Continuing Authorities Program
Section 1135 Project

Engineering Appendix

for

Rio Anton Ruiz Restoration Project

Municipality of Humacao, Puerto Rico

to the

Draft Integrated Feasibility Report

and Environmental Assessment

17 February 2017

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ATTACHMENTS

1.0 Purpose

The purpose of this engineering appendix to the feasibility report (aka detailed project report) is to discuss the methods and plans for solutions to reduce the salinity levels for the Rio Anton Ruiz Restoration project, Humacao, Puerto Rico, Continuing Authorities Program (CAP) Section 1135 (Project Modifications for Improvements to the Environment) that was introduced after construction of a prior CAP Section 205 flood control project. This engineering appendix will include alternatives evaluated, costs and benefits, preliminary designs, and recommendations. Upon approval, this document will be included as an appendix to the Final Integrated Feasibility Report.

2.0 Project Background

2.1 Location

The project is located in the Municipality of Humacao on the southeast coast of Puerto Rico. See **Figure 1** below.

Figure 1. Project Location

2.2 Original Project (Section 205)

The authorized purpose of the Río Antón Ruiz CAP Section 205 flood control project ("Rio Anton Ruiz at Humacao, Puerto Rico, Final Detailed Project Report and Environmental Assessment, Section 205, Flood Control", dated October 1993) is to reduce flooding damages from the Rio Anton Ruiz to the communities of Punta Santiago, Verde Mar, and Villa Palmira. To achieve the authorized purpose, the project has the following features:

- 11,870 feet (3,619 meters) of standard project flood (SPF) levee for flood protection
- 5,150 feet (1,570 meters) of diversion channel
- 8,270 feet (2,521 meters) of interior drainage ditch, collecting the drainage from the interior communities and outfalling to the diversion channel and Boca Prieta outlet
- 127-foot long, three-barrel, 72-inch (1.8 meter) CMP, flap gated structure to serve as the interior drainage outfall
- Two, 195-foot (59 meter) gaps in the Boca Prieta dike
- Two salt water intrusion measures (canal plugs) within the diversion channel (post-original construction, but included as part of the 205 project)

Construction was completed under two contracts with the levee, diversion channel, interior drainage ditch and culvert structure completed in June of 2001 and the salt water intrusion measure (SWIM) completed on March of 2007. See **Figure 2** for project features completed in 2001.

Figure 2. Section 205 Project Features

2.3 Post-Construction

Since completion of the initial flood control project features in 2001, the lagoon system and its surrounding environment have been adversely affected by saltwater intrusion. Some of the primary effects include those to the Humacao Natural Preserve (HNR) Pterocarpus Forest, which is one of the largest remaining forested freshwater swamps in Puerto Rico. These swamps are dominated by bloodwood (*Pterocarpus officinalis*) trees. This tree species is protected and has a low saltwater tolerance, requiring mainly freshwater to survive. Since the completion of the project, field observations by DNER and USACE indicated that a vast number of bloodwood trees on the north shore of the Río Antón Ruiz were subject to environmental stress (e.g. wilting, loss of foliage, and dry bark and trunks), likely due to increased salinity levels. In addition, changes in the lagoon system biodiversity have been observed. For example, some species of plants, such as mangroves, that rely on both fresh and saltwater have increased spatially, and fish not previously documented in the area have appeared since the completion of the flood control project.

2.4 Temporary Saltwater Intrusion Measures (SWIM)

Salinity data from 1999 thru 2001 indicates that the salinity levels at Mandri Stations 2 and 3 were below 10 ppt prior to project completion. After completion of the project and connection of the diversion channel to the lagoon system, data gathered from 2004 thru 2007 indicated that the salinity levels had more than tripled within the Mandri Lagoons. The highest salinity level recorded was 35.2 ppt.

Based on field inspections conducted by USACE staff and the monitoring data provided by DNER, per a letter dated July 14, 2005, USACE agreed that ecosystem changes were evident in the vicinity of the Río Antón Ruiz Flood Control Project, likely due to construction of the diversion channel. In this letter, USACE suggested the investigation and implementation of temporary SWIM to lower the salinity levels and a study to assess the saline effects on the natural system. In order to preserve the Pterocarpus Forest and some of the biodiversity of both the freshwater and saltwater fauna and flora, a study was conducted and a series of temporary saltwater intrusion measures were developed and constructed to limit the amount of saltwater effects on the lagoon system and the Pterocarpus Forest. The temporary measures study was entitled “Rio Anton Ruiz Flood Control Project, Temporary Saltwater Intrusion Measures (SWIM), Humacao, Puerto Rico, November 11, 2011”. The salinity information gathered by DNER after construction of the SWIMs would be used to determine the effectiveness of the SWIMs, if additional studies would be required, and if the construction of permanent tidal exchange measures would be warranted.

After installation of the SWIMs in 2007, data indicates that salinity levels at all the monitoring stations decreased and met the initial target (below 10 ppt). Salinity levels measured at the stations ranged from approximately 0.1 to 7.0 ppt. These levels could be attributed to the SWIMs, as well as rainfall events.

However, at the end of 2008, salinity levels increased at most of the stations, and in several monitoring events they exceeded 10 ppt concentrations. It is assumed that the salinity levels are increasing because the SWIM plugs (sandbags placed at the bottom of the channel, up to mean low water level) have deteriorated mostly as a result of damage to the sandbags from small boats used by fishermen. The plugs are losing their effectiveness and allowing saltwater intrusion into the lagoon system. It should be noted that the SWIMs deterioration was expected to eventually occur. SWIMs were intended only as a temporary measure to lower the salinity levels during the data gathering to determine if the construction of permanent tidal exchange measures would be warranted.

2.5 Current (Proposed) Section 1135 Project

The current project is authorized under Section 1135 Project Modifications for Improvement to the Environment of the Continuing Authorities Program. As discussed previously, this project area was part of a CAP Section 205 flood control project that included construction of a diversion channel. A subsequent study on saltwater intrusion measures was conducted, and temporary saltwater intrusion measures (sandbags) were placed. The temporary measures were monitored for salinity levels and deemed to have successfully lowered the salinity levels. This Section 1135 project involves the design and construction of permanent features to reduce the salinity levels within the diversion channel (and thus lagoon system and Pterocarpus Forest). The current project includes evaluating three alternatives:

Alternative 1 consists of concrete-capped sheet pile weirs located at the two existing temporary SWIM sites (see Figure 6). One location is within the Rio Anton Ruiz, just north of the confluence of the Rio Anton Ruiz and the diversion channel. The other location is within the diversion channel, approximately ½ mile from the mouth of the diversion channel.

Alterative 2 consists of the same concrete-capped sheet pile weir, but only one at a new site. The location selected was within the diversion channel prior to the confluence of the diversion channel and the Rio Anton Ruiz.

The non-structural plan considered involves sand placement within the diversion channel, near the mouth of the diversion channel. There was previously a sand bar at this location that continues to wash out and build back during various storm events. The alternative was considered as a non-structural plan that would provide a more consistent sand bar, as the non-Federal sponsor has indicated that the sand bar does not develop as quickly as it used to (prior to the Section 205 project).

Alternatives 1a and 1b were developed to also review the incremental costs and benefits of constructing only one of the weirs. Alternative 1a consists of constructing the same concrete-capped sheet pile as in Alternative 1, but at only within the Rio Anton Ruiz channel. Alternative 1b was also the same as Alternative 1, but constructed only within the diversion channel.

The existing three barrel drainage culvert is severely deteriorated. It was originally considered as part of the project, but screened out early on. The replacement of the culvert would not provide any benefits for the purposes of environmental restoration (under 1135) as its purpose is to serve as an outfall for the interior drainage into the diversion channel. The culvert is located downstream of the lagoon and forest and thus would not provide any "flushing out" or other benefits for the 1135 project, but would nearly double the costs of the project.

3.0 Hydrology and Hydraulics

3.1 Project History

The coastal areas of Punta Santiago historically experienced frequent flooding, possible at any time during the year. Punta Santiago is a community within the project area directly on the coast. Flood damages were occurring when runoff from the mountains exceeded the detention capacity of the Mandri, Palmas, and Santa Teresa lagoons and flooded the low coastal areas in and around Punta Santiago. The authorizing document that details the hydrologic and hydraulic analysis of the study area is the "Rio Anton Ruiz at Humacao, Puerto Rico, Final Detailed Project Report and Environmental Assessment, Section 205, Flood Control", dated October 1993. See **Figures 3, 4, & 5** for location and drainage areas as outlined in the October 1993 Detailed Project Report.

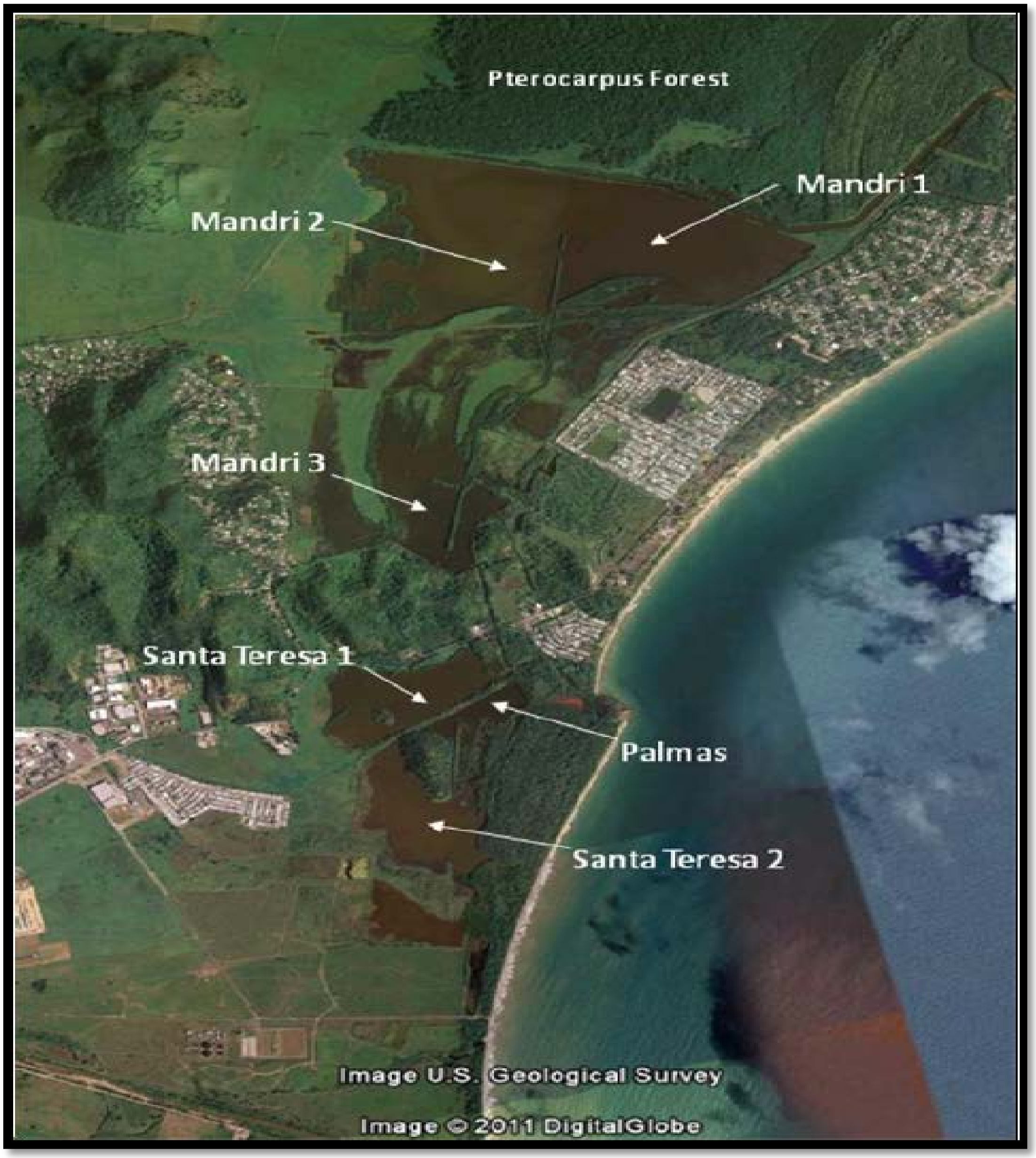


Figure 3. Humacao Natural Reserve lagoon system

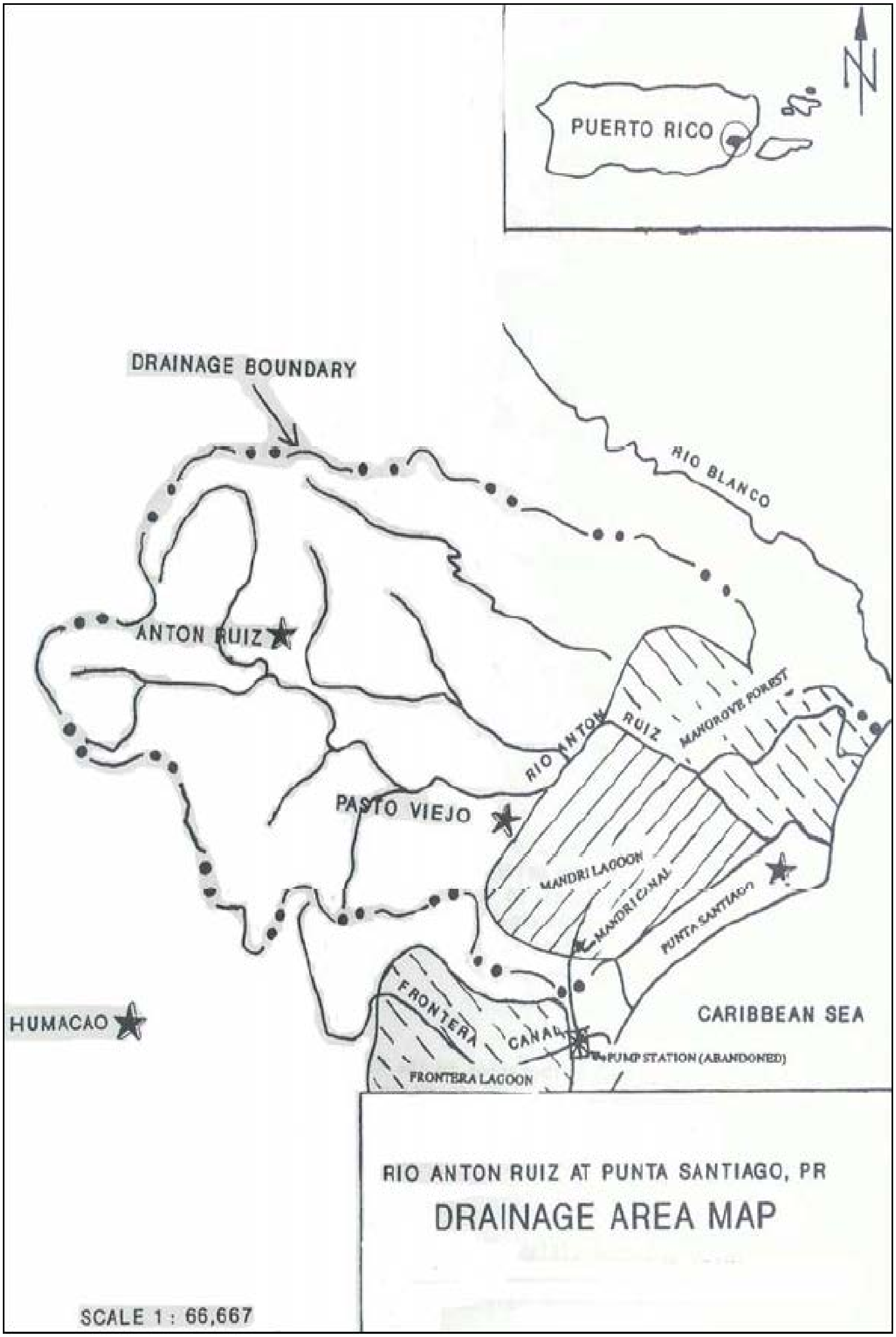


Figure 4. Drainage Area Map (from 1993 Detailed Project Report)

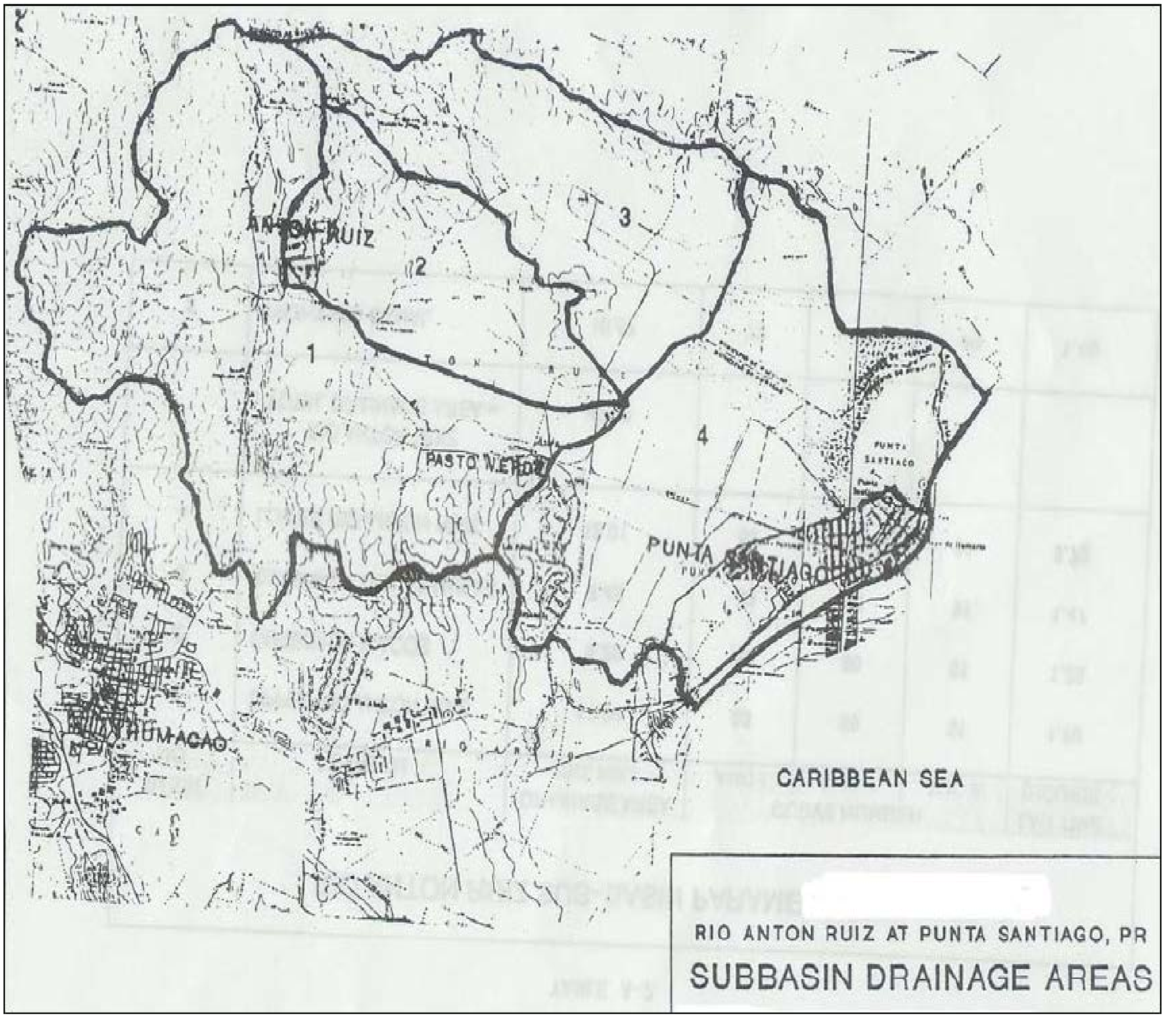


Figure 5. Subbasin Drainage Areas (from 1993 Detailed Project Report)

Hydrologic analyses detailed within the aforementioned report remains the most current related to the study area. Shortly after completion of contract 1 of the authorized flood control project, it was noted by Puerto Rico's Department of Natural and Environmental Resources (DNER) that the lagoon system and its surrounding environment were adversely affected by salt water intrusion. This led to a post construction change with the installation of two new temporary SWIM plugs in March 2007 (contract 1A). One located within the diversion channel near the Mandri lagoon and the other across the Rio Anton Ruiz immediately upstream of its confluence with the diversion channel. See **Figures 6, 7, & 8** for location of SWIM plugs and post installation photographs.

Section 6 of the final "SWIM Monitoring Report", November 2011, indicated the elevation of the SWIM plugs were set to the Mean Low Water (MLW) tide level based on the closest NOAA tide gage, which is located near the Roosevelt Roads Naval Air Station in the Municipality of Ceiba, approximately six (6) kilometers (3.73 miles) north of the project site. The purpose for establishing the plug elevations at MLW was to ensure an exchange of salt and fresh waters between the Caribbean Sea and the Lagoon system was still possible (i.e. not to completely eliminate saltwater intrusion, only reduce salinity concentrations); and additionally, to allow small boat traffic during the day-to-day monitoring of the lagoon system by DNER.

Section 6 (Project Performance) paragraph "g" within the Operation, Maintenance, Repair, Replacement, and Rehabilitation Manual (OMRR&R) states the following in relation to the SWIM's:

"The constructed SWIM feature is a temporary measure that consists of 2 plugs, one across the diversion channel near the lagoon and the other across the Rio Anton Ruiz above its confluence with the diversion channel. The two measures are approximately 150 feet to 200 feet long and consist of sand bags. The total amount of sand bags and the ranges of sizes are about 100 sand bags that weigh about 12 tons each, 40 heavy lift bags that weigh about 5,000 lbs each, and 8,000 sand bags that weigh about 70 lbs each. The top elevation of those bags was set to allow surface water to flow over the bags and limit the amount of salt water tide flowing into the system. DNER will be monitoring the lagoon and Rio Anton Ruiz River for at least 5 years from the construction date of the SWIM plugs in order to determine the design requirements for the permanent SWIM measure."

Figure 6. Location of SWIM Plugs (as per survey 16-027)

Figure 7. SWIM installed across the diversion channel (March 2007)

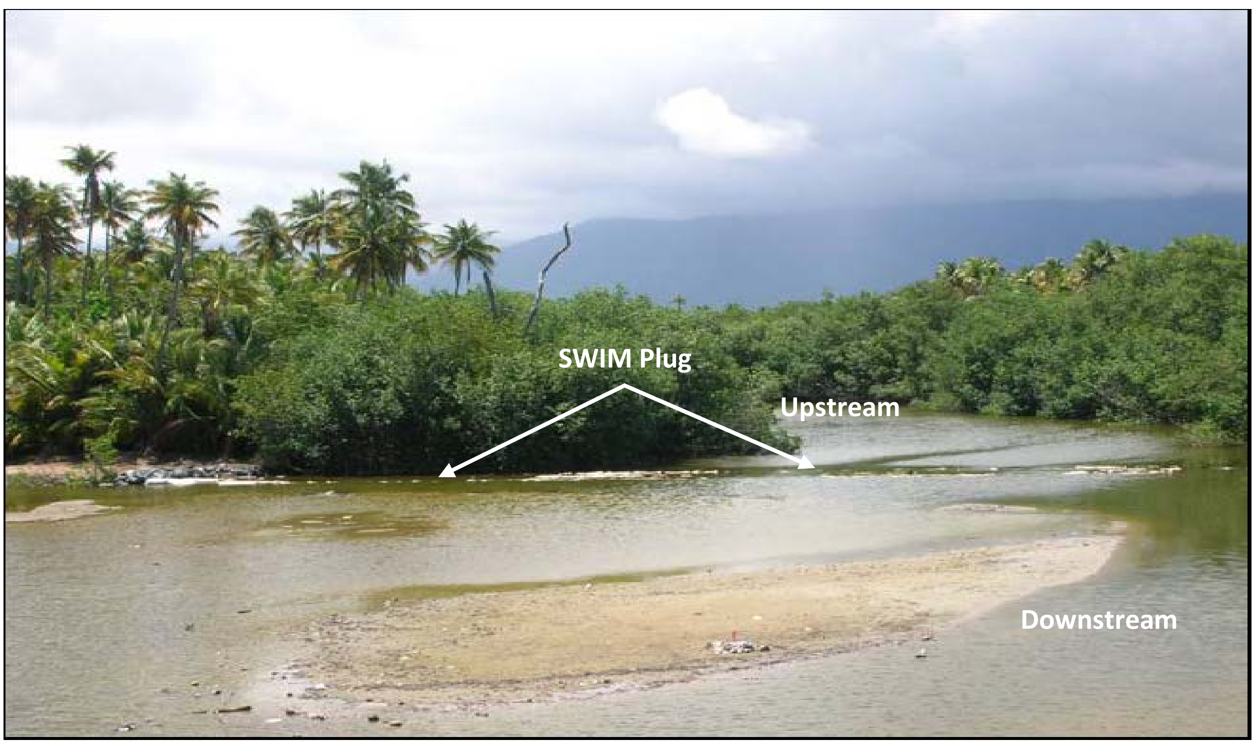


Figure 8. SWIM installed across the Rio Anton Ruiz (March 2007)

3.2 Current (Proposed) Project Modifications

USACE is authorized to assist in the restoration of degraded ecosystems through the modification of USACE structures, operations, or implementation of measures in affected areas as outlined in Section 1135 of the Continuing Authorities Program (CAP). This project seeks to provide a permanent solution to the previously installed temporary SWIM plugs. It should be noted that after installation of the temporary SWIM plugs, salinity data retrieved from DNER monitoring stations indicated a decrease in salinity that met initial targets successfully (below 10 parts per thousand (ppt)). Based on this data, the decrease in salinity can be attributed to the temporary SWIM plugs.

