5299.2	1.8	5299.2
	2104	-	5291.2	5297.2	13.5	1.8	5299.1	5291.2	5297.2	13.5	1.8	5299.1	1.8	5299.1
	2100	Bottom of Proposed Drop	5291.1	5297.1	13.6	1.8	5299.0	5291.1	5297.1	13.6	1.8	5299.0	1.8	5299.0
	2095	-	5291.1	5297.0	13.5	1.8	5298.9	5291.1	5297.0	13.5	1.8	5298.9	1.8	5298.9
	2090	-	5291.0	5296.9	13.5	1.8	5298.8	5291.0	5296.9	13.5	1.8	5298.8	1.8	5298.8
	2085	-	5290.9	5296.9	13.4	1.8	5298.7	5290.9	5296.9	13.4	1.8	5298.7	1.8	5298.7
	2080	-	5290.9	5296.8	13.4	1.8	5298.6	5290.9	5296.8	13.4	1.8	5298.6	1.8	5298.6
	2075	-	5290.8	5296.7	13.4	1.8	5298.5	5290.8	5296.7	13.4	1.8	5298.5	1.8	5298.5
	2070	-	5290.8	5296.7	13.2	1.8	5298.5	5290.8	5296.7	13.2	1.8	5298.5	1.8	5298.5
	2065	-	5290.7	5296.7	13.1	1.8	5298.5	5290.7	5296.7	13.1	1.8	5298.5	1.8	5298.5
	2060	-	5290.6	5296.7	13.0	1.8	5298.5	5290.6	5296.7	13.0	1.8	5298.5	1.8	5298.5
	2055	-	5290.6	5296.6	12.9	1.8	5298.4	5290.6	5296.6	12.9	1.8	5298.4	1.8	5298.4
	2050	-	5290.5	5296.6	12.9	1.8	5298.4	5290.5	5296.6	12.9	1.8	5298.4	1.8	5298.4
	2045	-	5290.4	5296.6	12.7	1.8	5298.4	5290.4	5296.6	12.7	1.8	5298.4	1.8	5298.4
	2040	-	5290.3	5296.6	12.3	1.8	5298.4	5290.3	5296.6	12.3	1.8	5298.4	1.8	5298.4
	2035	-	5290.3	5296.7	11.9	1.8	5298.5	5290.3	5296.7	11.9	1.8	5298.5	1.8	5298.5
	2030	-	5290.2	5296.7	11.7	1.8	5298.5	5290.2	5296.7	11.7	1.8	5298.5	1.8	5298.5
	2025	-	5290.2	5296.7	11.5	1.8	5298.5	5290.2	5296.7	11.5	1.8	5298.5	1.8	5298.5
	2020	-	5290.1	5296.7	11.4	1.8	5298.5	5290.1	5296.7	11.4	1.8	5298.5	1.8	5298.5
	2015	-	5290.0	5296.7	11.3	1.8	5298.5	5290.0	5296.7	11.3	1.8	5298.5	1.8	5298.5
	2010	-	5289.9	5296.7	11.2	1.8	5298.5	5289.9	5296.7	11.2	1.8	5298.5	1.8	5298.5
	2005	-	5289.7	5296.7	11.1	1.8	5298.5	5289.7	5296.7	11.1	1.8	5298.5	1.8	5298.5
	1999	-	5289.6	5296.7	10.9	1.8	5298.5	5289.6	5296.7	10.9	1.8	5298.5	1.8	5298.5
	1995	-	5289.7	5296.7	10.9	1.8	5298.5	5289.7	5296.7	10.9	1.8	5298.5	1.8	5298.5
	1990	-	5289.7	5296.7	10.8	1.8	5298.5	5289.7	5296.7	10.8	1.8	5298.5	1.8	5298.5
	1984		5289.7	5296.7	10.7	1.8	5298.5	5289.7	5296.7	10.7	1.8	5298.5	1.8	5298.5
	1980	US side of Sill	5289.7	5296.7	10.4	1.7	5298.5	5289.7	5296.7	10.4	1.7	5298.5	1.7	5298.5
	1968	DS side of Sill	5289.4	5296.7	10.1	1.7	5298.5	5289.4	5296.7	10.1	1.7	5298.5	1.7	5298.5