Replacement of the SWIM plugs will have no adverse influence on the hydrologic condition of the study area; therefore, no update to the original Section 205 has been undertaken. No major changes in land use have occurred in the basin. The hydraulic analysis performed resulted in a weir design that ensures that the permanent replacements for the temporary SWIM's will match, at minimum, the effectiveness in reducing salinity values upstream while not adversely impacting flood discharges to tide (i.e. no impact on flood damage reduction provided by original project). **Figure 9** indicates the location of the existing SWIM plug within the diversion channel as per hydrographic survey 16-027 (February 2016). **Figure 10** indicates the location of the existing SWIM plug within the Rio Anton Ruiz as per survey 16-027. **Figure 11** is a cross section of the diversion channel and SWIM plug converted from meters into feet with stationing from left bank to right bank looking downstream. **Figure 12** is a cross section of the Rio Anton Ruiz and SWIM plug converted from meters into feet with stationing from left bank to right bank looking downstream.

An analysis was performed of the vertical datum relationship between the Puerto Rico Vertical Datum of 2002 (PRVD02) and tidal datums relative to this project. Purpose for requesting this analysis was to ensure consistency with respect to project elevation reporting with that of survey 16-027. Elevations of tidal datums referenced to PRVD02 in feet are as follows:

Mean Higher High Water	MHHW = +0.807 ft, PRVD02
Mean High Water	MHW = +0.545 ft, PRVD02
Mean Sea Level	MSL = 0.000 ft, PRVD02
Mean Tide Level	MTL = -0.007 ft, PRVD02
Mean Low Water	MLW = -0.561 ft, PRVD02
Mean Lower Low Water	MLLW = -0.768 ft, PRVD02

The above tidal elevations are taken as the tailwater (downstream) elevations for both the diversion channel and Rio Anton Ruiz SWIM plugs. That is, discharge possible across the SWIM plugs is a function of head above the SWIM plug (weir) crest and degree of submergence of the SWIM plug caused by the tailwater elevation.

Figure 9. Survey of SWIM within diversion channel (survey 16-027, March 2016)

Figure 10. Survey of SWIM within Rio Anton Ruiz (survey 16-027, March 2016)

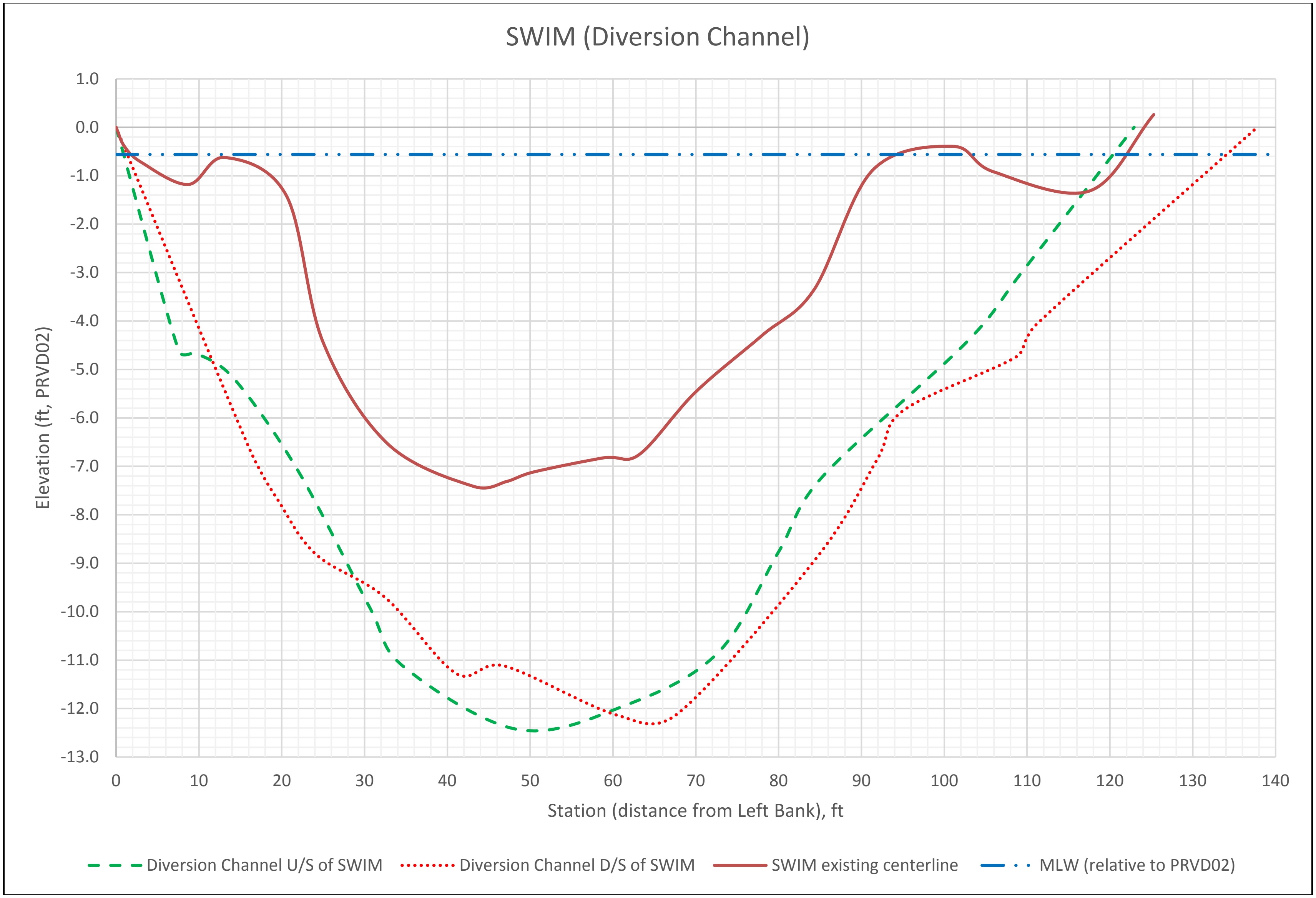


Figure 11. Cross section of diversion channel SWIM plug (survey 16-027, March 2016)

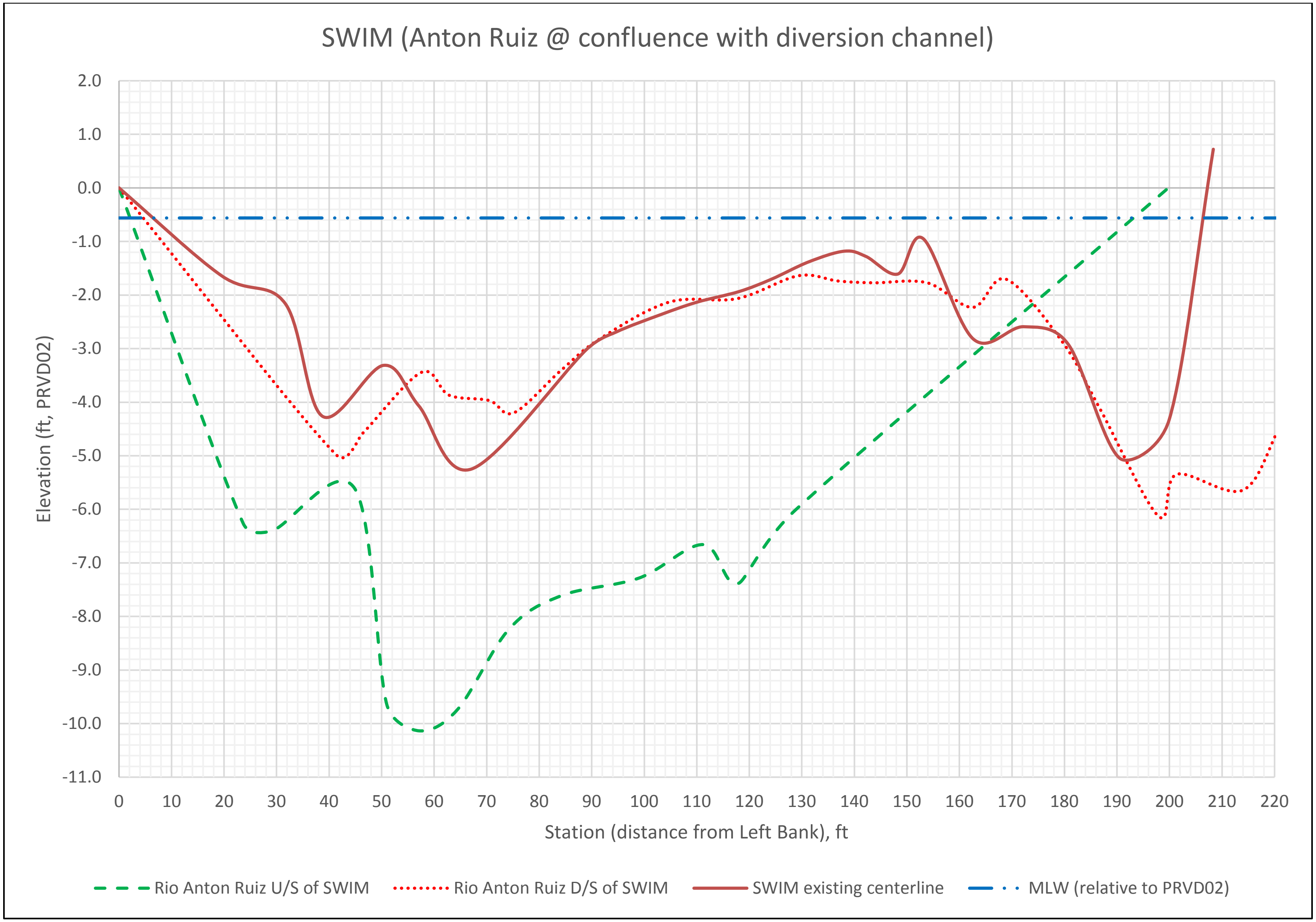


Figure 12. Cross section of Rio Anton Ruiz SWIM plug (survey 16-027, March 2016)

3.3 Hydraulic Analysis

The temporary SWIM plugs act as broad crested weirs where the as-built breadth of the weir was approximately 15 ft. Discharge over the weirs can be approximated using the following equation:

$$Q = CLH_e^{3/2} \qquad \text{(Equation 1)}$$

Where:

Q = Volumetric discharge (cfs)
C = Coefficient of discharge (variable*)
L = Weir Length (ft)
H_e = Energy head above weir crest (ft)

*The coefficient of discharge varies depending upon many factors (e.g. breadth of weir, head above weir crest, submergence of weir crest, etc.). Typical values of "C" for a broad crested weir of breadth 15 ft range from 2.63 – 2.70 (Brater & King, Handbook of Hydraulics, 6th edition) assuming a "free, uncontrolled" hydraulic flow regime, i.e. headwater is not influenced by tailwater. For instances where the weir crest is submerged, i.e. headwater is influenced by tailwater, the hydraulic flow regime transitions to "submerged, uncontrolled", and the discharge coefficient "C" is reduced based upon a submergence ratio (d/D or in other words, TW above weir crest / HW above weir crest) as developed by the U.S. Deep Waterways submerged-weir model (USGS, Water-Supply and Irrigation Paper No. 200). See **Table 1** for the coefficient reduction based on submergence ratio. Coefficient of discharge within equation 1 is modified to "C_s" when performing calculations for the submerged hydraulic condition.

Table 1. Relative Coefficients, U.S. Deep Waterways submerged-weir model

$\frac{d}{D}$	$\frac{C'}{C}$	$\frac{d}{D}$	$\frac{C'}{C}$
0.0	1.000	0.5	0.937
.1	.991	.6	.907
.2	.983	.7	.856
.3	.972	.8	.778
.4	.956	.9	.621
		1.0	

Alternative 1 Plan (see Attachment A, Alternative 1 Site Layout) is to install permanent SWIM consisting of two sheet pile, concrete capped weirs at the same locations as the two originally placed temporary SWIM plugs. Top of weirs are intended to be 0.25 ft above Mean Low Water (MLW) elevation with a 15 foot wide by 3 foot deep "notch" within the center of the diversion channel and Rio Anton Ruiz respectively. Top

elevation of the notch section will be 2.75 ft below MLW to allow navigation of the diversion channel and river at low water elevations while mitigating salt water intrusion into the Mandri lagoon system and further upstream of the Rio Anton Ruiz . DNER and other agencies need access to the monitoring stations. Thus, the notches are a design feature to ensure that small boat traffic can traverse the weirs at low water. It is recommended that buoys or some other form of channel marker or navigational aids be included with the project to direct boat traffic toward the notched opening.

The permanent SWIM plugs will also act as broad crested weirs where the breadth of the concrete cap will be 1.5 ft. Discharge over the permanent weirs can also be approximated using Equation 1. The coefficient of discharge for the permanent SWIM plugs with a weir breadth of 1.5 ft range from 2.62 – 3.32 (Brater & King, Handbook of Hydraulics, 6th edition) assuming a “free, uncontrolled” hydraulic flow regime, i.e. headwater is not influenced by tailwater. For instances where the weir crest is submerged, i.e. headwater is influenced by tailwater, the hydraulic flow regime transitions to “submerged, uncontrolled”, and the discharge coefficient “C” is reduced based upon a submergence ratio (d/D or in other words, TW above weir crest / HW above weir crest) as developed by the U.S. Deep Waterways submerged-weir model (USGS, Water-Supply and Irrigation Paper No. 200) as shown in **Table 1**. Coefficient of discharge within equation 1 is modified to “C_s” when performing calculations for the submerged hydraulic condition.

The temporary SWIM plugs were installed with a crest elevation equal to the MLW tidal elevation; therefore, it can be assumed that “free, uncontrolled” discharge occurs when the tailwater of the weir is at or below this elevation and the headwater is above. As the tailwater rises above the MLW elevation, the SWIM plugs become submerged and therefore discharge over the weir transitions to “submerged, uncontrolled” flow. See **Figures 13** and **14** for “free, uncontrolled” discharge ratings with respect to both the temporary and permanent SWIM plugs within the diversion channel and Rio Anton Ruiz respectively. Additionally, see **Figures 15** and **16** for “submerged, uncontrolled” discharge ratings with respect to both the temporary and permanent SWIM plugs within the diversion channel and Rio Anton Ruiz respectively. Note, that for the “submerged, uncontrolled” condition, the tailwater at each weir location was assumed to be equal to the Mean High Water (MHW) tidal elevation that causes the weirs to be fully submerged with headwater and discharge influenced by the tailwater elevation and degree of submergence of the weir.

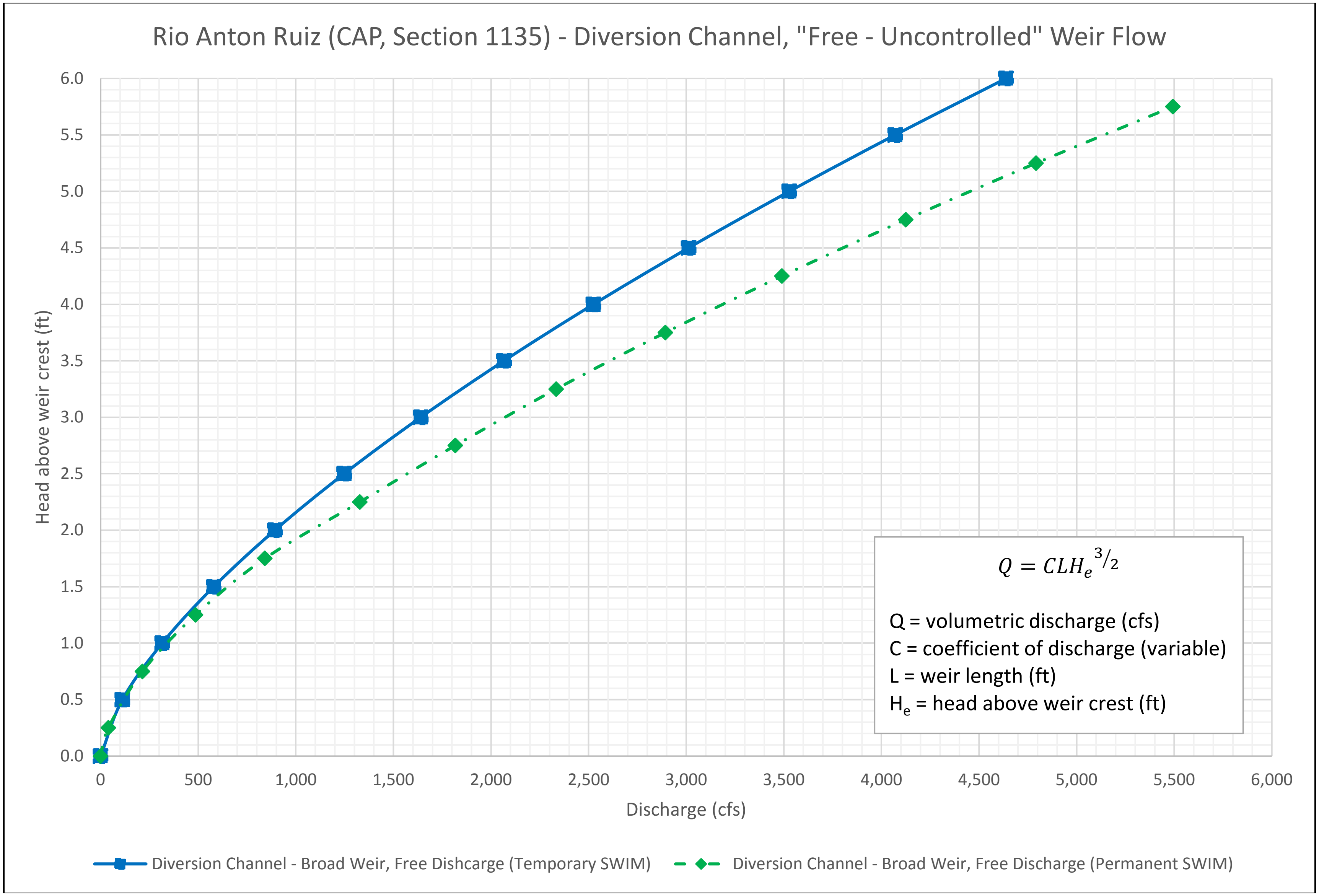


Figure 13. Diversion Channel, “Free – Uncontrolled” discharge rating

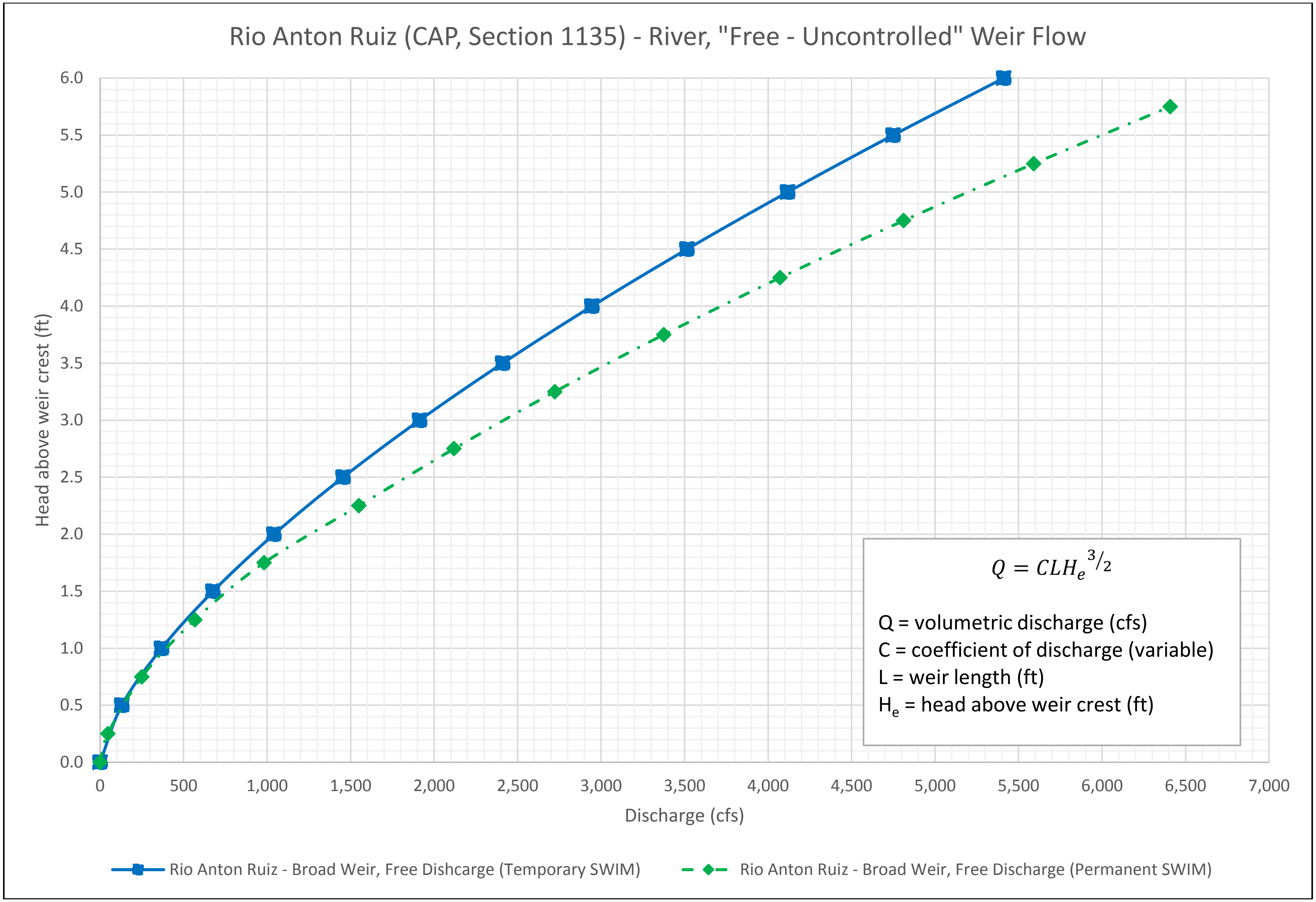


Figure 14. Rio Anton Ruiz, "Free – Uncontrolled" discharge rating

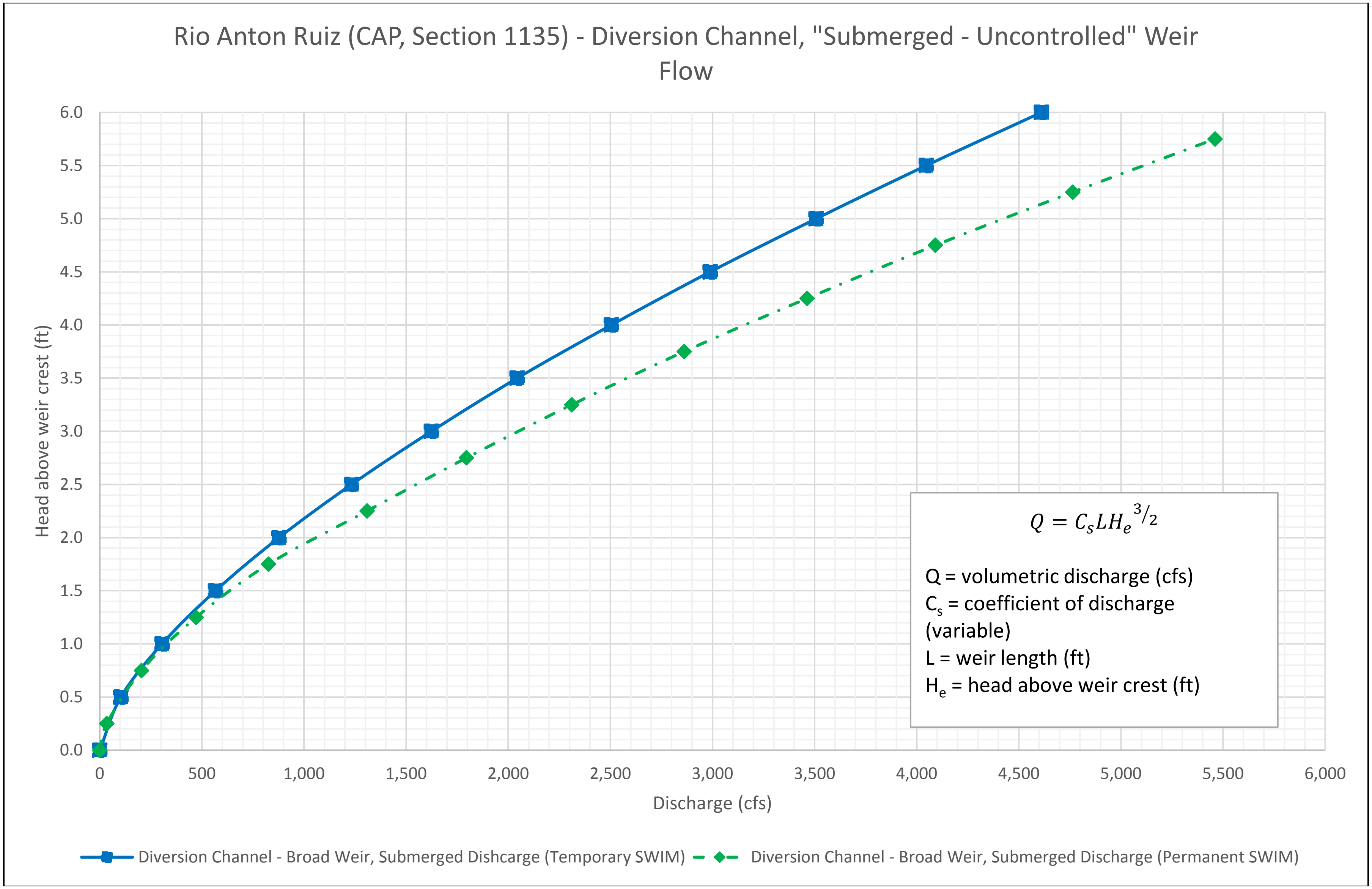


Figure 15. Diversion Channel, "Submerged – Uncontrolled" discharge rating

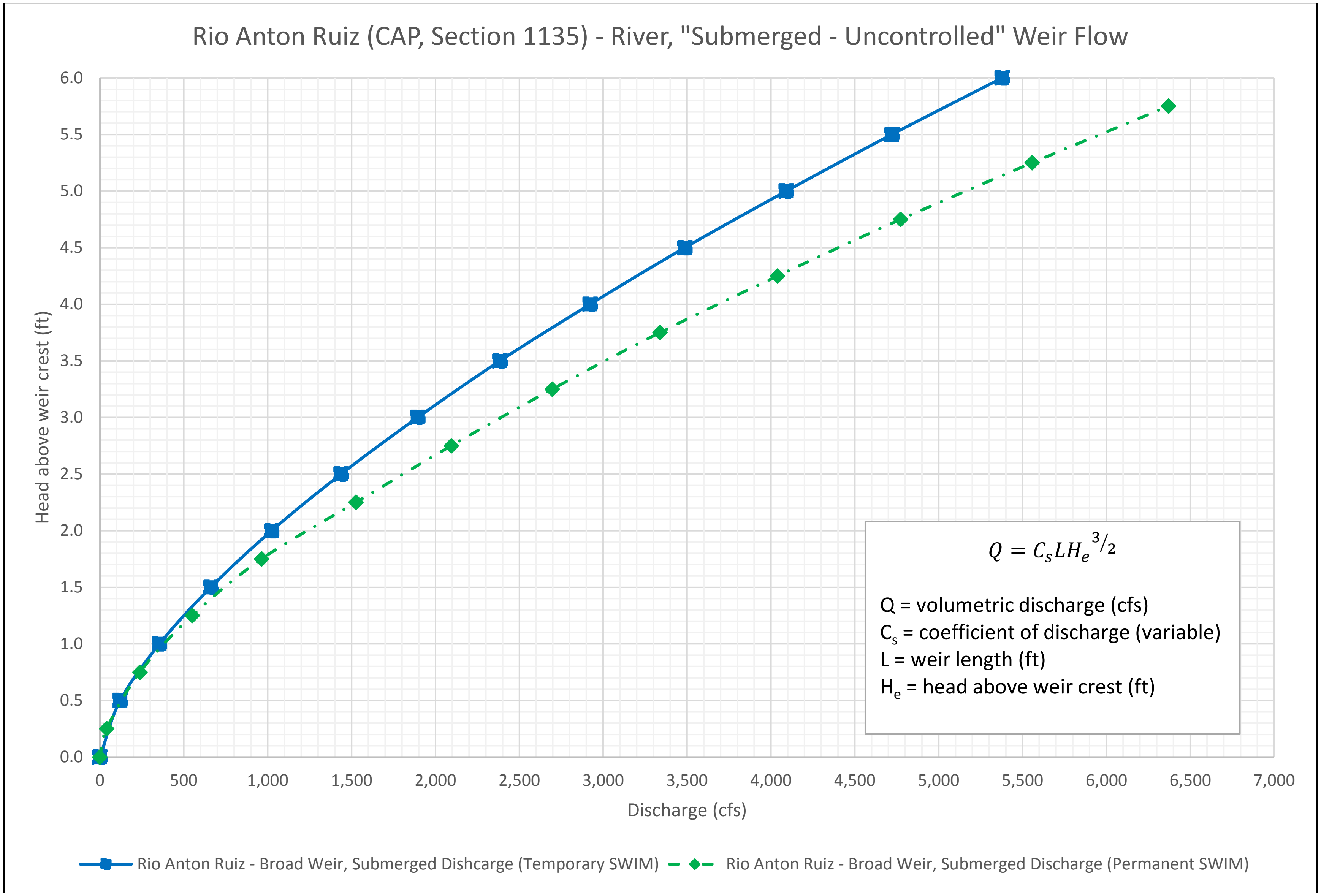


Figure 16. Rio Anton Ruiz, “Submerged – Uncontrolled” discharge rating

Figures 13 through **16** clearly indicate that the permanent SWIM weirs match discharge performance of the temporary SWIM weirs with head above the weir crest between 0.1 and 1.0 feet. As head above the SWIM weirs exceed 1.0 feet, the permanent SWIM weirs outperform the temporary SWIM weirs; thus showing no degradation of system functionality by implementation of the permanent SWIM weirs.

Alternative 2 Plan (see Attachment A, Alternative 2 Site Layout) is to install a single permanent SWIM consisting of a sheet pile, concrete capped weir downstream of the confluence of the Rio Anton Ruiz and diversion channels. Top of weir is intended to be 0.25 ft above Mean Low Water (MLW) elevation with a 15 foot wide by 3 foot deep "notch" within the center of the channel. Top elevation of the notch section will be 2.75 ft below MLW to allow navigation of the diversion channel and river at low water elevations while mitigating salt water intrusion into the Mandri lagoon system and further upstream of the Rio Anton Ruiz. An identical hydraulic analysis as was performed for two weirs would also apply to Alternative 2 Plan as the design and elevation for the sheet pile weir would be the same. However, the one weir located prior to the confluence of both channels would reduce the flood reduction benefits from the original Section 205 project by impeding flows out from the drainage culvert, as well as being located within a cultural resource area.

The non-structural plan is to place sand at the existing sand bar to increase natural formation of the sand bar, which has been noted not to form as quickly. This would require consistent maintenance efforts and higher maintenance costs. The sandbar should also be able to naturally wash out during high water or storm events and is preferred by the local sponsor to remain natural forming, making this plan not a permanent feature.

3.3.1 Permanent SWIM Weirs with “Notch”

The permanent SWIM weirs will each include a 15 foot wide by 3 foot deep "notch" within the center of the diversion and Rio Anton Ruiz channels. When the water surface elevation both upstream and downstream of the permanent weirs is at or below elevation -0.31 ft, PRVD02, discharge through the weir is possible via the “notch” section. Discharge through the “notch” section can be approximated using **Equation 2**.

$$Q = C_s L' H_e^{3/2} \qquad \text{(Equation 2)}$$

Where:

Q = Volumetric discharge (cfs)

Cs = Coefficient of discharge (variable)*

L' = Effective Weir Length (ft)

H_e = Energy head above weir crest (ft)

$$L' = L(0.1NH_e)$$

Where:

L = Total weir Length (ft)

N = Number of contractions (#)

*C_s is the variable coefficient of discharge resulting from the degree of submergence of the weir crest that has been discussed previously within this document.

Figure 17 is a discharge rating for flows possible through the "notch" section when the headwater elevation is exactly 3.0 ft above the weir "notch" crest (i.e. headwater elevation = -0.31 ft, PRVD02), and tailwater varies within the 3.0 ft "notch" opening range. It should be noted that while Figure 16 displays discharge possible through the weir "notch" under a 3.0 ft range of tailwater fluctuation, it is anticipated that the tailwater elevation will rarely fall below the MLW elevation. The following are "depths of submergence" of the weir "notch" crest at various tailwater elevations:

MTL (mean tide level): 2.99 ft above weir "notch" crest

MLW (mean low water): 2.75 ft above weir "notch" crest

MLLW (mean lower low water): 2.54 ft above weir "notch" crest

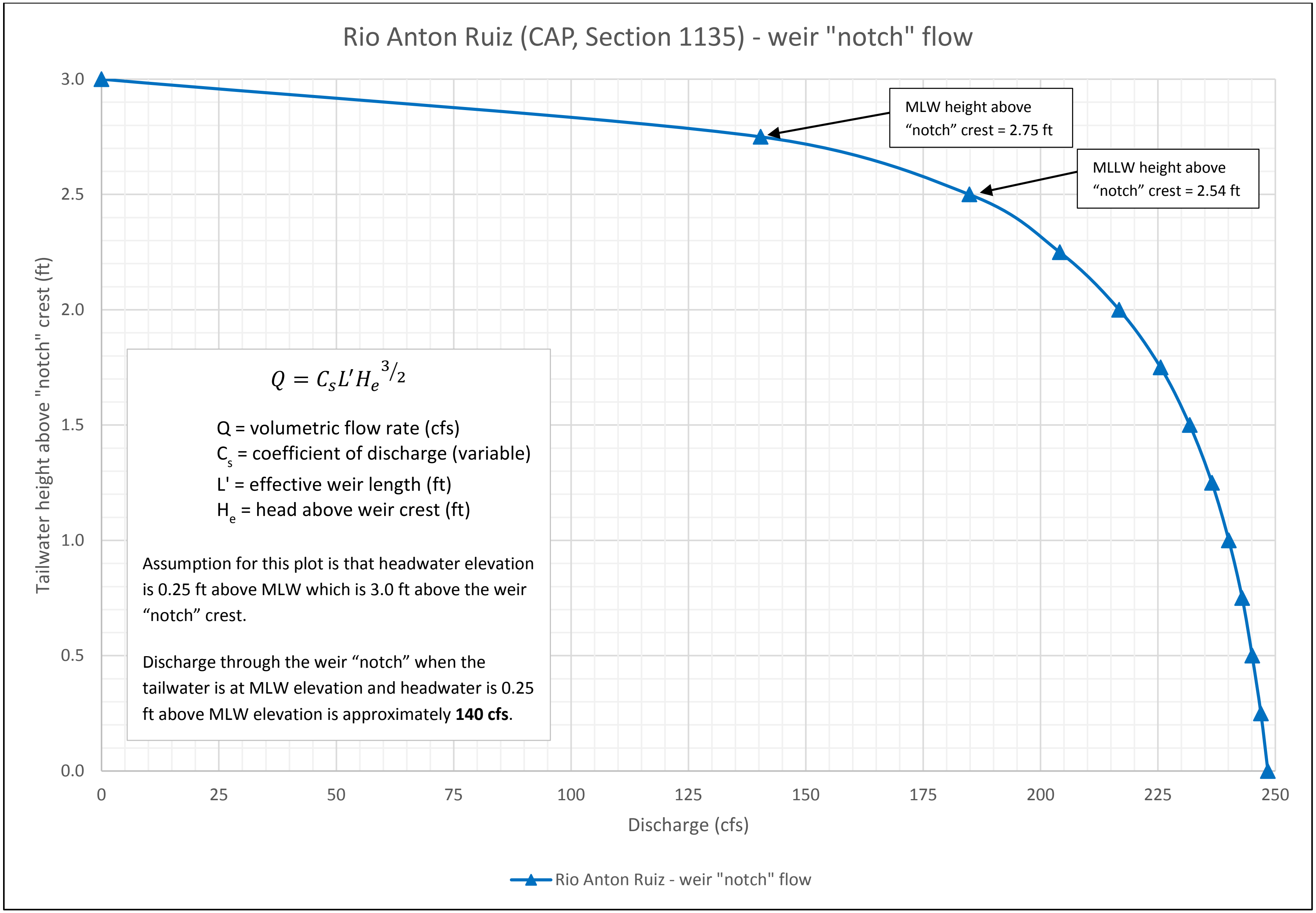


Figure 17. Permanent SWIM weir "notch" flow

Table 2 contains discharge flow rates possible through the weir "notch" section when the headwater elevation is -0.31 ft, PRVD02 and tailwater elevations vary within the 3.0 ft "notch" opening range. Where applicable the corresponding tidal elevation designation is annotated.

Table 2. Permanent SWIM weir "notch" flow

HW (ft, PRVD02)	TW (ft, PRVD02)	tidal designation	Q (cfs)
-0.31	-0.31		0
-0.31	-0.56	MLW	140
-0.31	-0.81	MLLW	184
-0.31	-1.31		217
-0.31	-2.31		240
-0.31	-3.31		248

The weir "notch" will allow for navigation of the diversion channel and river to DNER salinity monitoring stations within the Humacao Natural Reserve (HNR) (see **Figure 18**) at low water elevations while also mitigating saltwater intrusion further upstream of the weirs. It is anticipated that the permanent SWIM weirs will function to meet target salinity levels (below 10 parts per thousand (ppt)) within the HNR that was the purpose for installing the temporary SWIM plugs.

Figures 19 and 20 are the final comparison plots of the pre-project (temporary SWIM weirs) and post-project (permanent SWIM weirs with "notch") features where "total" weir flow i.e., the entire weir including "notch" flow for the permanent SWIM is calculated. These plots serve to confirm that the permanent SWIM weirs outperform the temporary SWIM weirs with respect to potential discharge; thus displaying no degradation of system functionality by implementation of the permanent SWIM weirs. It should be noted that tailwater elevation was assumed to be 0.545 ft, PRVD02 (MHW) for these computations therefore discharge over the weir is "submerged, uncontrolled" with the coefficient of discharge "Cs" varying based upon the degree of submergence.

Reverse flow conditions will occur when water surface elevations within the lagoon system are lower than those on the tidal (ocean) side of the weir. Flow through the notch section can be approximated using equation 2 and should be expected to be identical to those that would occur if head differential were reversed. Thus, tide (ocean) is considered headwater and lagoon is considered tailwater. This condition is relevant not only to the notch section, but the entire weir under both existing and proposed replacement conditions.

To reiterate, it is recommended that buoy's or some other form of channel marker or navigational aids be included with the project to direct boat traffic toward the weirs notched opening.

Figure 18. DNER salinity monitoring stations (approximate locations)

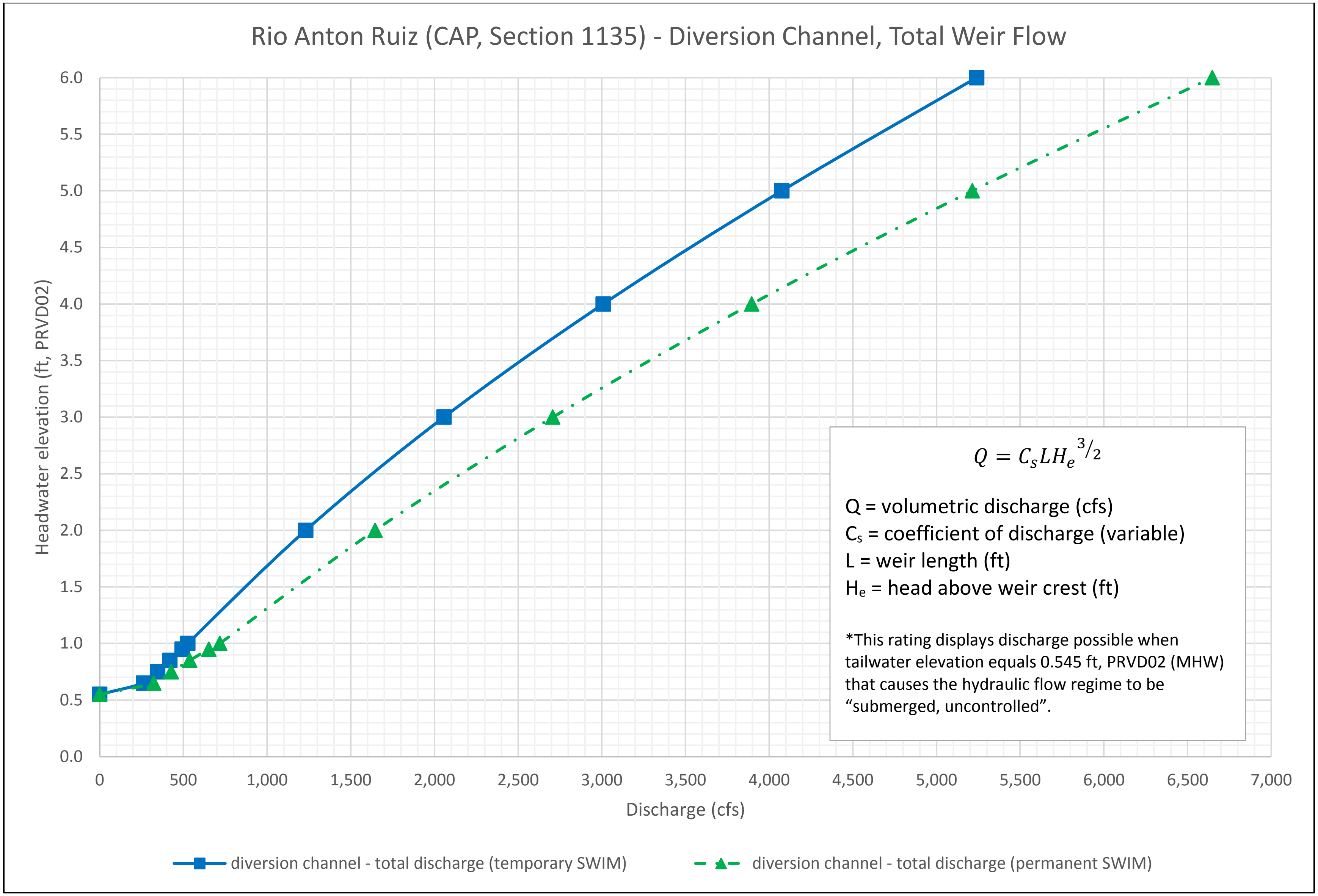


Figure 19. Diversion Channel, "Total" weir flow discharge rating, "submerged, uncontrolled" regime

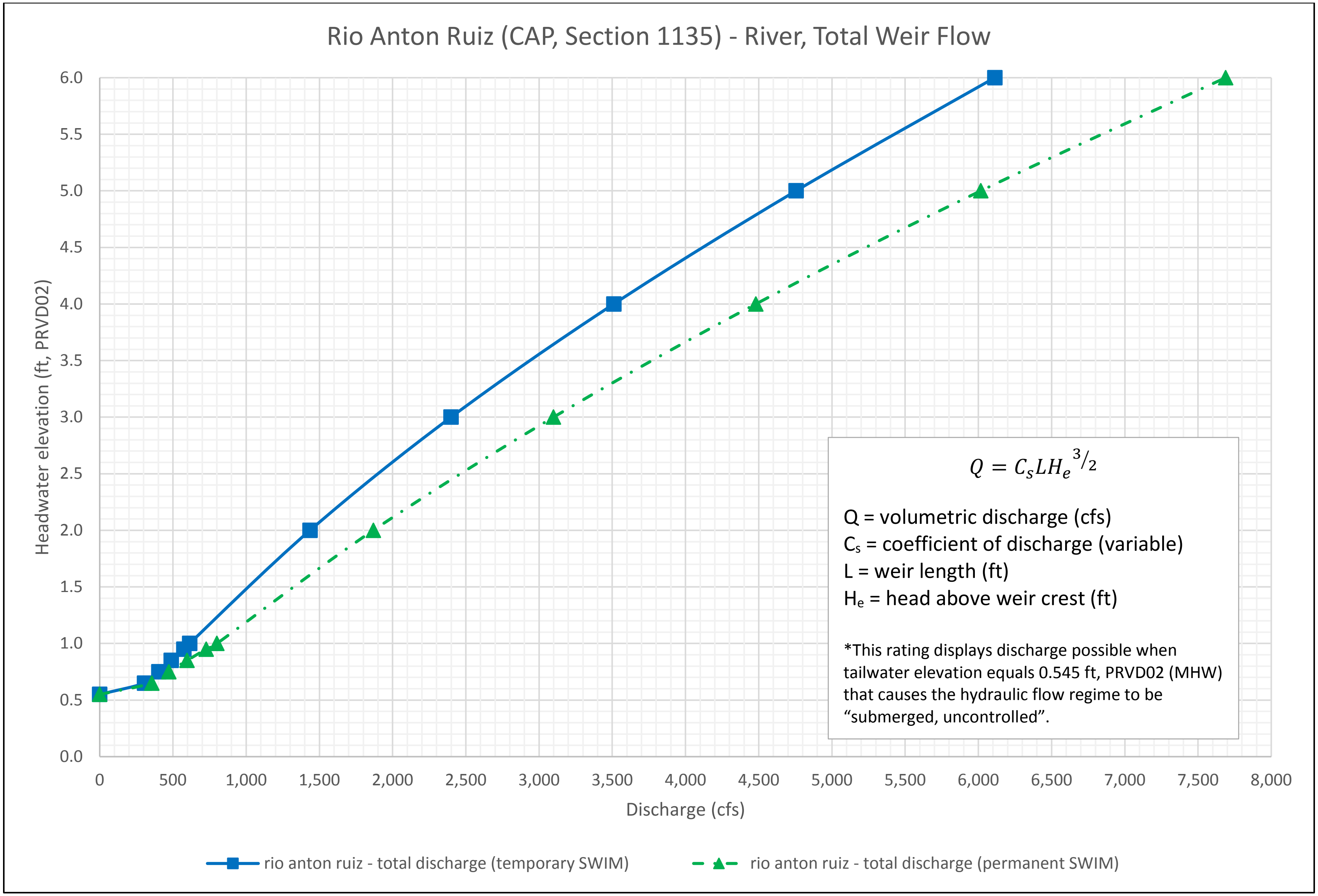


Figure 20. Rio Anton Ruiz, "Total" weir flow discharge rating, "submerged, uncontrolled" regime

3.4 Sea Level Rise Adaptability

Over the next 100 years, it is possible that rising sea levels associated with climate change could have a dramatic impact on the project area. The magnitude of those impacts will depend on which of three projected trends adopted by the USACE occurs. **Figure 21** displays the low, intermediate, and high sea level range projections (graphic and tabular) relative to NOAA station 9755371 (San Juan, PR).