Location	Cross Section	Description	Coincide					WB Only					Maximums	
	(1)		Min Ch El (ft)	W.S. Elev (ft) (2)	Vel Chnl (ft/s)	Freeboard (ft) (3)	Top of BP Elevation (ft) (4)	Min Ch El (ft)	W.S. Elev (ft) (2)	Vel Chnl (ft/s)	Freeboard (ft) (3)	Top of BP Elevation (ft) (4)	Freeboard (ft)	Top of BP Elevation (ft) (5)
West Branch	1732	DS Edge of Crest	5290.0	5293.9	10.9	1.7	5295.6	5290.0	5293.9	10.9	1.7	5295.6	1.7	5295.6
	1726	-	5288.3	5292.3	11.0	1.7	5294.0	5288.3	5292.3	11.0	1.7	5294.0	1.7	5294.0
	1720	-	5286.3	5290.3	11.0	1.7	5292.0	5286.3	5290.3	11.0	1.7	5292.0	1.7	5292.0
	1714	-	5285.5	5290.9	6.6	1.6	5292.5	5285.5	5289.2	10.4	1.7	5290.9	1.7	5292.5
	1708	-	5285.4	5291.0	5.9	1.6	5292.6	5285.4	5288.9	10.2	1.7	5290.5	1.7	5292.6
	1702	-	5285.3	5291.1	5.4	1.6	5292.7	5285.3	5288.6	10.0	1.7	5290.2	1.7	5292.7
	1696	-	5285.3	5291.1	5.1	1.6	5292.7	5285.3	5288.3	9.8	1.7	5290.0	1.7	5292.7
	1690	-	5285.2	5291.2	4.8	1.6	5292.7	5285.2	5288.2	9.7	1.6	5289.8	1.6	5292.7
	1684	-	5285.1	5291.2	4.6	1.5	5292.7	5285.1	5288.1	9.6	1.6	5289.7	1.6	5292.7
	1678	-	5285.0	5291.2	4.4	1.5	5292.7	5285.0	5287.9	9.5	1.6	5289.6	1.6	5292.7
	1672	Bottom of Proposed Drop	5285.0	5291.2	4.2	1.5	5292.7	5285.0	5287.8	9.4	1.6	5289.5	1.6	5292.7
	1666	-	5284.9	5291.2	4.0	1.5	5292.7	5284.9	5287.7	9.3	1.6	5289.3	1.6	5292.7
	1660	-	5284.8	5291.2	3.8	1.5	5292.7	5284.8	5287.6	9.2	1.6	5289.2	1.6	5292.7
	1654	-	5284.8	5291.2	3.7	1.5	5292.7	5284.8	5287.5	9.0	1.6	5289.1	1.6	5292.7
	1648	-	5284.7	5291.2	3.5	1.5	5292.7	5284.7	5287.4	9.0	1.6	5289.0	1.6	5292.7
	1642	-	5284.6	5291.2	3.4	1.5	5292.7	5284.6	5287.3	8.9	1.6	5289.0	1.6	5292.7
	1636	-	5284.6	5291.2	3.3	1.5	5292.7	5284.6	5287.3	8.9	1.6	5288.9	1.6	5292.7
	1630	-	5284.5	5291.2	3.2	1.5	5292.7	5284.5	5287.2	8.8	1.6	5288.8	1.6	5292.7
	1624	-	5284.4	5291.2	3.2	1.5	5292.7	5284.4	5287.1	8.8	1.6	5288.7	1.6	5292.7
	1618	-	5284.4	5291.3	3.1	1.5	5292.8	5284.4	5287.0	8.7	1.6	5288.6	1.6	5292.8
	1616	US side of Sill	5284.3	5291.3	3.0	1.5	5292.8	5284.3	5287.0	8.7	1.6	5288.6	1.6	5292.8
	1604	DS side of Sill	5284.2	5291.3	2.8	1.5	5292.8	5284.2	5286.9	8.3	1.6	5288.5	1.6	5292.8

Notes:

(1) Calculated for cross sections from the toe of the proposed drop slope to the downstream end of the sill
(2) Min Ch El, W.S. Elev, and Vel Chnl taken from HEC-RAS output for the subcritical, finished, plans for both locations (FinishedNB_Sub and FinishedWB_Sub)
(3) Freeboard calculated per COA DPM Equation 6.35 which is for trapezoidal channels with flow rates of 100 c.f.s. or greater and average flow velocity of 35 ft/sec or greater (for open channels, AMAFCA requires designers to use the requirements of the respective jurisdiction - CABQ, NMDOT, or Bernalillo County)
(4) Necessary top of BP Elevation at each XS = WSEL + freeboard
(5) Maximums takes the maximum freeboard and elevation for any given XS between the coincide vs N/WB Only flow profiles (NOTE: these freeboard and elevation values may be from different cross sections)

6-8(D)(8)(ii) Trapezoidal Channels and Associated Types

Adequate channel freeboard above the designed water surface must be provided and shall not be less than the channel freeboard determined in the following 2 equations:

1. For flow rates of less than 100 c.f.s. and average flow velocity of less than 35 ft/sec:

   EQUATION 6.34 **Freeboard (Feet)** $= 1.0 + 0.025Vd^{1/3}$

2. For flow rates of 100 c.f.s. or greater and average flow velocity of 35 ft/sec or greater:

   EQUATION 6.35 **Freeboard (Feet)** $= 0.7\ (2.0 + 0.025Vd^{1/3})$

Freeboard will be in addition to any superelevation of the water surface, standing waves and/or other water surface disturbances. When the total expected height of disturbances is less than 0.5 feet, disregard their contribution.