Estimated Relative Sea Level Change from 2016 To 2116
9755371, San Juan, PR
NOAA's Published Rate: 0.00541 feet/yr
All values are expressed in feet

Year	USACE Low	USACE Int	USACE High
2016	0.00	0.00	0.00
2020	0.02	0.04	0.10
2025	0.05	0.09	0.24
2030	0.08	0.15	0.40
2035	0.10	0.22	0.57
2040	0.13	0.28	0.77
2045	0.16	0.36	0.99
2050	0.18	0.43	1.22
2055	0.21	0.51	1.47
2060	0.24	0.60	1.74
2065	0.27	0.69	2.03
2070	0.29	0.78	2.33
2075	0.32	0.88	2.66
2080	0.35	0.98	3.00
2085	0.37	1.09	3.37
2090	0.40	1.20	3.75
2095	0.43	1.32	4.15
2100	0.46	1.44	4.57
2105	0.48	1.57	5.00
2110	0.51	1.70	5.46
2115	0.54	1.83	5.93
2116	0.54	1.86	6.03

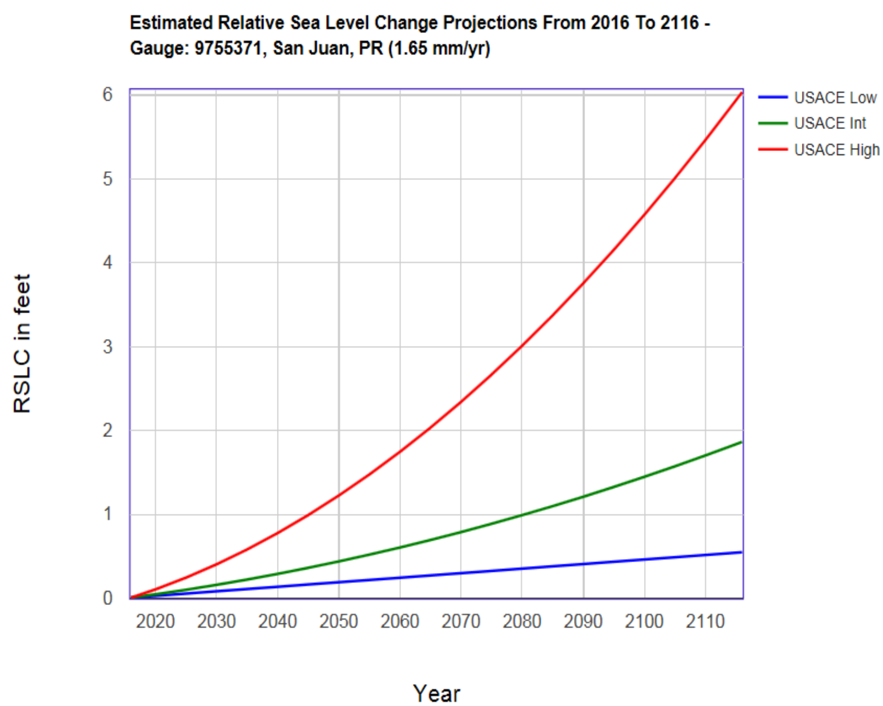


Figure 21. Relative Sea Level Change Projection (retrieved from http://corpsclimate.us)

Based on sea level projections, it is anticipated that within 50 years sea level rise will be approximately 0.3 feet for the low rate, 0.8 feet for the intermediate rate, and 2 feet for the high rate. Within 100 years, the sea level rise will be approximately 0.5 feet for the low and up to 6 feet for the high rate. The project is designed to be able to be adapted to sea level rise as needed. To mitigate for anticipated sea level rise, the permanent SWIM weirs shall be constructed such that additional height can be added uniformly across the entire length of the respective weir via additional concrete or weir boards bolted on. This feature shall ensure the project functions as designed both under existing and future sea level conditions. The adaptive management that would be required for any future sea level rise would be the sponsor's responsibility. The project can adapt to the low and intermediate rates of rise for the 50 year projection, and still serve as an effective saltwater intrusion measure. The high rates of rise are high enough that waters would begin to flank the channel banks and protected areas, reducing both the saltwater intrusion and flood reduction benefits.

3.5 Climate Change Analysis

The overarching USACE climate change policy document, *USACE Climate Preparedness and Resilience Policy Statement* (June 2014), requires consideration of climate change at every step in the project life cycle for all existing and planned USACE projects to reduce vulnerabilities and enhance the resilience of our water-resource infrastructure. Guidance for incorporating climate change and hydrologic analyses is provided in Engineering And Construction Bulletin (ECB) No. 2016-25 (16 Sept 2016), Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects. This applies to all current and future studies and any completed projects for which Federal funds are being used to rehabilitate a project, but does not apply to short-term water management decisions. The analysis provides for consideration of specific climate change projections in the project area and potential impacts to the particular hydrologic analysis.

The required qualitative analysis involves two phases. Current climate change trends are analyzed during Phase I, and projected future changes to hydrology is analyzed during Phase II. Phase I consists of literature review and investigation of annual maximum stream flow trends using the USACE Climate Hydrology Assessment and USACE Nonstationarity Detection Tools. Phase II consists of investigating projected future trends in annual maximum stream flows using the same two USACE tools mentioned previously, and performing a vulnerability assessment using the USACE Watershed Vulnerability Assessment Tool. The Climate Change assessment for this project are presented in the following sections.

3.5.1 Phase I: Relevant Current Climate and Climate Change.

Humacao, Puerto Rico.

Humacao, Puerto Rico has a tropical climate characterized by relatively high temperatures and approximately 75% humidity. The warmest month is August, with an average maximum temperature of 88°F; and the coolest month is December, with an average maximum temperature of 83°F. The rainy season spans from May through December. May is the wettest month with an average monthly precipitation of 6 inches, and February is the driest month with an average monthly precipitation of approximate 2 inches. The HUC for this watershed is 21010005.

According to USACE (2015a), which references the results of numerous climate studies in Puerto Rico and the Caribbean, reports an increasing trend in observed nightly and daily maximum air temperatures in the study region over the period of record between 1950 – 2004. The third NCA report (Carter et al., 2014) presents a study finding by the Puerto

Rico Climate Change Council (PRCCC) that the annual average temperature in Puerto Rico has experienced an increase of 1. 8°F between 1900 and 2010. Station analyses during the same period across Puerto Rico show an increase of annual average temperatures at a rate of 0.022-0.025 °F/yr. It was noted that some areas of the island have experienced a faster warming trend than others due to the urban heat island effect.

With respect to precipitation as reported (USACE 2015a), trend results vary between different reports, as well as across Puerto Rico. For example, the USACE study reported one analysis of station data showed no changes, while another indicated a 0.003 in/day/year decrease in rainfall between 1948 and 2007. Overall however, numerous literature syntheses reported in increased amount of rainfall during isolated extreme events, with an overall decrease in annual total precipitation (USACE 2015a). According to USACE 2015a, the precipitation trends in Puerto Rico differ both regionally and seasonally. The southern region of Puerto Rico has experienced an increase in precipitation, while the northern and western areas of have experienced a decrease. Additionally, summers appear to be trending dryer, while winters are trending wetter (USACE 2015a).

Observed Changes.

The USACE Climate Hydrology Assessment Tool was utilized to examine observed streamflow trends in the vicinity of the example project. However, the Climate Hydrology Assessment Tool did not contain stream gage information for HUCs in Puerto Rico at the time of the assessment.

The Nonstationarity Detection Tool was also utilized to examine the hydrologic time series at a gage in Rio Anton Ruiz. However, the Nonstationarity Detection Tool did not contain stream gage information in Puerto Rico at the time of the assessment. During the time of writing this report, SAJ was in the process of providing rainfall information for incorporation into the Nonstationarity Tool.

Projected Changes in Climate.

The NOAA National Environmental Satellite, Data and Information Service (NESDIS) released a report in January 2013 assessing climate trends and scenarios into the next 50–100 years for the Southeast CONUS region (NOAA 2013). The report indicates that over the period of hydroclimatological record for the Southeastern United States, both temperature and precipitation have shown either a statistically insignificant trend or no trend in change. The only trend noted was a slight increase in precipitation in the Gulf region. To account for climate change, the projected meteorological conditions in the region considers the past temperature and precipitation records, as well as the modeled future conditions in the area through 2099. According to the NESDIS report, a warming trend of approximately 2-5°F and no discernable precipitation trend can be expected over the next 50 years, although these estimates have significant uncertainty.

3.5.2 Phase II: Projected Changes to Watershed Hydrology and Assessment of Vulnerability to Climate Change.

The USACE Climate Hydrology Assessment Tool was used to examine observed and projected trends in watershed hydrology to support the qualitative assessment. However, the Climate Hydrology Assessment Tool did not contain stream gage information for HUCs in Puerto Rico at the time of the assessment.

The USACE Watershed Vulnerability Assessment (VA) Tool was used to examine the vulnerability of the project area to future flood risk. The VA Tool did not contain any watersheds in Puerto Rico at the time of the analysis. However, the tool did contain data on precipitation and temperature trends in the Southeastern United States, with some specific data for the island of Puerto Rico. The Regional Overview for the Southeast United States (which includes Puerto Rico) discusses threats to three key topics; increased sea level rise threats, increasing temperatures, and decreased water availability. For specific precipitation trends, this tool shows that Puerto Rico has experienced a 33% increase between 1958 and 2012 in precipitation amount during very heavy rain events (Figure 22). The tool also reports a modeled prediction of an over 30% increase in consecutive dry days in southeast Puerto Rico for the years 2070-2099 (as compared to the years between 1971-2000), if continued emissions increases (Figure 23). Regarding temperature trends, the VA Tool shows an average increase in the annual number of frost-free days between 10-14 days in Puerto Rico (Figure 24). The increased number of consecutive dry days combined with the higher temperatures and increased severity in large rainfall events has significant implications for native Puerto Rico flora and fauna, increased soil erosion, and human health.

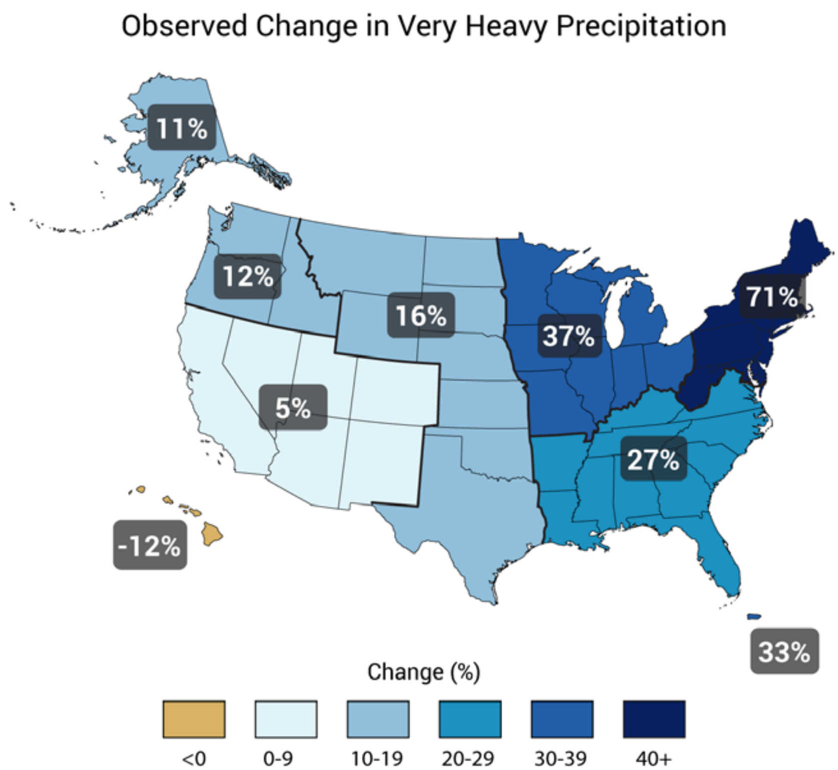


Figure 22. Observed Change in Very Heavy Precipitation

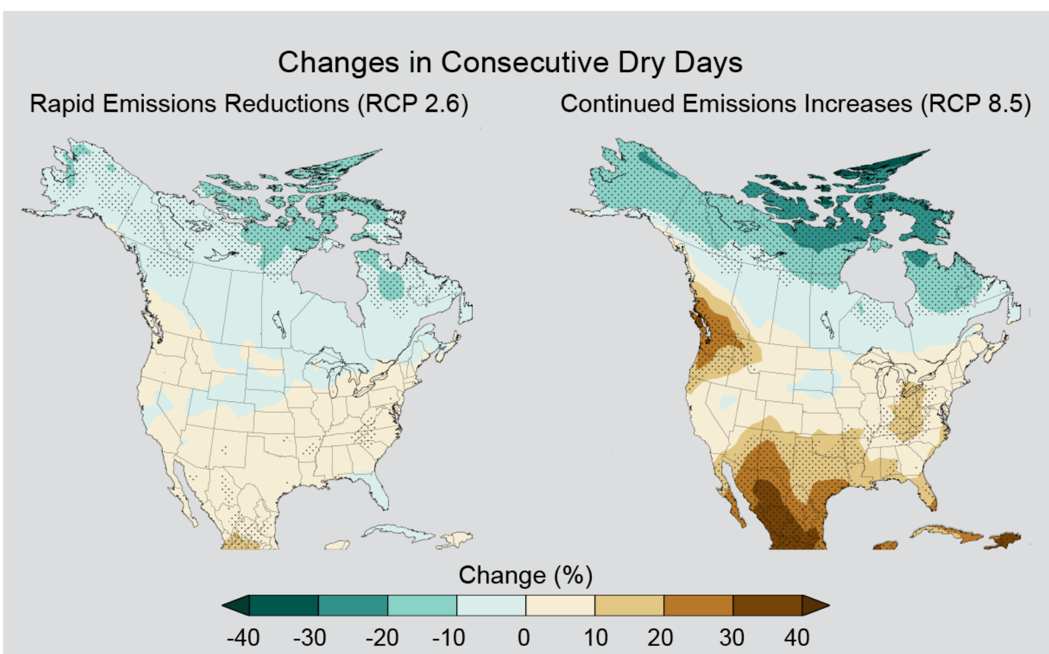


Figure 23. Observed Changes in Consecutive Dry Days

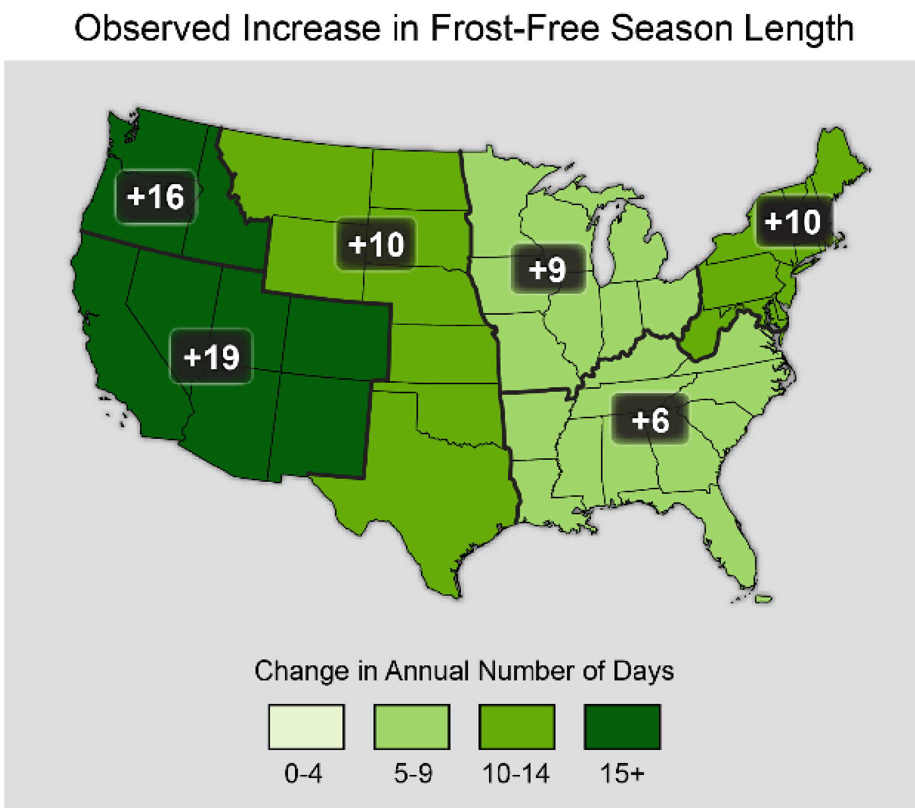


Figure 24. Observed Increase in Frost-Free Season Length

The actions that can be taken in the context of the current study to make the community more resilient to higher future flows, overall wetter conditions, and higher temperatures are similar to those to be taken in the event of sea level change.

3.6 Design Phase

During the design and implementation phase, it is recommended to acquire additional survey of the channel to include set interval hydrographic cross sections within the channel and topographic survey along the channel banks. This additional information will be used to verify all of the current design elevations with a hydraulic model.

4.0 Surveying and Mapping Requirements

Survey was collected for the original Section 205 project during design phase. Hydrographic survey was collected (February 2016) within the diversion channel and the Rio Anton Ruiz at just the temporary plug locations for current elevations of the SWIMs. The construction plans from the Section 205 (and survey for design and implementation phase of Section 205) along with the current hydrographic survey of the temporary plugs was used for analyses and design for this feasibility phase. Additional survey of the diversion channel and Rio Anton Ruiz will be collected during design and implementation phase to verify channel widths, depths, and elevations.

5.0 Geotechnical

This portion of the report addresses the geotechnical design and considerations with respect to the permanent salt water intrusion alternative of the Río Antón Ruiz Restoration project. Since completion of the Río Antón Ruiz authorized flood control project in 2001, the lagoon system and its surrounding environment have been affected by saltwater intrusion. Two temporary salt water intrusion measures (SWIM) plugs were installed at the end of March 2007. The plugs consisted of heavy (high-density polyethylene and UV resistant) lift bag barriers and sand bags placed on the channel and river beds in water depths of 5 to 6 feet at the diversion channel and up to 10 feet in the Río Antón Ruiz location. The plugs were armored with riprap on the upstream and downstream sides to resist damage during storm discharge. The temporary SWIM reduced salinity levels to the initial target rate (less than 10 ppm). Over time, however, the temporary SWIM features have deteriorated and salinity levels in the lagoon system have increased once again.

A screening of alternatives was conducted using plan formulation objectives and response criteria to determine the possible permanent measures. Three alternative plans were proposed as the permanent solution: 1) Sheetpile weir at two locations - the diversion channel and Rio Anton Ruiz; 2) Sheetpile weir at only one location - the mouth of the channel; and 3) Non-structural sand placement at the mouth of the diversion channel (no erosion protection is anticipated for this alternative as the sand would need to be able to wash out during a flood event to allow water flows out of the channel). Although the first two alternatives were intrinsically the same when comparing the type of structure and design, the location of Alternative 2 included a cultural resources area. In addition, the designated location in Alternative 2 would impact the flood risk reduction objectives of the initial flood control project. On the other hand, Non-structural plan would result in high maintenance costs, with no confirmation that the measure would reduce the salinity level. Alternative 1 would be installed at the locations where temporary SWIM had been placed. Based on the salinity measurements after the previous SWIM were placed, it was confirmed that these locations were adequate to minimize the saltwater intrusion into the lagoon. For the reasons described above, Alternatives 2 and 3 were not considered for the project. Therefore, Alternative 1 was selected as the proposed permanent measure. Alternative 1 consists of a sheet pile wall with top elevation of -0.31 feet to a tip elevation -24.0 feet driven across the channel. The wall includes a 15 feet wide, 3 feet deep rectangular notch within the center of the channel to accommodate small boat traffic. Upstream and downstream sides of the wall include stone protection to resist damage during storm discharge.

Any elevations mentioned are referenced, in feet, the Puerto Rico Vertical Datum of 2002 (PRVD02), unless noted otherwise.

5.1 Geology

5.1.1 Regional Geology

The site is located within the Central Igneous Province (CIP) of Puerto Rico. The CIP is further divided by the Cerro Mula Fault Zone (CMFZ). Río Antón Ruiz is located within the CMFZ, and its geomorphic expression is highly influenced by the fault zone. To the south of the CMFZ, the area is characterized by plutonic rocks of the San Lorenzo Batholith, and surrounded by metamorphosed rocks, which are overlain by quaternary alluvium and beach deposits. To the north of the CMFZ, the east coast is dominated by a comfortable sequence of Early Cretaceous basaltic-andesitic lavas and volcanoclastic sedimentary rocks overlain by alluvium and beach deposits. Intrusive and extrusive volcanoclastic rocks range in age from Cretaceous to Eocene.

5.1.2 Local Geology

Locally, the area is comprised by beach and swamp deposits. The beach deposits are unconsolidated fine to coarse-grained sand and pebble deposits of Quaternary age. They are mostly composed of quartz, feldspars grains as well as plutonic and volcanic rock fragments, with some marine sand (i.e., composed of shell, algal, and coral fragments). Swamp deposits are also of Quaternary age, and are characterized as black to dark brown, organic-rich soils, and muck located in poorly drained parts of the alluvial plain. Large part of these deposits are covered by mangroves. Both of these deposits are gradational in nature, and partially overlain each other with other alluvial deposits.

5.2 Field Exploration and Laboratory Testing

5.2.1 Encountered Materials

A field exploration was not performed as part of this Section 1135 study. Instead, existing field data from previous design efforts, were utilized to evaluate site conditions. Three previously (1990) drilled Standard Penetration Test (SPT) borings conducted under the original Section 205 project are located within the study area as shown in **Figure 22**, and as summarized in Table 3. Unconsolidated material was sampled to a depth of 30 feet, continuously, every 3 feet. **Figure 22** shows the approximate location of the borings. Boring logs are included at the end of this document in Attachment B.

Table 3. Approximate location of SPT borings within the Study Area

SPT boring Designation	State Plane, PR State Plane, NAD1927*		Project Location
	X	Y	
CB-AR-10	737082	122451	Río Antón Ruiz, Humacao, P.R.
CB-AR-11	738476	123228	
CB-AR-12	739959	122743	
* Coordinates presented correspond to the project coordinate system and datum			

Materials encountered consisted of fill, sands and silts, with lesser amounts of clay. Fill material is characterized by gravelly silts to sands and silts with some rock fragments. Sands are characterized as poorly graded to silty sands, with some pebble-sized rock, and shell fragments. Silty material also contains shell fragments. Some clay is also found occurring with silt. While the visual classification of the soils show large deposits of clay material, laboratory testing indicate that this material is predominantly silt.

5.2.2 Laboratory Testing

Sieve analysis, consolidation tests, triaxial tests and Atterberg limits were performed on select samples. A summary of the testing results is shown in Table 4. Consolidation and triaxial tests results and detailed laboratory results are included at the end of this document in Attachment B.

Table 4. Summary of laboratory results for select samples

Boring Designation	Sample Designation	Sample Depth (ft)	Water Content (%)	USCS
CB-AR-10	2	1.5-3.0	26	SM
CB-AR-10	8	10.5-12.0	39	SP
CB-AR-10	11	15.0-16.5	34	ML
CB-AR-10	13	18.0-19.5	-	SM
CB-AR-10	18	25.5-27.0	29	ML
CB-AR-11	1	0.0-1.5	21	SM
CB-AR-11	4	4.0-6.5	-	SM
CB-AR-11	11	15.0-16.5	50	SM
CB-AR-11	-	25.0-27.0	32.1	SM*
CB-AR-12	4	4.0-6.5	-	SM
CB-AR-12	11	15.0-16.5	35	SP
USCS: Unified Soil Classification System *Atterberg limits tests performed, results were non-plastic				

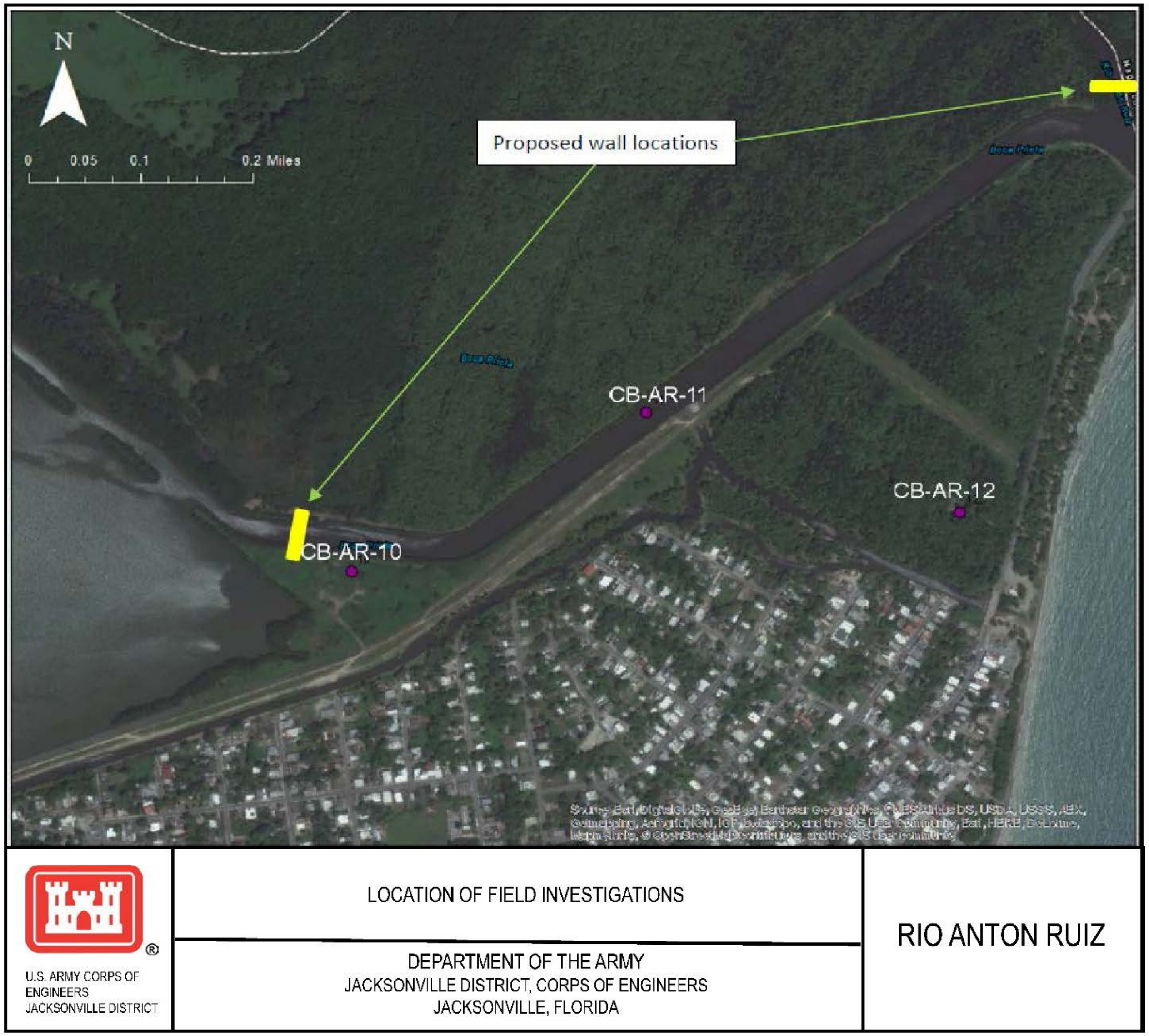


Figure 25. Boring locations

5.3 Geotechnical Evaluation

Geotechnical analyses for this project included stability, sheet pile, and seismic evaluation for the proposed wall. Global stability analyses were performed using the Spencer's method of slices and the circular search routine of the SLOPE/W computer program. The SLOPE/W program is part of the GeoStudio suite of software developed by Geo-Slope International Ltd. In addition to the analyses mentioned above, the CWALSHT software was used to evaluate the minimum tip elevations of the sheet pile wall system based on the stability requirements of the wall. The CWALSHT software was developed by the US Army Corps of Engineers Waterways Experiment Station, CASE program. Details of the geotechnical analyses performed are detailed below

5.3.1 Soil Parameters

Although data was available for the three borings within the project area, the estimated soil parameters for the proposed sheet pile wall were based only on boring CB-AR-11 as it was determined to represent the most critical or worst soil conditions and is the closest to the project area. However, it should be noted that no new subsurface investigations were performed for this project, thus, soil conditions could vary given that site specific data is collected in the future. The wall was evaluated using long-term soil strength parameters due to the presence of mostly granular material, hence, *S* soil strength parameters were used. The table below presents the simplified soil profile along the Boca Prieta diversion channel based on the information available which are the parameters used in the evaluation of the wall. The estimated soil properties are based on the SPT data, in particular, blow count per foot, limited laboratory test data, typical values of similar materials within the Humacao area, and engineering judgment.

Table 5. Rio Anton Ruiz Restoration Project Soil Parameters

Elevation (NAVD 88)		Soil Classification (USCS)	γ_{sat} (pcf)	γ' (pcf)	Undrained (Q)		Drained (S)	
From	To				ϕ (deg.)	c (psf)	ϕ' (deg.)	c' (psf)
3.5	-2.5	FILL SM	110	105	28	0	28	0
-2.5	-19	SM	115	110	30	0	30	0
-19	-30	SM	110	105	29	0	29	0

5.3.2 Stability Analyses

Considerations to evaluate the stability of the proposed wall include global and lateral stability. Global stability analyses were only considered for a sudden high water event equal to Standard Project Flood (SPF) conditions.

During SPF, water levels upstream would suddenly rise approximately to elevation 7.8 feet, while downstream conditions would be at the Mean High Higher Water (MHHW) level of 0.8 feet. Other global stability analyses were not deemed necessary as loading conditions on both sides of the proposed wall would be approximately the same as the weirs would be submerged.

Lateral stability was evaluated considering the impact force of a small boat which was assumed to be 500 pounds per foot applied to the top of the wall. The boat impact force was calculated using the kinetic energy principle and stopping distance. Calculations and assumptions are presented in Attachment B.

5.3.3 Stone Protection Design

Stone protection is included as a component of the proposed wall design. No flow analyses were available or performed for this project by the Hydrology and Hydraulics (H&H) group. Thus, the stone protection design was initially based on the previous design from Río Antón Ruiz authorized flood control project (2001). According to the previous design, all stone used should have a minimum unit weight of 160 pounds per cubic feet. The original riprap and bedding stone thickness was a minimum of 12 and 6 inches, respectively, well graded and the maximum riprap stone weight was 35 pounds. However, when comparing previous gradation to standard sizes from ASTM D6092, it was concluded that these gradations were customized for the project because there is no standard gradation that meet the same requirements for riprap stone and bedding layer. Moreover, for the new design, it was determined that additional protection was required to protect a potential scour zone that would result from a SPF event approximately 25 feet downstream from the location of the wall. The scour zone would require a minimum average stone size from 8 to 10 inches which would be a larger size than the original riprap design. Therefore, the new riprap gradation was revised to meet the scour zone requirement as shown below in Tables 3 and 4. The riprap to be considered is an R-60 standard riprap gradation following ASTM D6092, with a bedding layer of No. 1 stone, with maximum aggregate size of 4 inches. The thickness of the riprap should be revised to 1.5 feet and be increased by 50% if placed in the wet or under water to provide for uncertainties associated with this type of placement. The new bedding layer should be 9 inches thick as a minimum.

Table 6. Rip Rap Gradation

R-60 Riprap Standard Gradation	
Percent Finer by Weight	Stone Size (inches)
100	13.6
50	10.0
15	8.0
0	5.5

Table 7. Bedding Stone Gradation

No. 1 Bedding Stone	
Stone Size (inches)	Percent Finer by Weight
4	100
3 1/2	95
2 1/2	42.5
1 1/2	7.5
3/4	2.5

5.3.4 Sheet Pile Design

The CWALSHT software was used to estimate the minimum required sheet pile tip elevation considering a cantilever type sheet pile wall system to satisfy the structural stability of the wall system. Usual and impact loading cases were evaluated using the *S* soil strength parameters as discussed earlier. The water level in the channel used in the analyses was the Mean Lower Low Water level (MLLW) at elevation of -0.768 feet. The safety factor used for design of the wall was 1.5 based on Table 5-1 from Engineering Manual, EM-1110-2-2504, *Design of Sheet Pile Walls*. The results of the design indicate the tip of the wall should be embedded to an elevation of -24.0 feet.

5.3.5 Seismic Evaluation

The project is located within the Cerro Mula Fault Zone (CMFZ). Therefore, seismicity should be evaluated in the design. Engineering Regulation, ER 1110-2-1806, *Earthquake Design and Evaluation for Civil Works Projects*, "Table B-1", established a hazard potential classification for civil work projects. Based on this criteria, the potential hazard related to failure of the wall during a major seismic event is in the low category. Failure would not likely result in loss of life from inundation, should not significantly affect lifelines or critical structures, should not result in property losses, and would result in minimal incremental damage with respect to environmental impacts.

Based on the United States Geological Survey (USGS) seismic hazard map, ground accelerations for the Humacao area with a spectral acceleration of 0.3 seconds and an earthquake recurrence of approximately every 500 years or 10% in 50 years, ranges between 0.30g and 0.40g. For a spectral acceleration of 0.3 seconds and an earthquake recurrence of approximately every 2,500 years or 2% in 50 years, ranges between 0.70g and 1.2g.

5.3.5.1 Liquefaction

Liquefaction potential was evaluated using the ground accelerations discussed above for a 500 years event and an estimated earthquake magnitude of 5.8 based on historical earthquakes reported by the Puerto Rico Seismic Network at the Humacao station between years 2008 and 2010 with magnitudes between 5.4 and 6.1. Analysis results indicated factors of safety between 0.26 and 1.1 using a correction factor for overburden. These results indicate liquefaction is likely given the granular foundation and seismic conditions as earlier discussed. Estimated liquefaction induced settlement values using the correlation between corrected N values and cyclic stress ratio (Seed et al., 1984), indicated an approximately 16 inches of settlement. Calculations are included in the Appendix.

5.4 Design Phase Recommendations

This section describes the considerations to be taken into account for the design and implementation phase of the project.

5.4.1 Subsurface Investigations

Soils information used for this feasibility study was from investigations taken in the prior Section 205 project in the vicinity of the current project site. In order to obtain site specific conditions and narrow soil parameters of the area, it is recommended that site specific investigations are performed.

5.4.2 Seismic

Seismic evaluation specific to the site was not available. However, studies on the Cerro Mula Fault and nearby faults indicate no recent fault movement or displacement have occurred. Conversely, the seismic history of Puerto Rico indicate tremors could be expected, although should be minor. Seismic evaluation should be considered in the wall design including the Design Earthquake and Most Credible Earthquake values as well as measures to prevent potential liquefaction.

5.4.3 Stone Protection

Stone protection was based on previous design of the project and assumptions made for the scour zone downstream from the wall. Conditions could have changed throughout the years following the construction. Therefore, flow evaluation and hydraulic conditions should be evaluated in order to determine the corresponding flow velocities within project limits. Knowing the associated flow velocities in the canal would aid in designing the required stone protection.

6.0 Civil/Site

6.1 Site Layout

The site layout for the permanent notched weir structures will be placed at the same locations as the temporary SWIMs for Alternative 1, as shown in Attachment A drawing. The location for the one permanent weir structure (Alternative 2) will be placed near the mouth of the diversion channel, as shown in Attachment A drawing. The existing project right-of-way/easement will be used and can accommodate the project features. No additional lands or easements are anticipated.

6.2 Access

The previous Section 205 Flood Damage Reduction Project right-of-way/easement will be utilized for this project. The right-of-way allows for approximately 20 ft of access on either side of the levee that is adjacent to the diversion channel. The levee itself has a crown width of 12 ft with 1V:3H side slopes and can be used for access as well. Integrity of the levee for use as an access route will be assessed during the design phase. No additional lands or easements are anticipated for construction or maintenance of the project features.

Navigational aids and/or channel markers should be provided within the channel to direct boat traffic through the notches in the sheet pile weirs.

6.3 Staging/Stockpiling Areas

There are areas along either side of the levee (approximately 20 ft on either side) that can be used as staging or stockpiling areas for the limited amount of equipment and materials that will be used for this project. There is also an approximately 1 acre triangular area between the diversion channel and the levee, where the diversion channel veers further north away from the levee. This area was a previous disposal and borrow area for both the prior Section 205 project and temporary SWIM construction project, and can be used for staging/stockpiling areas for this project. It is not anticipated that any additional staging or stockpiling areas will be needed for the project construction or maintenance.

6.4 Relocations

There are no known or observed utilities or facilities within the project right-of-way.

7.0 Structural Requirements

7.1 Design Basis

Options for the Rio Anton Ruiz Restoration include a single, notched weir (Alternative 2) and two notched weirs (Alternative 1). The notched weirs will have soil at equal elevations on both sides; thus, they do not typically resist a load from retained soil. Two load cases were investigated: 1) 5-foot scour occurring on one side of the weir; and 2) impact by a small commercial watercraft. The weir was designed as a cantilever sheet pile wall in accordance with USACE criteria, EM 1110-2-2504, *Design of Sheet Pile Walls* (March 1994).

7.2 Design Analysis

Soil properties were obtained from prior geotechnical investigations for use in sheet pile analysis software CWALSHT. For design, a simplified single soil layer was assumed, using the properties of the worst condition soil layer detailed. Wall friction was ignored during design. Both load cases were considered Usual in accordance with EM 1110-2-2504. A stability design with associated safety factor for the sheet pile was performed only; structural design was conservatively based on results of the stability analysis. Results of the sheet pile design can be found in the structural Attachment C. A more refined, less conservative approach to the design analyses may be beneficial during design and implementation phase to obtain a more optimized design. The design was checked for correctness and conformance with USACE design criteria.

7.3 Sheet Pile

Due to its wide availability and history of use, hot rolled steel sheet pile was selected. A PZC-13 steel sheet pile section was assumed for design. Since the weir will be permanently submerged, marine grade ASTM A690 sheet pile was selected for its resistance to corrosion.

7.4 Concrete Cap

The weir will have a typical 1.5-ft by 1.5-ft reinforced concrete cap. Effects of the concrete cap are negligible and thus were ignored during design.

7.5 Design Phase Recommendations

During the design phase, more data should be collected for analysis including refined soil strata parameters, wave effects, scour depths, and additional anticipated impact forces. Additional load cases should be analyzed including both stability and structural analyses. The sheet pile material should be investigated during final design including the use of cold rolled steel or vinyl sheet pile sections to possibly reduce costs.

8.0 Recommended Plan

Alternative 1 provides for permanent notched weirs at the existing temporary SWIM locations. These SWIM features at the locations and design elevations being used for Alternative 1 were monitored for salinity levels. The salinity monitoring stations showed a decrease in the salinity levels, indicating the temporary SWIM features performed as needed.

Alternatives 1a and 1b both provided some benefits with less cost. However, neither meet all of the project objectives and did not provide additional or same benefits for less cost. Thus, they were screened out.

Alternative 2 is providing for a less costly alternative by placing only one weir, near the mouth of the diversion channel. To make use of one weir, it was placed at prior to the confluence to Rio Anton Ruiz and the diversion channel. This ideally would reduce salinity levels in both channels. However, that location places it downstream of the discharge culvert that passes through the levee. This could impact a portion of the originally authorized Section 205 by reducing the level of flood protection provided by the culverts discharging into the diversion channel from Punta Santiago community. In addition, the location of the weir placed it in a cultural resource area, near a highway bridge and sandbar limiting locations for the weir to be placed. Thus, Alternative 2 was screened out.

The non-structural plan is placement of sand at the mouth of the channel, basically recreating the natural sand bar that develops there. The Sponsor was in favor of the natural occurring sand bar where it develops over time but "blows out" during large storm events to allow the flows to discharge. This alternative would not be a permanent feature. Once constructed, it would serve its purpose until a large storm event occurred. After a large storm event, the Sponsor would have to recreate the sand bar with additional sand placement as an operation and maintenance activity. It would be difficult to create a maintenance schedule that would mimic that of a naturally forming sandbar and would create higher long term maintenance costs. The natural ability of it to wash out during high water or storm events would also render it immediately ineffective after such an event, until the maintenance activity could be moved out to correct it. Sand is also not an easily accessible source on the island and would further increase long term maintenance costs. The alternative is not be a locally preferred plan. This alternative was screened out due to the higher long term maintenance costs, and would also not act as a permanent feature.

Based on the monitoring data gathered, the temporary SWIM features successfully functioned as designed. Thus, Alternative 1 was selected as the recommended plan for the permanent features.

9.0 Construction Procedures

The construction sequence for the project is anticipated to be installation of erosion and sediment control features including silt fence along the work perimeters and floating turbidity barriers within the Rio Anton Ruiz and diversion channels, upstream and downstream of the structure locations. The structures will be sheet pile driven from the bank of the Rio Anton Ruiz and the diversion channel. The sheet pile weirs will have a concrete cap. Depending on the tidal conditions, there may be the need to draw down the water level directly adjacent to the sheet pile in order to construct the concrete cap. Sheet pile or other means to create a small dewatering cell and use of pumping directly back into the channel should be sufficient if the concrete cap is placed in sections. No diversion of water (diversion channel) is anticipated for the dewatering efforts.

10.0 Environmental Objective and Requirements

Environmental objectives and requirements are discussed in the main body of the Feasibility Report. The objectives include reducing salinity levels to below 10-12ppt, improving and increasing pterocarpus forest habitat, increasing submerged and emergent aquatic vegetation spatial extent, and improving habitat for beneficial freshwater fish species previously in the project area.

11.0 Operation and Maintenance

The operation and maintenance manual for the previous Section 205 project will still apply. The Rio Anton Ruiz and diversion channels shall be kept clear of debris and vegetation with regular clearing of the channel. The new sheet pile weirs shall be monitored for any cracking or spalling on the concrete; evidence of significant corrosion or tilting of the sheet pile; or any observed damage to the project features.

12.0 Access Roads

Access to the site will be via existing public roadways, and then via the existing project right-of-way. No additional temporary or permanent access roads are anticipated.

13.0 Cost Estimates

Cost estimates for each alternative are provided in the Cost Appendix to the Feasibility Report, separate from this engineering appendix. The estimated construction cost for each alternative is listed below (construction cost only, does not include contingency):

- Alternative 1 $2,100,000
- Alternative 1a $1,571,000
- Alternative 1b $1,264,000
- Alternative 2 $1,350,000

A cost estimate was not conducted on Non-structural plan as it would be an O&M plan and cost and not a construction cost. It was screened out as previously discussed. Other measures or features (such as replacing culvert) that were screened out in early in the plan formulation process did not proceed on for costs. The recommended plan, Alternative 1, construction cost with contingency is $2,167,000. While it is the most costly alternative, it is the only one that meets all of the objectives, provides full benefits, and also provides for more benefits (output or habitat units) for the cost.

14.0 Schedule for Design and Construction

The design including review periods is expected to take approximately 6-7 months to complete. Construction of the project is anticipated to take 282 days (approximately 9-10 months) to complete.

15.0 References

References used or referenced, not already referenced within text above:

Carter, L. M., J. W. Jones, L. Berry, V. Burkett, J. F. Murley, J. Obeysekera, P. J. Schramm, and D. Wear. (2014). Ch. 17: Southeast and the Caribbean. Climate Change Impacts in the United States: The Third National Climate Assessment, J. M. Melillo, Terese (T.C.) Richmond, and G. W. Yohe, Eds., U.S. Global Change Research Program, 396-417.

Friedman, D., J. Schechter, B. Baker, C. Mueller, G. Villarini, and K.D. White. (2016). US Army Corps of Engineers Nonstationarity Detection Tool User Guide. US Army Corps of Engineers: Washington, DC.

NOAA. 2013. Regional Climate Trends and Scenarios for the U.S. National Climate Assessment. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Environmental Satellite Data, and Information Service: Washington, DC. http://www.nesdis.noaa.gov/technical_reports/142_Climate_Scenarios.html

United States Army Corps of Engineers. (2016). Engineering and Construction Bulletin No. 2016-25: Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects. US Army Corps of Engineers: Washington, DC.

White, K.D., J.R. Arnold. (2015). Recent US Climate Change and Hydrology Literature Applicable to US Army Corps of Engineers Missions – Caribbean Region 21. US Army Corps of Engineers: Washington, DC.

ER 1110-1-12, Engineering and Design Quality Management

EC 1165-2-214, Civil Works Review Policy

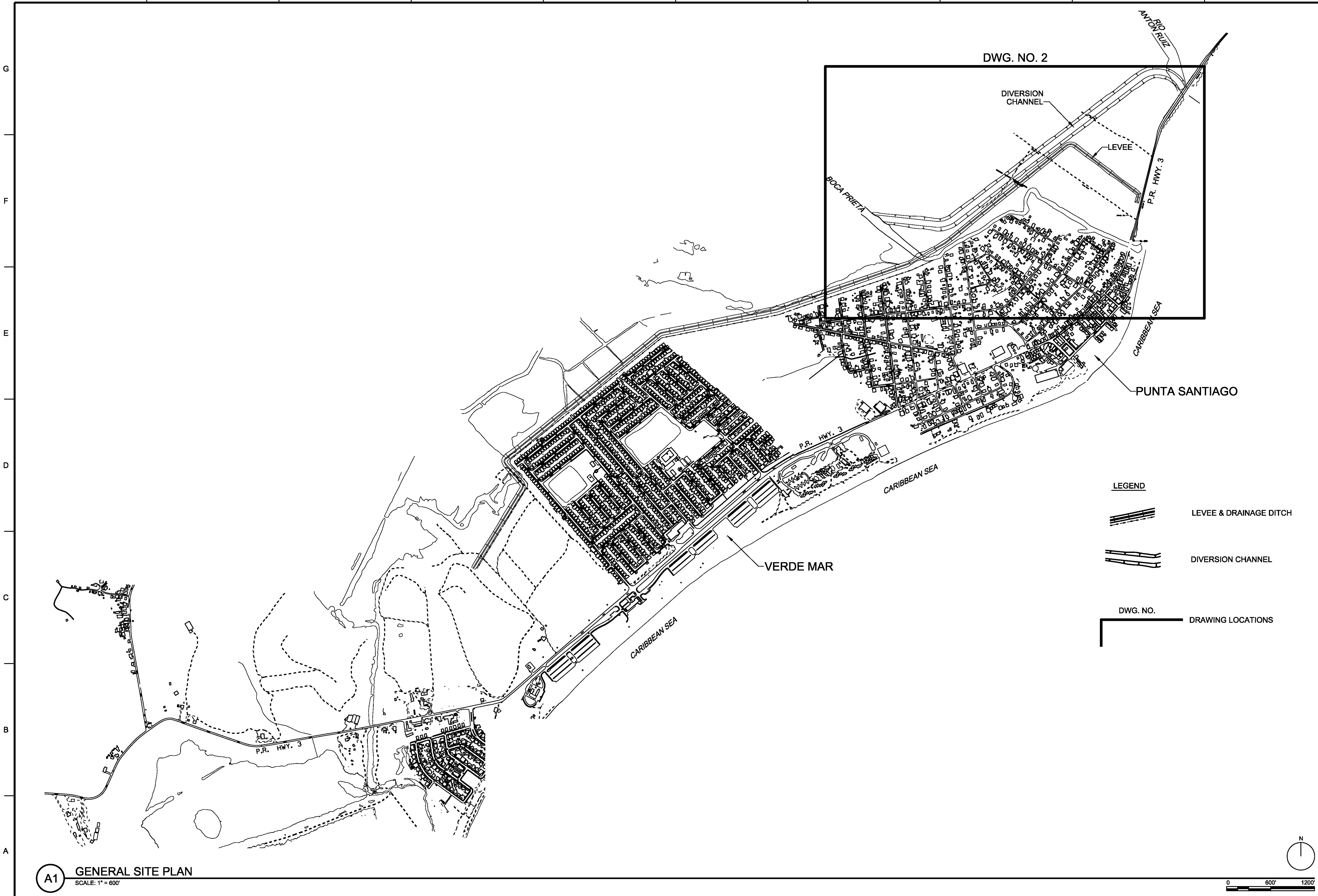
ER 1110-2-1150, Engineering and Design for Civil Works (Appendix C covers EN Appendix to Feasibility Report)

Memorandum, 9 April 2015, CESAD-CG, subject: South Atlantic Division Regional Programmatic Review Plan for the Continuing Authorities Program

Memorandum, 19 January 2011, CECW-p, subject: Continuing Authority Program Planning Process Improvements
Agency Technical Review Guidance for Cost Engineering Products, 30 April 2010
00600-SAJ Documents and Records
02614-SAJ Quality Control of In-House Products: Civil Works Feasibility
33500-SAJ After Action Review and Lessons Learned

Attachment A

Site Layouts



A1 GENERAL SITE PLAN

SCALE: 1" = 600'

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

U.S. ARMY CORPS OF ENGINEERS
JACKSONVILLE DISTRICT
701 SAN MARCO BLVD
JACKSONVILLE FL, 32207

DESIGNED BY:	ISSUE DATE: MMMMMMMM YYYY
DRAWN BY:	SOLICITATION NO.: XXXXXX-XX-X-XXXX
CHECKED BY:	CONTRACT NO.: N/A
SUBMITTED BY:	DRAWING DATE: 7/5/2016
SIZE: ANSI D	FILE NAME:

CAP SECTION 1135 RIO ANTON RUIZ
HUMANCO, PUERTO RICO

GENERAL SITE PLAN

SHEET ID

1

pw:\\COE-SAJPWP02JAX.saj.ds.usace.army.mil:SAJPW\Documents\CAP Projects\Rio Anton Ruiz (1135)\080 - Plans\05 Civil\Working\EXH-01

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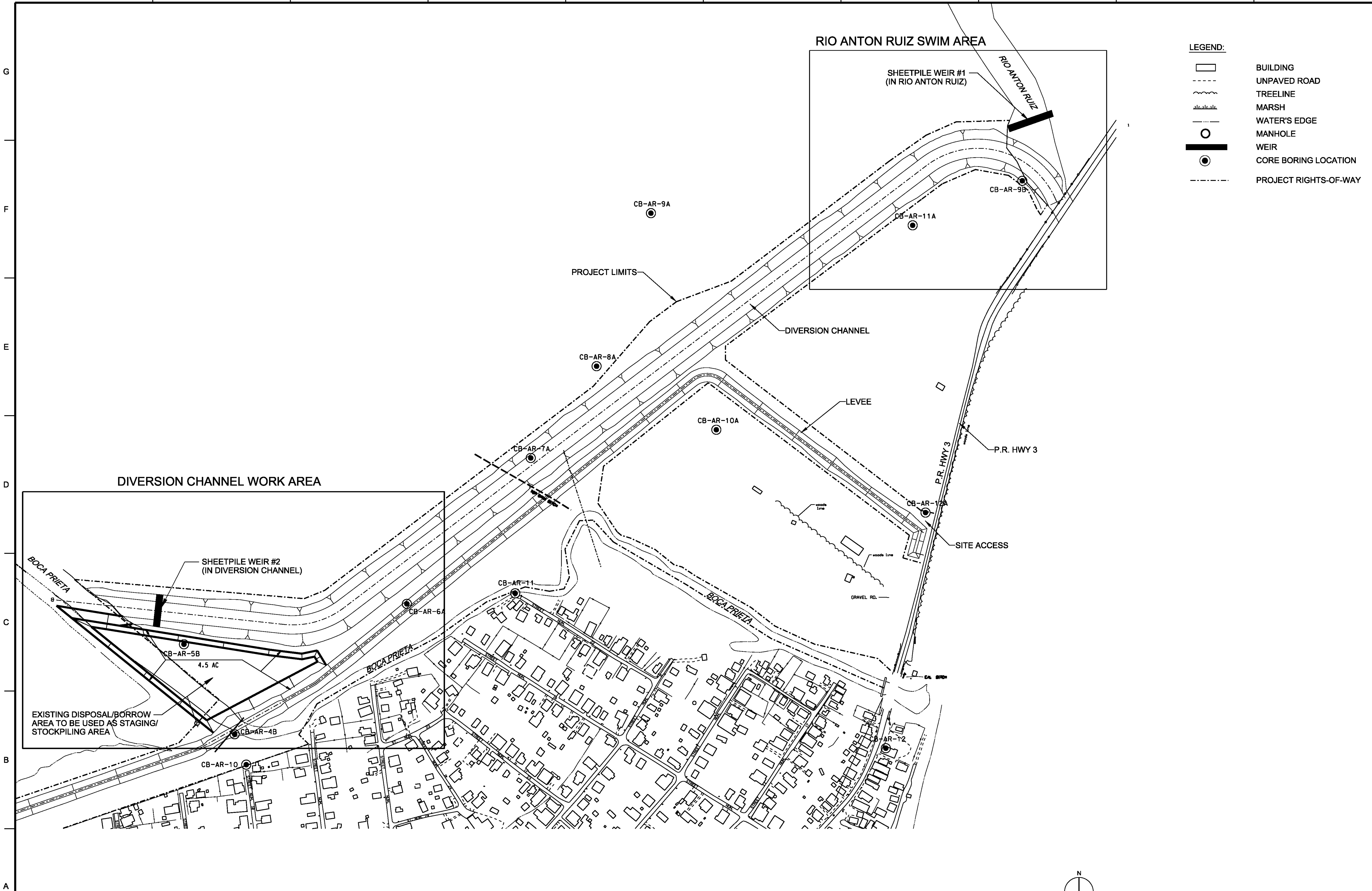

A1 ALTERNATIVE 1 PLAN
SCALE: 1" = 200'

US Army Corps of Engineers®

MARK	DESCRIPTION	DATE

DESIGNED BY:	ISSUE DATE: MMMMMMMMM YYYY
DRAWN BY:	SOLICITATION NO.: XXXXXX-XX-X-XXXX
CHECKED BY:	CONTRACT NO.: N/A
SUBMITTED BY:	DRAWING DATE: 7/5/2016
SIZE: ANSI D	FILE NAME:

U.S. ARMY CORPS OF ENGINEERS
JACKSONVILLE DISTRICT
701 SAN MARCO BLVD
JACKSONVILLE FL. 32207

CAP SECTION 1135 RIO ANTON RUIZ
HUMACAO, PUERTO RICO

ALTERNATIVE 1 PLAN

SHEET ID
2

pw:\\COE-SAJPWP02JAX.saj.ds.usace.army.mil:SAJPW\Documents\CAP Projects\Rio Anton Ruiz (1135)\080 - Plans\05 Civil\Working\EXH-02

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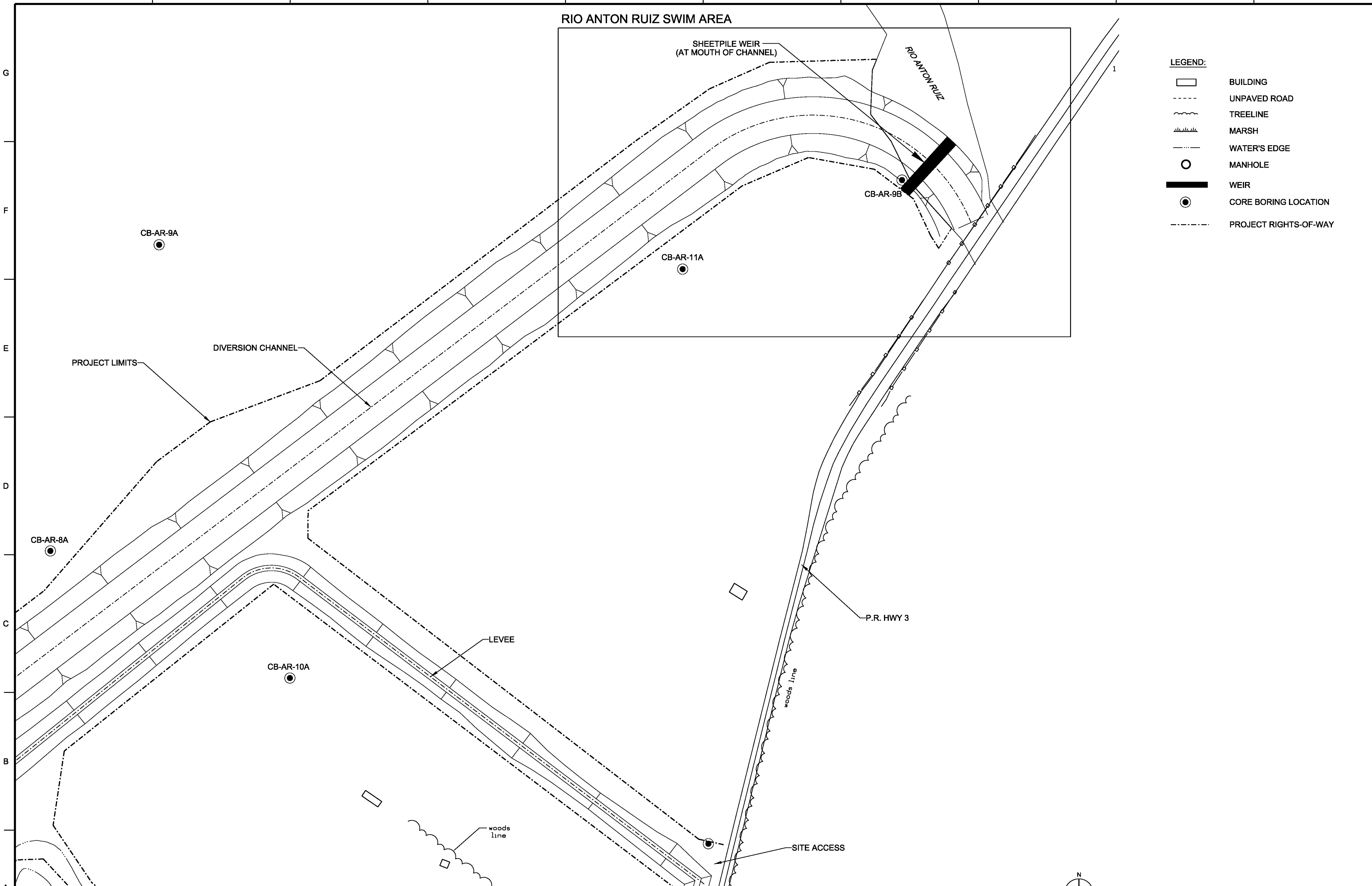

A1 ALTERNATIVE 2 PLAN

SCALE: 1" = 100'

DESIGNED BY:	ISSUE DATE: MMMMMMMMM YYYY
DRAWN BY:	SOLICITATION NO.: XXXXXX-XX-X-XXXX
CHECKED BY:	CONTRACT NO.: N/A
SUBMITTED BY:	DRAWING DATE: 7/5/2016
SIZE: ANSI D	FILE NAME:

MARK	DESCRIPTION	DATE

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U.S. ARMY CORPS OF ENGINEERS
JACKSONVILLE DISTRICT
701 SAN MARCO BLVD
JACKSONVILLE FL, 32207

CAP SECTION 1135 RIO ANTON RUIZ
HUMANCO, PUERTO RICO

ALTERNATIVE 2 PLAN

SHEET ID

3

pw:\\COE-SAJPWP02JAX.saj.ds.usace.army.mil:SAJPW\Documents\CAP Projects\Rio Anton Ruiz (1135)\080 - Plans\05 Civil\Working\EXH-03

%STATUS%

Attachment B

Geotechnical Data and Analyses

The following soil parameters are provided for the design of a steel sheet pile wall. The short-term, long-term and seismic conditions should all be analyzed, and the most critical condition used for design purposes. These parameters are based on boring log CB-AR-11, laboratory testing results and typical soil data from the area available through USDA web soil survey and per Table 3-1 from EM 1110-2-2504.

RIO ANTON RUIZ RESTORATION PROJECT SOIL PARAMETERS															
Elevation (NAVD 88)			Depth (ft)			USCS	γ_{sat} (pcf)	γ' (pcf)	Undrained (Q)		Drained (S)		Seismic (0.80R)	
									ϕ (deg.)	c (psf)	ϕ' (deg.)	c' (psf)	ϕ' (deg.)	c' (psf)
3.5	to	-2.5	0	to	-6	FILL SM	110	105	28	0	28	0	26	0
-2.5	to	-19	-6	to	-22.5	SM	115	110	30	0	30	0	28	0
-19	to	-30	-22.5	to	-33.5	SM	110	105	29	0	29	0	25	0

Rio Anton Ruiz Restoration Project
Punta Santiago, Humacao, Puerto Rico

El of Ground Water	-1.43
El of Ground	3.5
Weight of Water	62.4
Hammer Type	Safety
p_a	1 Reference Stress (tsf)
N_{SPT}	Blows per foot from boring logs
N_{60}	Normalized to effective energy delivered
N_1	Normalized to 1 tsf
$(N_1)_{60}$	Normalized to 1tsf and effective energy delivererd
C_{ER}	Rod energy correction factor, Table 3-2a
C_N	Overburden correction factor, Table 3-2b

Normalized Blow Counts*

Boring Designation	Depth	El	Drilling_Method	NSPT	γ (pcf)	σ'v (ksf)	C_N	C_{ER}	N_{60}	N_1	$(N_1)_{60}$	USCS	N_{60} Average
CB-AR-11	0	3.5	SPT	-	110	0.0	1.6	1				Fill - fine sand (SM)	5.0
CB-AR-11	1.5	2	SPT	5	110	0.2	1.6	1	8.0	17.4	27.9		
CB-AR-11	3	0.5	SPT	2	110	0.3	1.6	1	3.2	4.9	7.9		
CB-AR-11	4.5	-1	SPT	2	110	0.5	1.6	1	3.2	4.0	6.4		
CB-AR-11	6	-2.5	SPT	3	110	0.6	1.6	1	4.8	5.5	8.8		
CB-AR-11	7.5	-4	SPT	4	115	0.7	1.3	1	5.2	6.8	8.8	Sand, very fine grained (SM)	6.0
CB-AR-11	9	-5.5	SPT	7	115	0.8	1.3	1	9.1	11.2	14.6		
CB-AR-11	10.5	-7	SPT	5	115	0.9	1.3	1	6.5	7.6	9.9		
CB-AR-11	12	-8.5	SPT	19	115	0.9	1.3	1	24.7	27.7	36.1		
CB-AR-11	13.5	-10	SPT	9	115	1.0	1	1	9.0	12.6	12.6		
CB-AR-11	15	-11.5	SPT	5	115	1.1	1	1	5.0	6.8	6.8		
CB-AR-11	16.5	-13	SPT	2	115	1.2	1	1	2.0	2.6	2.6		
CB-AR-11	18	-14.5	SPT	1	115	1.3	1	1	1.0	1.3	1.3		
CB-AR-11	19.5	-16	SPT	3	115	1.3	1	1	3.0	3.7	3.7		
CB-AR-11	21	-17.5	SPT	2	115	1.4	1	1	2.0	2.4	2.4		
CB-AR-11	22.5	-19	SPT	2	115	1.5	1	1	2.0	2.3	2.3		
CB-AR-11	24	-20.5	SPT	2	110	1.5	1	1	2.0	2.3	2.3	Sand, very fine grained (SM)	2.0
CB-AR-11	25.5	-22	SPT	2	110	1.5	1	1	2.0	2.3	2.3		
CB-AR-11	27	-23.5	SPT	1	110	1.6	1	1	1.0	1.1	1.1		
CB-AR-11	28.5	-25	SPT	2	110	1.7	1	1	2.0	2.2	2.2		
CB-AR-11	30	-26.5	SPT	2	110	1.7	1	1	2.0	2.1	2.1		
Average=				4.0					5.0	6.0	8.0		

*See sheet "Ref and Eq" for references used to create this spreadsheet

TABLE 3-2

Relative Density and N_{60}

a. Rod Energy Correction Factor C_{ER}
(Data from Tokimatsu and Seed 1987)

Country	Hammer	Hammer Release	C_{ER}
Japan	Donut	Free-Fall	1.3
	Donut	Rope and Pulley with special throw release	1.12*
USA	Safety	Rope and Pulley	1.00*
	Donut	Rope and Pulley	0.75
Europe	Donut	Free-Fall	1.00*
China	Donut	Free-Fall	1.00*
	Donut	Rope and Pulley	0.83
		Auto Hammer	1.3

*Methods used in USA

b. Correction Factor C_N (Data from Tokimatsu and Seed 1984)

C_N	σ'_{vo}*, ksf
1.6	0.6
1.3	1.0
1.0	2.0
0.7	4.0
0.55	6.0
0.50	8.0

*σ'_{vo} = effective overburden pressure

$$N_{60} = C_{ER} \cdot C_N \cdot N_{SPT}$$

EM 1110-1-1905 BEARING CAPACITY OF SOILS

$$N_1 = N_{SPT}\sqrt{\frac{p_A}{\sigma'_v}} \qquad (N_1)_{60} = N_{\boxed{60}}\sqrt{\frac{p_A}{\sigma'_v}}$$

Rodrigo & Salgato The Engineering of Foundations 1Ed, Interpretation of SPT Results p. 290-291

Table 21. Relationship among relative density, penetration resistance, dry unit weight, and angle of internal friction of cohesionless soils (after Duncan and Buchignani, 1976)

Descriptive Relative Density	Relative Density **	Standard Penetration Resistance N_1 (see Note) *	Static Cone Resistance q_c	Angle of Internal Friction ϕ	Dry Unit Weight	
	%	blows/foot	tsf or kgf/cm²	degrees	KN/m³	PCF
Very Loose	< 15	< 4	< 50	< 30	< 14	<90
Loose	15 - 35	4 - 10	50 - 100	30 - 32	14 - 16	90 - 100
Medium Dense	35 - 65	10 - 30	100 - 150	32 - 35	16 - 18	100 - 115
Dense	65 - 85	30 - 50	150 - 200	35 - 38	18 - 20	115 - 130
Very Dense	85 - 100	> 50	> 200	> 38	> 20	>130

* N_1 = N -value corrected to an effective vertical overburden pressure of 1.0 tsf or 100 kPa

** Freshly deposited, normally consolidated sand

Note: As originally proposed, this correlation used the uncorrected SPT blowcount, N_1. However, hammers delivering 60% of the theoretical energy have been the most commonly used hammers for SPT tests, and it seems likely that the data on which the correlation was based was obtained primarily from tests with such hammers. It therefore seems logical to use $N_{1,60}$ with this correlation, and it is the recommendation of this report that this be done.

Duncan and Buchignani, 1976

Report—Physical Soil Properties

Physical Soil Properties–Humacao Area, Puerto Rico Eastern Part														
Map symbol and soil name	Depth	Sand	Silt	Clay	Moist bulk density	Saturated hydraulic conductivity	Available water capacity	Linear extensibility	Organic matter	Erosion factors			Wind erodibility group	Wind erodibility index
										Kw	Kf	T		
	In	Pct	Pct	Pct	g/cc	micro m/sec	In/In	Pct	Pct					
Ad—Aguadilla loamy sand														
Aguadilla	0-8	-84-	- 9-	3- 8- 12	1.50-1.53 -1.55	42.00-92.00-14 1.00	0.03-0.06-0. 08	0.0- 1.5- 2.9	1.0- 2.0- 3.0	.05	.05	5	2	134
	8-58	-97-	- 2-	1- 2- 3	1.50-1.55 -1.60	42.00-92.00-14 1.00	0.02-0.03-0. 04	0.0- 1.5- 2.9	0.0- 0.2- 0.3	.05	.05			
Cm—Coastal beaches														
Coastal beaches	0-6	-98-	- 2-	0- 1- 1	1.35-1.60 -1.85	42.00-92.00-14 1.00	0.03-0.04-0. 05	0.0- 1.5- 2.9	0.0- 0.1- 0.1	.05	.05		1	220
	6-80	-93-	- 7-	0- 1- 1	1.35-1.60 -1.85	42.00-92.00-14 1.00	0.03-0.04-0. 05	0.0- 1.5- 2.9	0.0- 0.1- 0.1	.10	.10			
Ts—Tidal swamp														
Tidal swamp	0-60	—	—	—	—	0.42-0.90-1.40	—	—	—			5	8	0

conversion 1 g/cc = 62.428 pcf

Data Source Information

Soil Survey Area: Humacao Area, Puerto Rico Eastern Part
Survey Area Data: Version 8, Sep 29, 2015

Table 3-1
Granular Soil Properties (after Teng 1962)

Compactness	Relative Density (%)	SPT N (blows per ft)	Angle of Internal Friction (deg)	Unit Weight Moist (pcf)	Unit Weight Submerged (pcf)
Very Loose	0-15	0-4	<28	<100	<60
Loose	16-35	5-10	28-30	95-125	55-65
Medium	36-65	11-30	31-36	110-130	60-70
Dense	66-85	31-50	37-41	110-140	65-85
Very Dense	86-100	>51	>41	>130	>75

DRILLING LOG	DIVISION: SOUTH ATLANTIC	INSTALLATION: JACKSONVILLE DISTRICT	SHEET 1 OF 2 SHEETS

1. PROJECT	ANTON RUIZ AT PUNTA SANTIAGO
2. LOCATION (Coordinates or Station)	X= 225088 Y= 37560
3. DRILLING AGENCY	GEOCONSULT
4. HOLE NO. (As shown on drawing title and file number)	CB-AR- 11
5. NAME OF DRILLER	J. Nieves
6. DIRECTION OF HOLE	☒ VERTICAL ☐ INCLINED ___ DEG. FROM VERT.
7. THICKNESS OF OVERBURDEN	
8. DEPTH DRILLED INTO ROCK	
9. TOTAL DEPTH OF HOLE	30.0' feet
10. SIZE AND TYPE OF BIT	
11. DATUM FOR ELEVATION SHOWN (TBM or MSL)	MSL
12. MANUFACTURER'S DESIGNATION OF DRILL	CME- 45B
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN	DISTURBED: 20 / UNDISTURBED:
14. TOTAL NUMBER CORE BOXES	
15. ELEVATION GROUND WATER	-1.43 feet
16. DATE HOLE	STARTED: 3-28-90 / COMPLETED: 3-28-90
17. ELEVATION TOP OF HOLE	3.57 feet
18. TOTAL CORE RECOVERY FOR BORING	53 %
19. SIGNATURE OF INSPECTOR	KERMIT SCHMIDT, PE

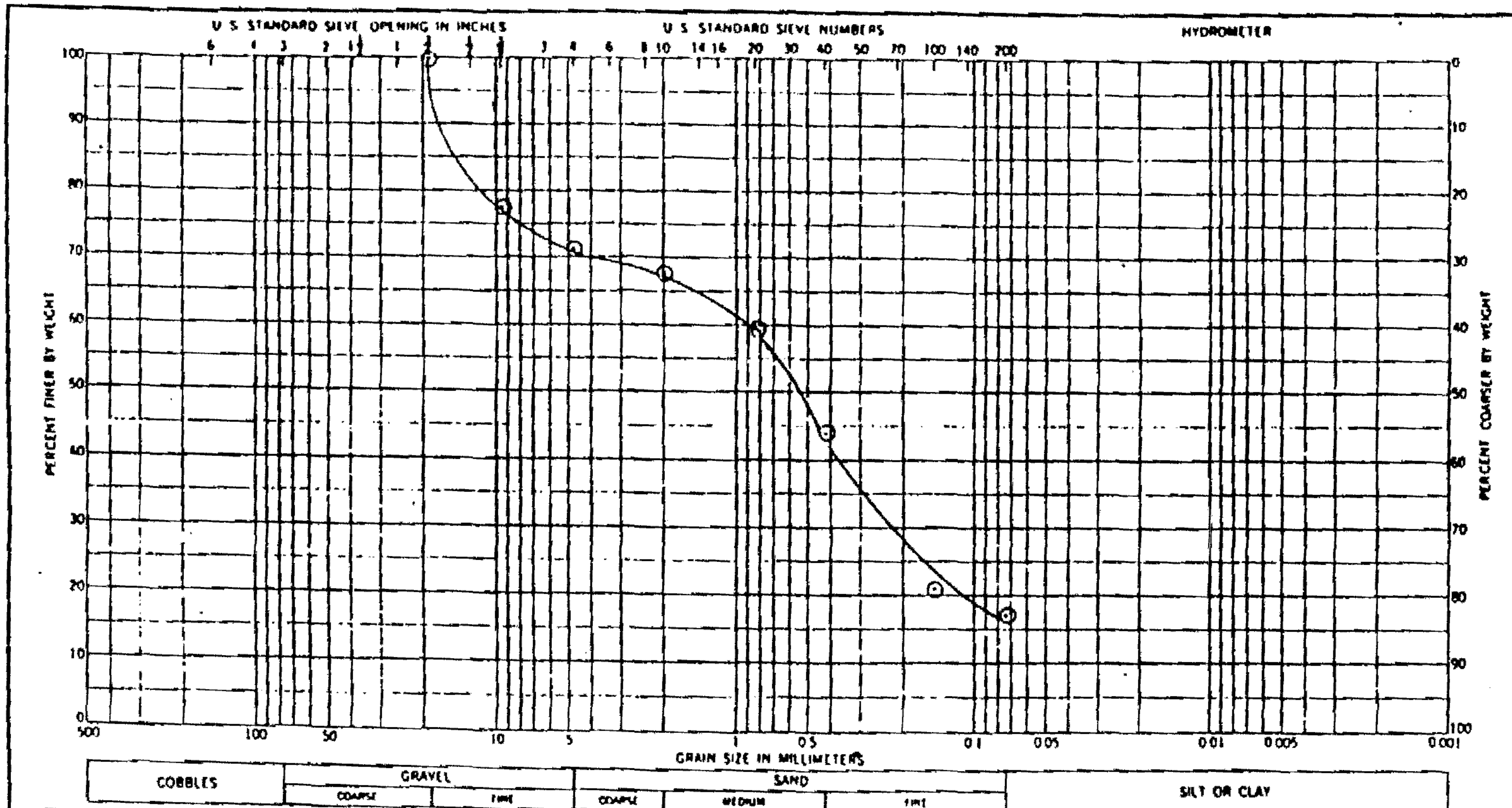
ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g	BLS/6"
3.57	0.0					BIT OR BARREL	
			FILL, sand fine grined, silty yellowish brown, some rock fragments, little clay, occ. roots (SM)	17	1	140# hammer with 30" drop on 1-3/8" ID x 2" OD std. spl. spoon	1 / 2 / 3
0.57	3.0'			17	2		3 / 1 / 1
			very silty, brown and gray, trace asphalt waste.	50	3		2 / 1 / 1
-2.43	6.0'			11	4		1 / 1 / 2
-3.93	7.5'		Sand, very fine grained, gray, some silt, little peat, fibrous shelly, brown mottled (SM)	33	5		1 / 2 / 2
			little silt and shell fragments	39	6		3 / 4 / 3
-6.93	10.5'			56	7		4 / 3 / 2
			shelly	94	8		6 / 9 / 10
				89	9		5 / 4 / 5
-11.43	15.0'			94	10		2 / 3 / 2
			silty and shelly	28	11		0 / 1 / 1
				67	12		1 / 0 / 1
-15.93	19.5'			89	13		1 / 1 / 2
[illegible]	21.0'		some coral	28	14		2 / 1 / [illegible]

ENG FORM [illegible] PROJECT HOLE NO.

DRILLING LOG (Cont Sheet) ELEVATION TOP OF HOLE 3.57 feet | Hole No. CB-AR- 11

PROJECT: ANTON RUIZ AT PUNTA SANTIAGO | INSTALLATION: JACKSONVILLE DISTRICT | SHEET 2 OF 2 SHEETS

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
-17.43	21.0					
-21.93	22.5			39	15	2/1/1
			Clay, very soft, silty olive gray, some sand, coral stone, occ. shell fragments (CL)	44	16	2/1/1
				39	17	1/1/1
				44	18	1/0/1
				94	19	1/1/1
-26.43	30.0			94	20	1/1/1



Sample No	Elev or Depth	Classification	Nat w %	LL	PL	PI
1	0 - 1.5'	Fill, sand, fine grain, silty, yellowish brown, some rock fragments, little clay, occ. roots (SM)	21			

Project Anton Ruiz at Punta Santiago
Assignment #5

Area Humacao, Puerto Rico

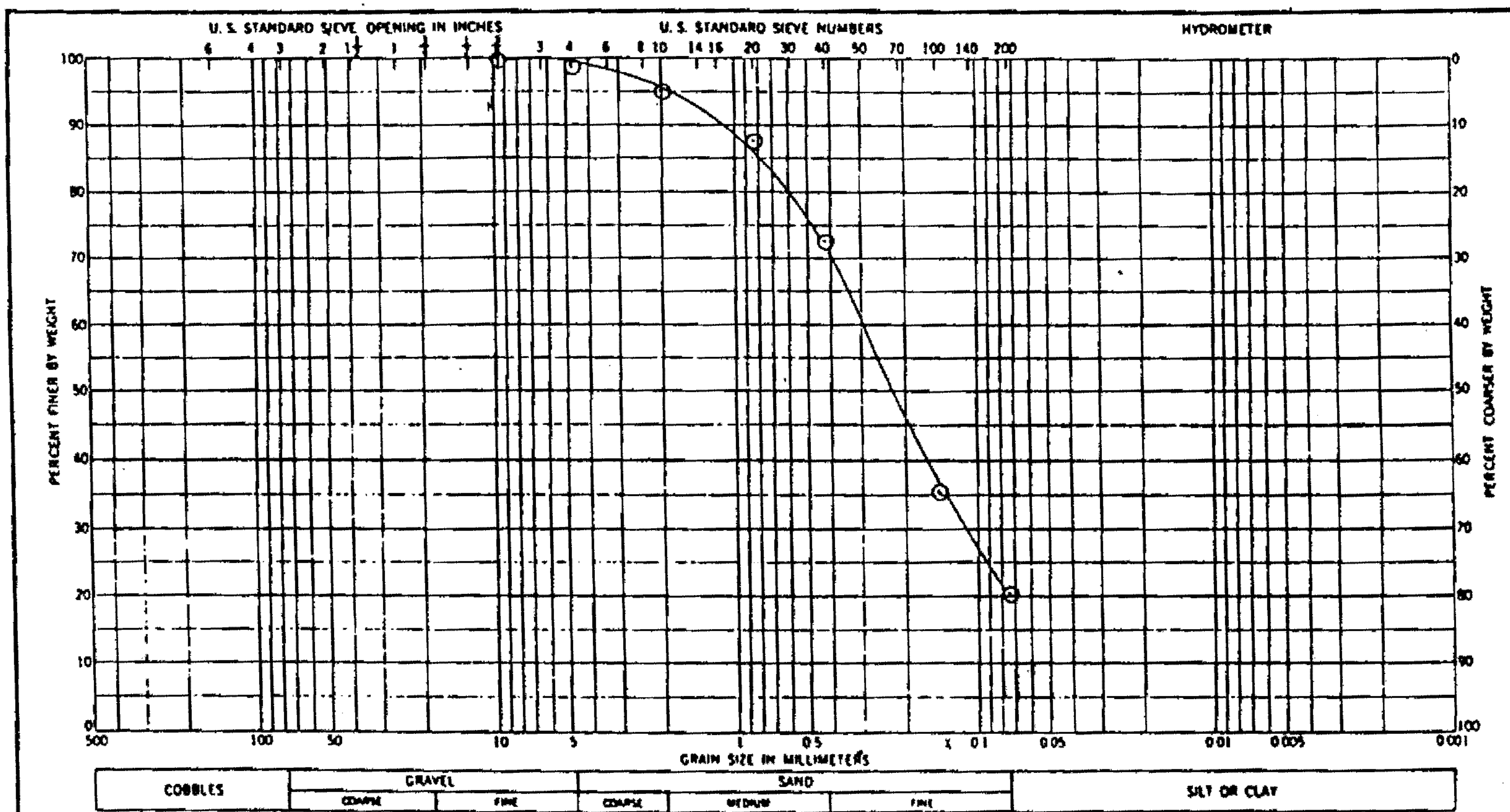
Boring No CB-AR-11

Date 5-21-90

GRADATION CURVES

ENG FORM 1 MAY 63 2087

PLATE C-



Sample No	Elev or Depth	Classification	Nat w %	LL	PL	PI
4	4 - 6.5'	Fill, sand, fine, grain, very silty, brown and gray, little clay (SM)				

Project Anton Ruiz at Punta Santiago
Assignment #5

Area Humacao, Puerto Rico

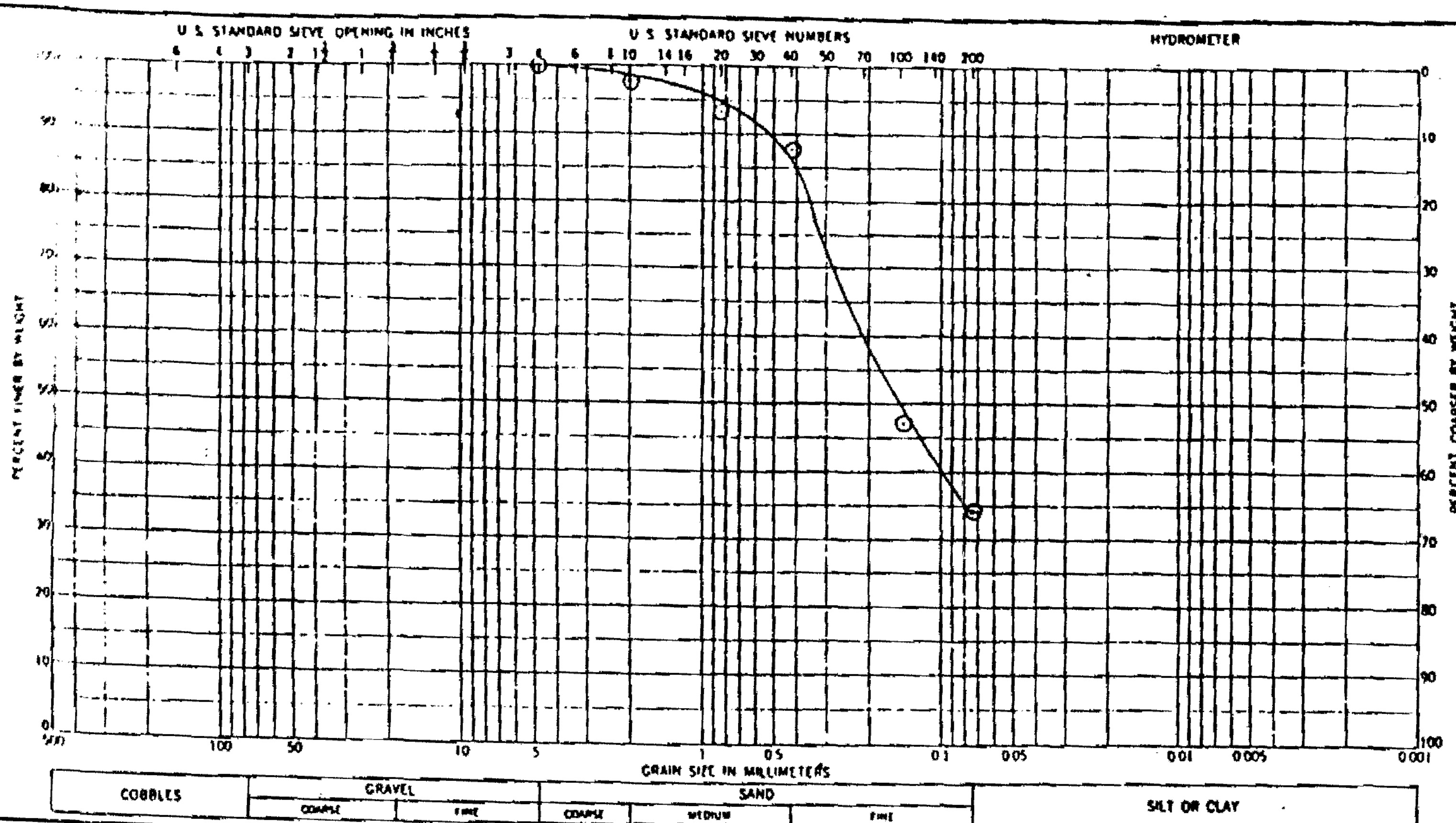
Boring No CB-AR-11

Date 5-21-90

GRADATION CURVES

ENG FORM 1 MAY 63 2087

PLATE C-7E



Sample No	Elev or Depth	Classification	Nat w %	LL	PL	PI
11	15 - 16.5'	Sand, very fine grain, gray, some silt, little peat, fibrous, shelly (SM)	50			

Project Anton Ruiz at Punta Santiago
Assignment #5

Area Humacao, Puerto Rico

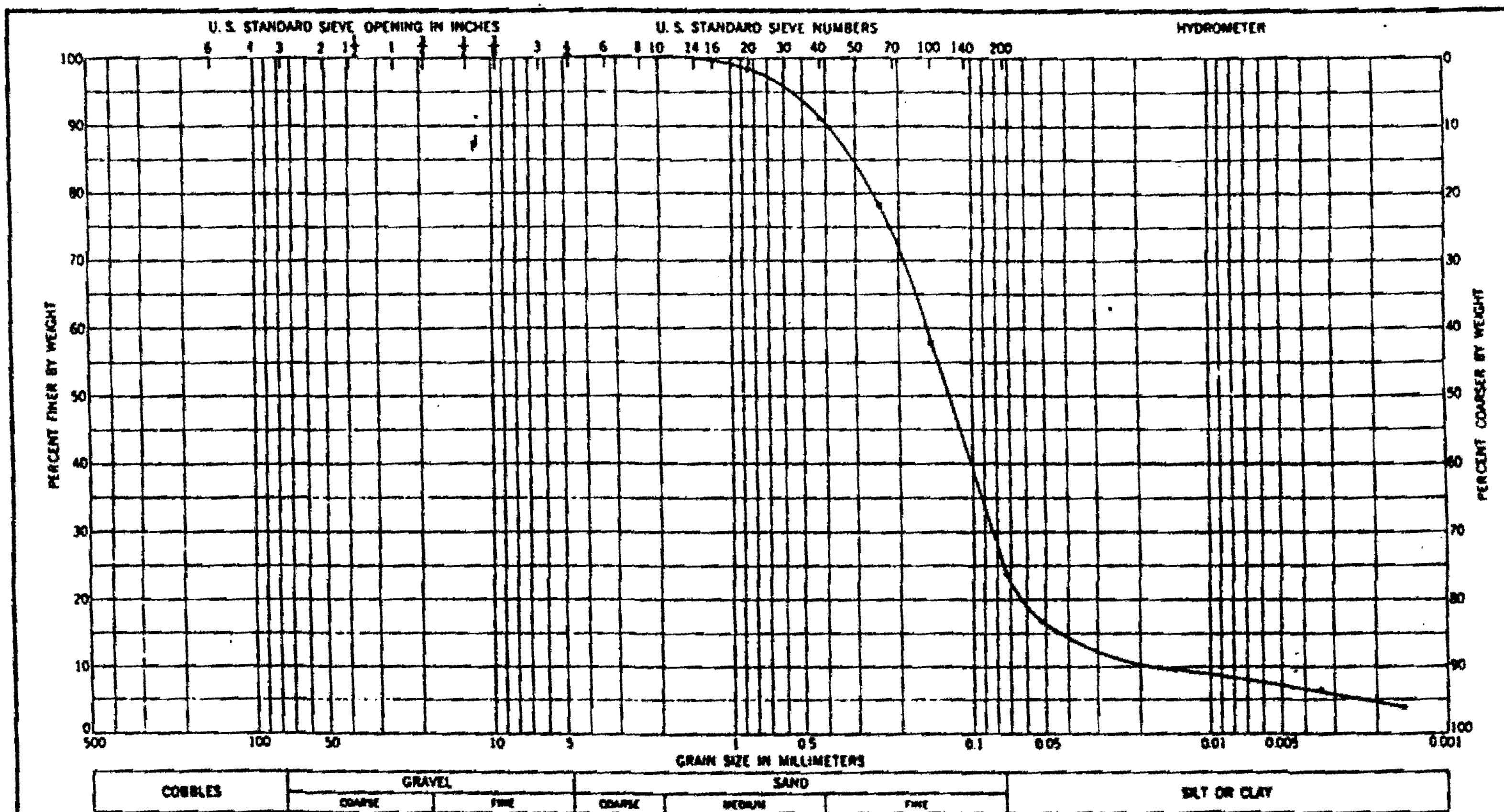
Boring No CB-AR-11

Date 5-21-90

GRADATION CURVES

ENG FORM 1 MAY 63 2087

PLATE (



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
-	25.0-27.0'	Gray, silty sand (SM) w/ shell fragments	32.1	NP	NP	NP

GRADATION CURVES

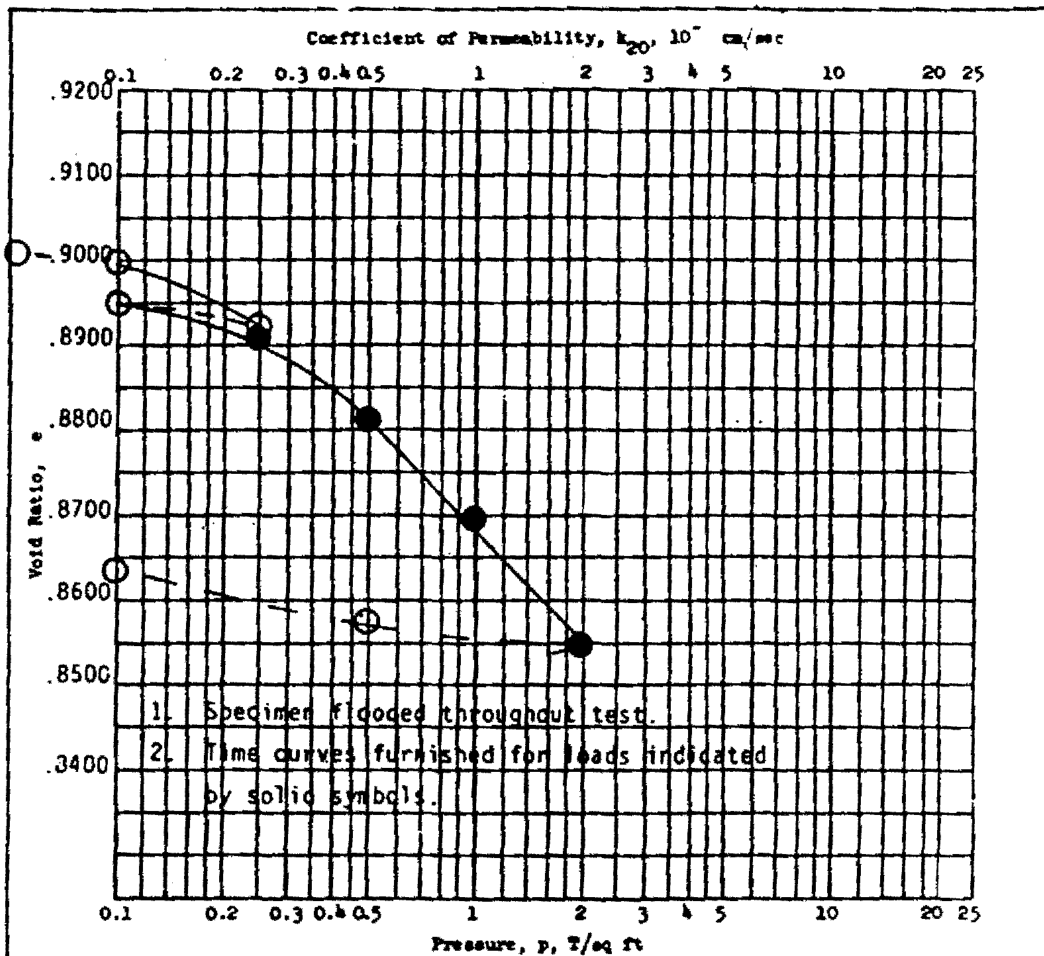
Project	Anton Ruiz Project
	Lab. No. 258/837*
Area	--
Boring No.	CB-AR-11
Date	26 Sept 90

ENG FORM 1 MAY 63 2087

*Consolidation, Q&$\overline{R}$ Triaxial Tests

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER NO. 6204
Req. No. RM-CW-90-0215



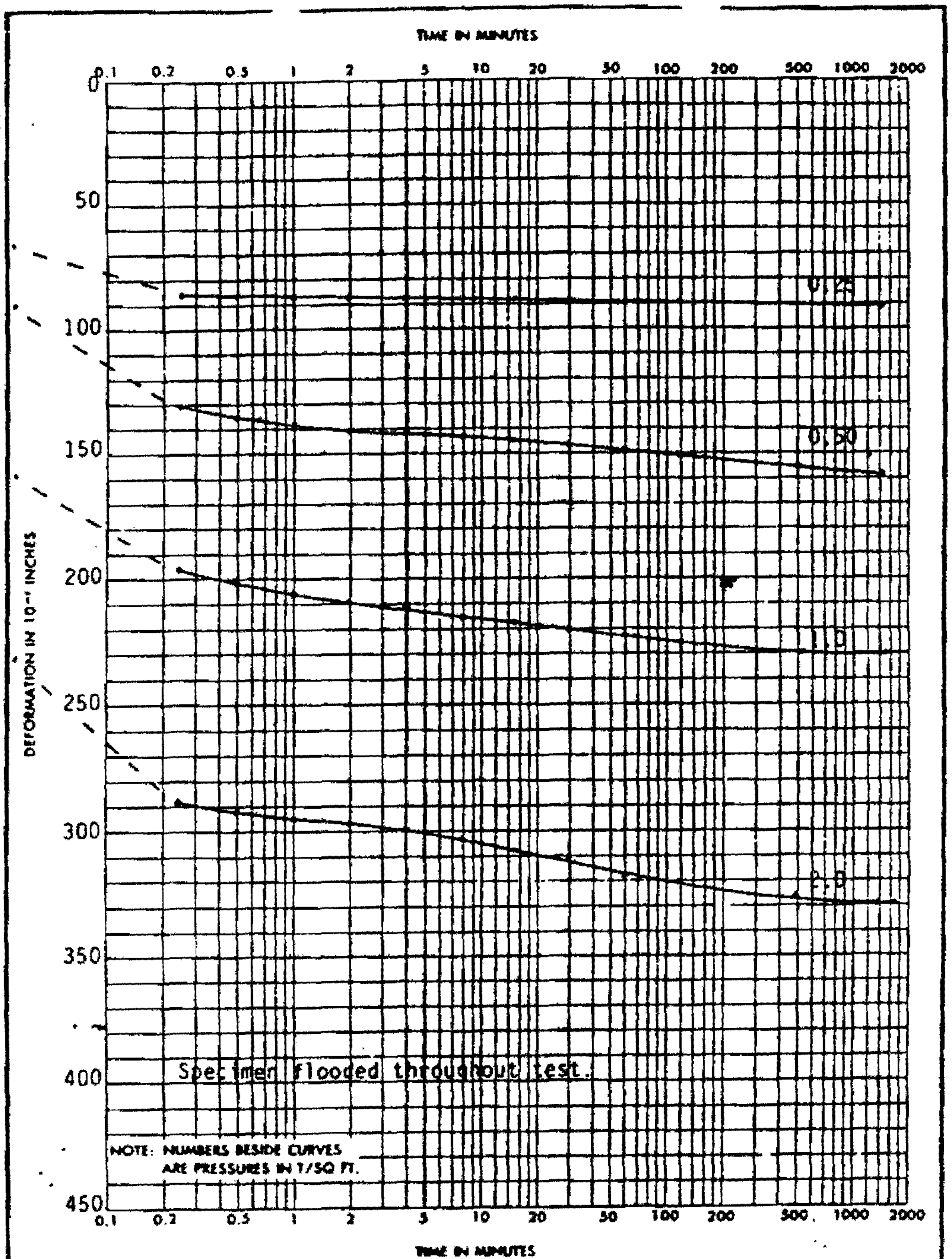
Type of Specimen: undisturbed		Before Test		After Test	
Diam 3.99 in.	Ht 1.25 in.	Water Content, w_o	32.1 %	w_f	31.6 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	.905	e_f	.864
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	96.7 %	S_f	100.0 %
Compression Index, C_c		Dry Density, γ_d	89.5 lb/ft^3		
Classification: Gray, silty sand (SM), w/ shell fragments		k_{20} at e_o = × 10^{-} cm/sec			
LL NP	G_s 2.73	Project: Anton Ruiz Project			
PL NP	D_{10} 0.018mm	Lab. No. 258/837			
Remarks: See gradation curve and additional information on ENG Form 2087.		Area --			
		Boring No. CB-AR-11		Sample No. --	
		Depth 25.0-27.0'		Date 26 Sept 90	

CONSOLIDATION TEST REPORT

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE

DEPARTMENT OF THE ARMY, SOUTH ATLANTIC DIVISION LABORATORY
CORPS OF ENGINEERS, 611 SOUTH COBB DRIVE, MARIETTA, GA. 30060

WORK ORDER NO. 6204
Req. No. RM-CW-90-0215

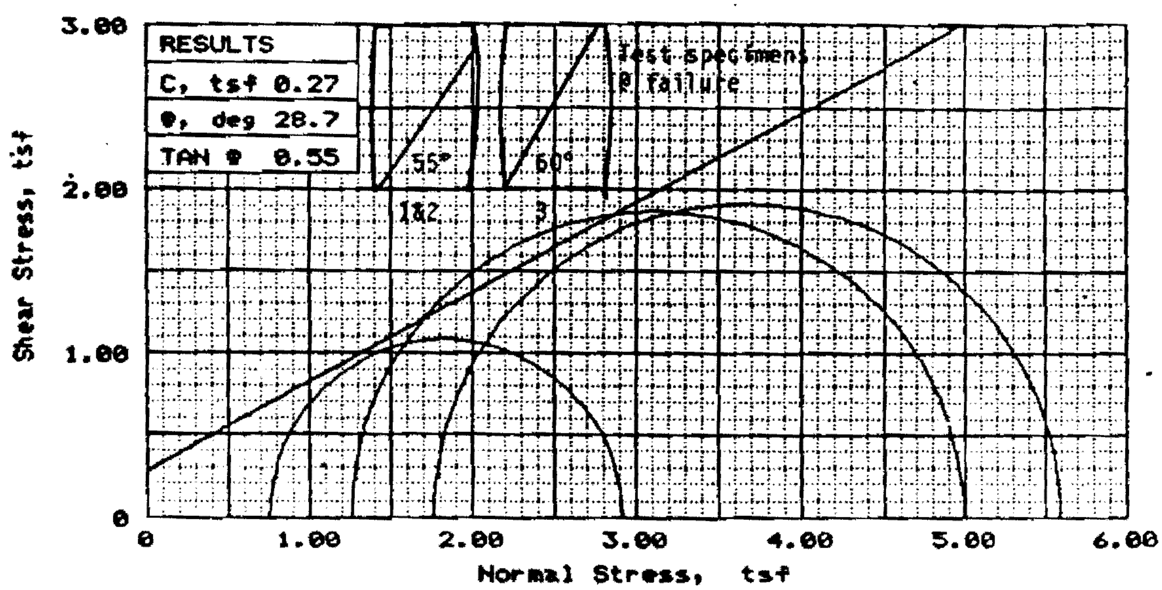


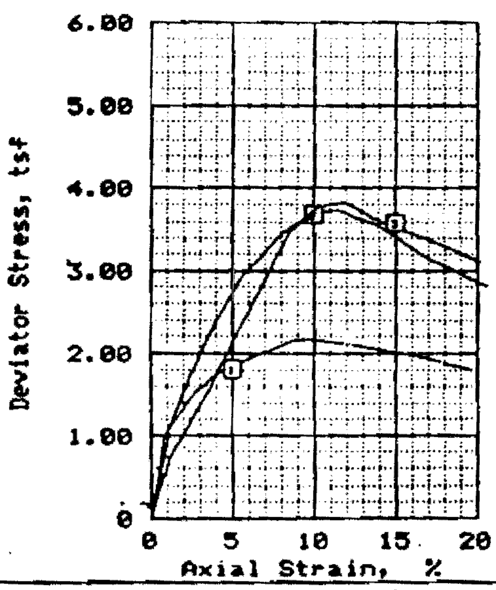
PROJECT Anton Ruiz Project			
AREA Lab. No. 258/837			
BORING NO CB-AR-11	SAMPLE NO --	DEPTH 25.0-27.0'	DATE 26 Sept 90

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST—TIME CURVES** (TRANSLUCENT)

SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS - MARIETTA, GEORGIA

Requisition No. RM-CW-90-0215 Work Order No. 6204





SPECIMEN NO.		1	2	3
INITIAL	WATER CONTENT, %	32.4	32.7	32.0
	DRY DENSITY, pcf	83.3	86.1	87.0
	SATURATION, %	84.5	91.0	90.9
	VOID RATIO	1.046	0.980	0.959
	DIAMETER, in	1.38	1.38	1.38
	HEIGHT, in	3.07	3.07	3.07
@ TESTING	WATER CONTENT, %	34.4	34.8	33.8
	DRY DENSITY, pcf	83.3	86.1	87.0
	SATURATION, %	89.7	97.0	96.1
	VOID RATIO	1.046	0.980	0.959
	DIAMETER, in	1.38	1.38	1.38
	HEIGHT, in	3.07	3.07	3.07
BACK PRESSURE, tsf		5.04	5.04	5.04
INIT. EFF. STR., tsf		0.76	1.26	1.76
MAX. DEV. STRESS, tsf		2.16	3.73	3.83
PORE PRESSURE, tsf				
TIME TO FAILURE, min.		14	15	16
RATE, %/min.		0.69	0.76	0.74
ULT. DEV. STRESS, tsf		2.01	3.42	3.53

DESCRIPTION: GRAY, SILTY SAND (SM), WITH SHELL FRAGMENTS.

LL= NP	PL= NP	PI= NP	Gs= 2.73	CONTROLLED STRAIN TEST
TYPE OF SPECIMEN: UNDISTURBED				TYPE OF TEST: Natural Q

REMARKS: SEE GRADATION CURVE ON ENG FORM 2087.

PROJECT: ANTON RUIZ

AREA

LAB NO. 258/837

BORING NO. CB-AR-11

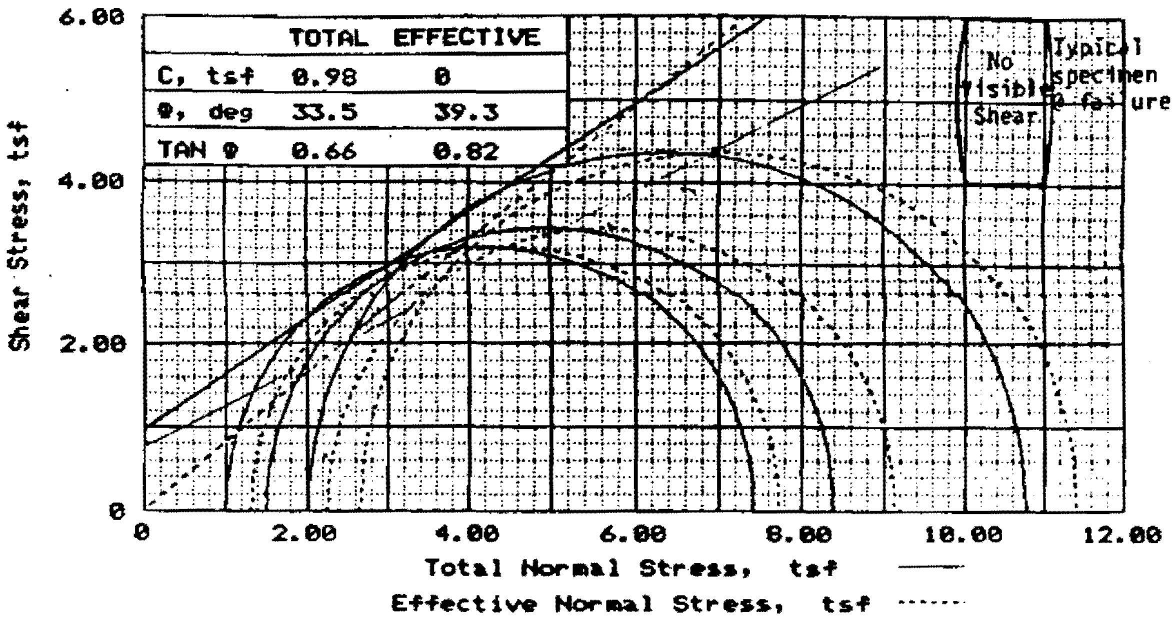
SAMPLE NO. ---

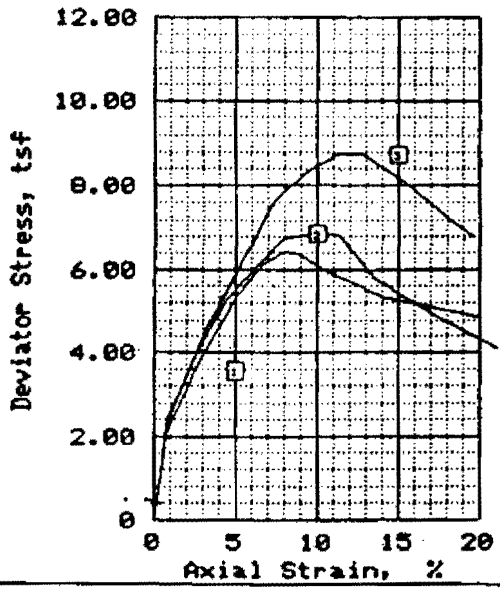
DEPTH/~~ELEV~~ 25.0-27.0'

LABORATORY: CESAD-EN-FL DATE 3 OCTOBER 199(

TRIAXIAL COMPRESSION TEST

	TOTAL	EFFECTIVE
C, tsf	0.98	0
Φ, deg	33.5	39.3
TAN Φ	0.66	0.82





	SPECIMEN NO.	1	2	3
INITIAL	WATER CONTENT, %	32.3	31.6	32.5
	DRY DENSITY, pcf	88.0	88.7	87.8
	SATURATION, %	94.0	93.7	94.3
	VOID RATIO	0.937	0.921	0.941
	DIAMETER, in	1.38	1.38	1.38
	HEIGHT, in	3.07	3.07	3.07
@ TESTING	WATER CONTENT, %	33.0	32.7	32.9
	DRY DENSITY, pcf	89.6	90.0	89.7
	SATURATION, %	100.0	100.0	100.0
	VOID RATIO	0.901	0.893	0.899
	DIAMETER, in	1.37	1.37	1.37
	HEIGHT, in	3.05	3.05	3.05
	BACK PRESSURE, tsf	5.04	5.04	5.04
	INIT. EFF. STR., tsf	1.01	1.51	2.02
	MAX. DEV. STRESS, tsf	6.42	6.89	8.75
	PORE PRESSURE, tsf	-0.31	-0.75	-0.65
	TIME TO FAILURE, min.	70	85	110
	RATE, %/min.	0.12	0.12	0.10
	ULT. DEV. STRESS, tsf	5.28	5.46	8.19

DESCRIPTION: GRAY, SILTY SAND (SM), WITH SHELL FRAGMENTS.

LL= NP	PL= NP	PI= NP	Gs= 2.73	CONTROLLED STRAIN TEST
TYPE OF SPECIMEN: UNDISTURBED				TYPE OF TEST: R w/pore pressures

REMARKS: SEE GRADATION CURVE ON ENG FORM 2087.

PROJECT: ANTON RUIZ

AREA

LAB NO. 258-837

BORING NO. CB-AR-11

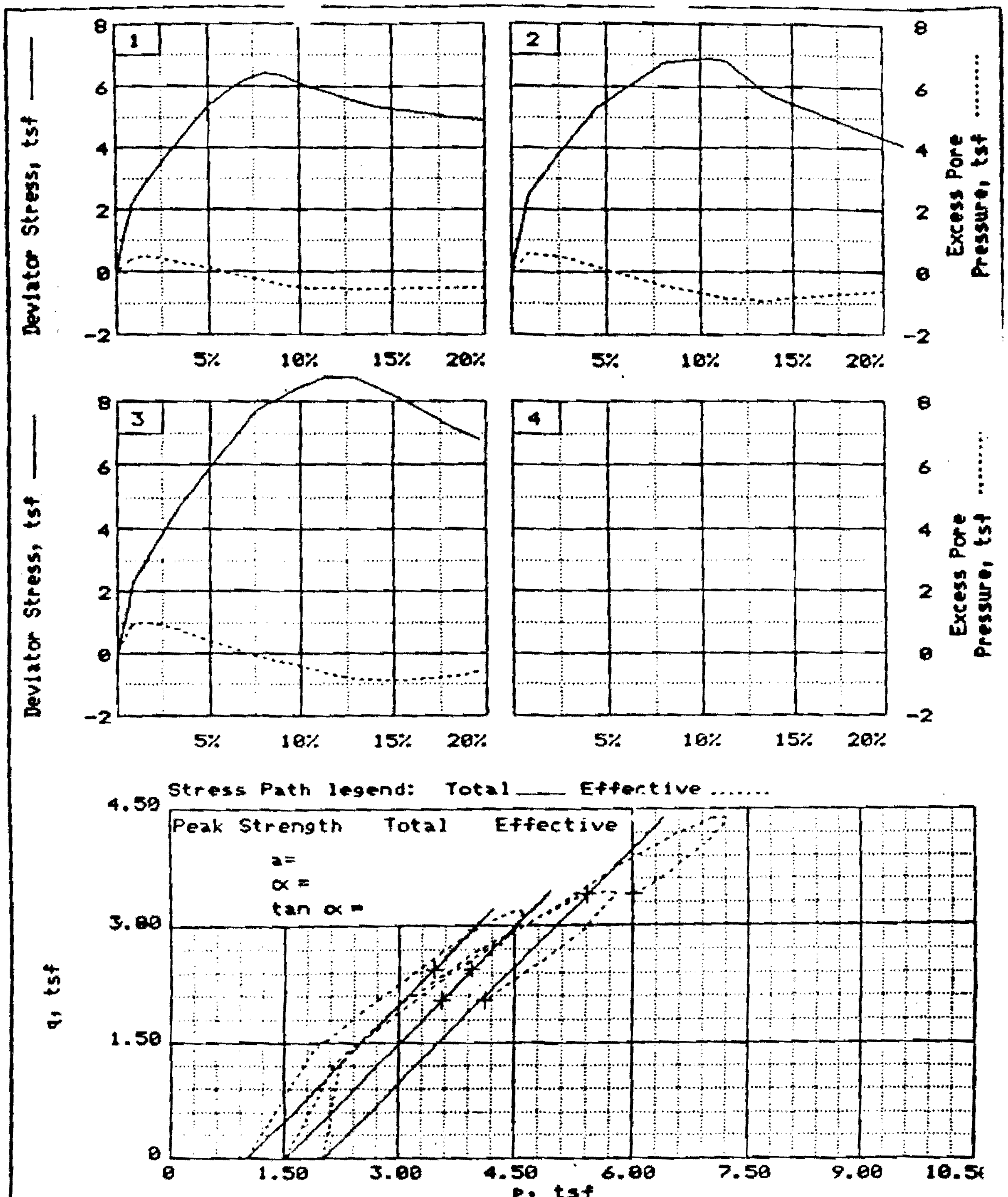
SAMPLE NO. ---

DEPTH/~~ELEV~~ 25.0-27.0'

LABORATORY: CESAD-EN-FL DATE 3 OCTOBER 1990

pg 1 of 2 TRIAXIAL COMPRESSION TEST

SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS - MARIETTA, GEORGIA
Work Order No. 6204
Requisition No. RM-CW-90-0215



PROJECT: ANTON RUIZ

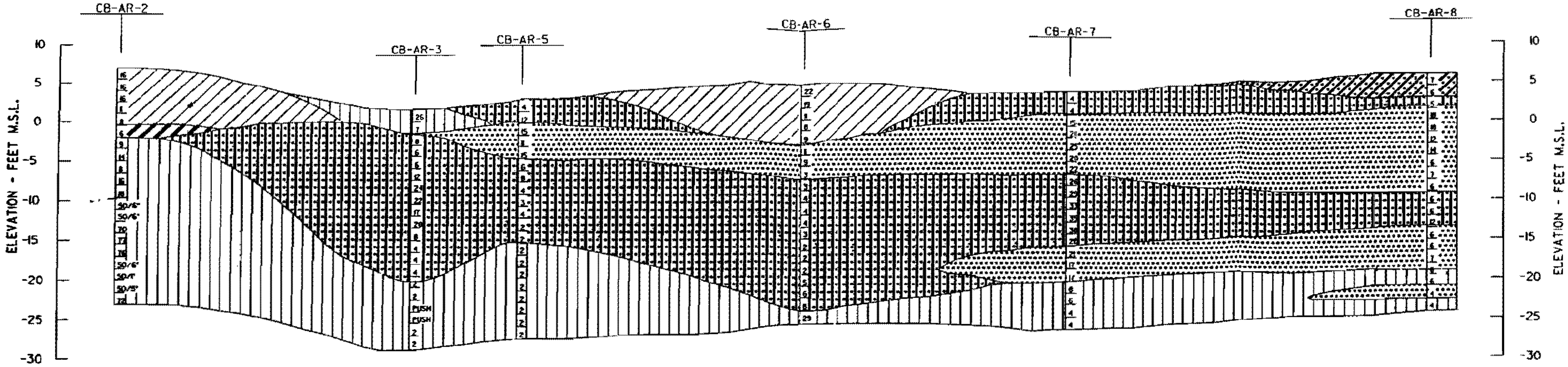
BORING CB-AR-11 SAMPLE --- DEPTH/~~ELEV~~ 25.0-27.0'

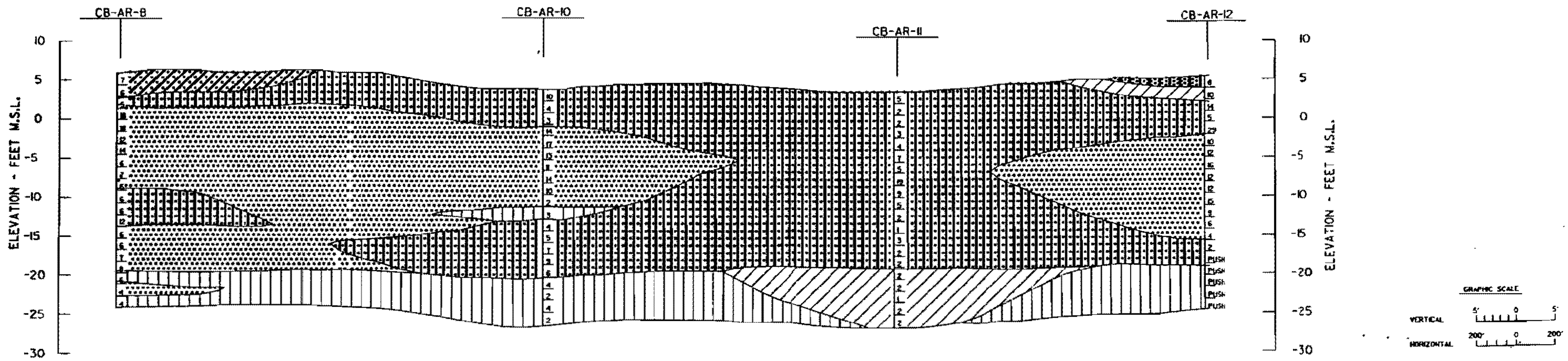
TYPE OF TEST: R w/pore pressures LAB NO. 258-837

LABORATORY: CESAD-EN-FL PAGE 2/2 DATE 3 OCTOBER 1990

SOUTH ATLANTIC DIVISION LABORATORY, CORPS OF ENGINEERS - MARIETTA, GEORGIA

Requisition No. RM-CW-90-0215 Work Order No. 6204





LEGEND

NUMBER OF HAMMER BLOWS REQUIRED TO ADVANCE A SPLIT SAMPLE SPOON (1 3/8" I.D. x 2" O.D.) ONE FOOT USING A 140 POUND HAMMER WITH A 30 INCH DROP. THE BLOW COUNT FOR THE FIRST ONE HALF FOOT OF EACH DRIVE IS HEREIN OMITTED FOR CONVENIENCE OF PRESENTATION.

- (GM) SILTY GRAVELS, GRAVEL-SILT MIXTURES
- (SP) POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES.
- (SM) SILTY SANDS, SAND-SILT MIXTURES.
- (CL) INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
- (SC) CLAYEY SANDS, SAND-CLAY MIXTURES
- INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASITCITY.
- INORGANIC CLAYS OF HIGH PLASITICITY, FAT CLAYS.

RIO ANTON RUIZ AT PUNTA SANTIAGO
HUMACAO, PUERTO RICO

DETAILED PROJECT REPORT

GEOLOGIC CROSS SECTION

SCALES AS SHOWN

DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

DATED: FILE NO. 101-36, 185 SHEET 1 OF 1

Boat Impact Force Calcs

Assumptions based on average boat data available

Small boat

- Length = 20 ft
- Width = 10 ft
- Weight = 3,000 lbs
- speed = 25 Knots
- distance to stop = 200 ft

Conversion:

$1 \text{ lb} = 0.031081 \text{ slug}$ $\quad$ slug units $= \dfrac{\text{lb}\cdot\text{s}^2}{\text{ft}}$

$1 \text{ mph} = 1.466667 \text{ ft/s}$ $\quad$ $1 \text{ knot} = 1.152 \text{ mph}$

Impact Force

$$F = \frac{1}{2}\frac{mv^2}{S}$$

S = stopping distance in ft

F = impact force in lbs

m = mass of boat in slugs

v = speed of boat in ft/s

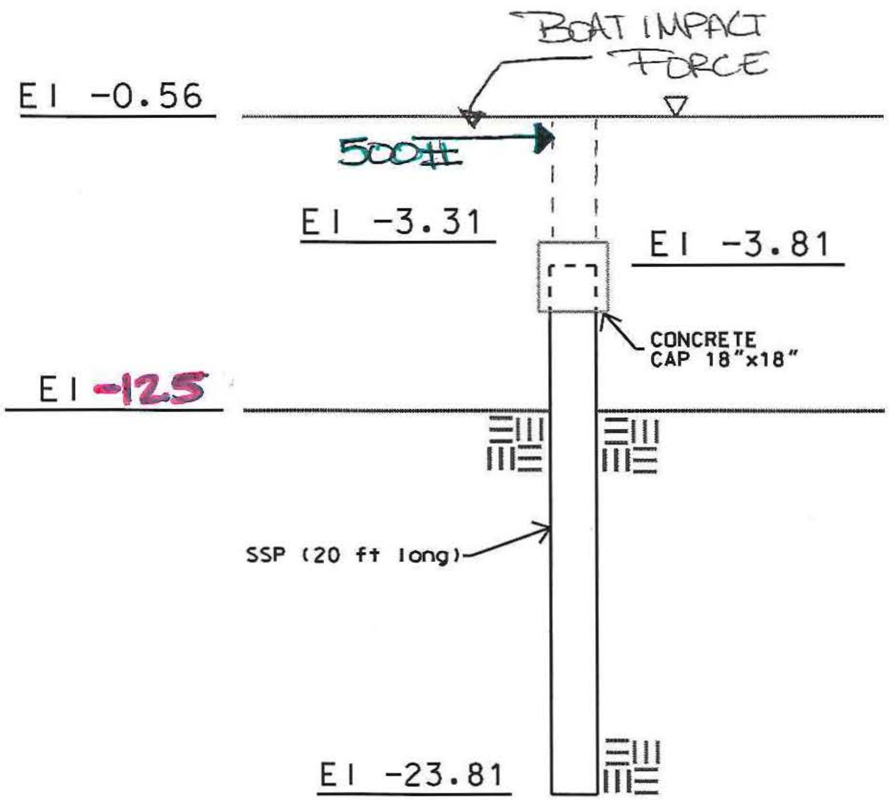
$$m = 3000 \text{ lbs}\left(\frac{0.031081 \text{ slug}}{1 \text{ lb}}\right) = 93.2 \frac{\text{lb}\cdot\text{s}^2}{\text{ft}}$$

$$v = 25 \text{ knots}\left(\frac{1.152 \text{ mph}}{1 \text{ knot}}\right)\left(\frac{1.466667 \text{ fps}}{1 \text{ mph}}\right) = 42.2 \text{ fps}$$

$$F = \frac{\frac{1}{2}\left(93.2 \frac{\text{lb}\cdot\cancel{\text{s}^2}}{\cancel{\text{ft}}}\right)\left(42.2 \frac{\cancel{\text{ft}}}{\cancel{\text{s}}}\right)^2}{200 \cancel{\text{ft}}} = 415.9 \text{ lbs}$$

USE 500 lbs

lateral force to be applied to the sheet pile as a result of boat impact



NOTES

1. SECTION SHOWN AT NOTCH. USE 20-FT SHEETS WITH CONCRETE CAP AS SHOWN OUTSIDE OF NOTCH.
2. SHEET PILE IS UNCOATED. SACRIFICIAL THICKNESS OF SHEET PILE SECTION IS NOT EXPECTED TO IMPACT STRUCTURAL ADEQUACY OVER SERVICE LIFE.
3. SSP OPTIONS FOR COST ESTIMATING:
   PZC 13
   PZ 22
   AZ 13-700

RIO ANTON RUIZ
SSP WEIR AT NOTCH

CAP Rio Anton Ruiz	Rip Rap Assumptions & Calcs	J. Serrano

Notes:

* No Flow analysis available, hence, no velocity provided for design.
* Riprap design based in previous stone protection recommended for same site and the requirement of stone protection to prevent scour in front of sheet pile wall as a result of plunging head differential caused by a SPF event. Scour protection requires a D_{50} ranging from 6-9 inches. (FROM H&H)
* Previous stone design had a customized gradation, R-20 was the closest standard gradation. R-20 average diameter ranges from 5.5-7 inches, therefore, doesn't meet D_{50} requirement for scour protection
* R-60 was selected as riprap stone for both banks and scour zone.

R-60 (Gradation in inches)

% Finer	Min	Max
100	12.0	13.6
50	8.0	10.0
15	5.5	8.0
0	0.0	5.5

Layer Thickness per EM 1601 (Page 3-4)

$1.5D_{50max}$ or $1.0D_{100max}$, whichever is greater

$1.5(10) = 15\text{ in} \Rightarrow \frac{15\text{ in}}{12\text{ in}} = 1.25\text{ ft} \Rightarrow$ $\boxed{\text{USE 1.5 ft for above water}}$

$1.0(13.6) = 13.6\text{ in}$

* For Riprap underwater add 50%

Then $1.25(1.5) = 1.88\text{ ft} \Rightarrow \boxed{\text{USE 2 ft}}$

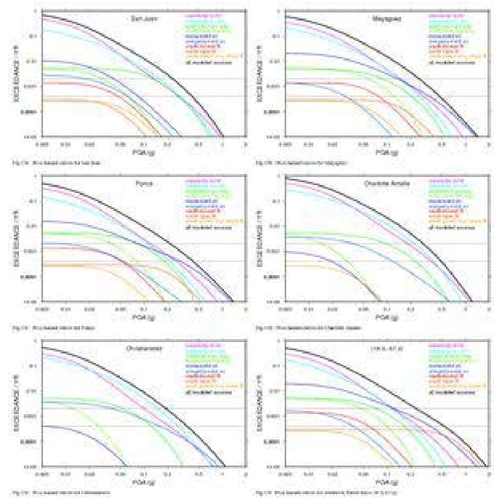
- Hazard Maps & Data
- Lower 48
- Alaska
- Hawaii
- Puerto Rico & U.S. Virgin Islands
- Guam & Marianas
- Samoa & Pacific Islands
- Urban & Regional
- Scenarios
- Time-Dependent EQ Probability
- Foreign

Maps

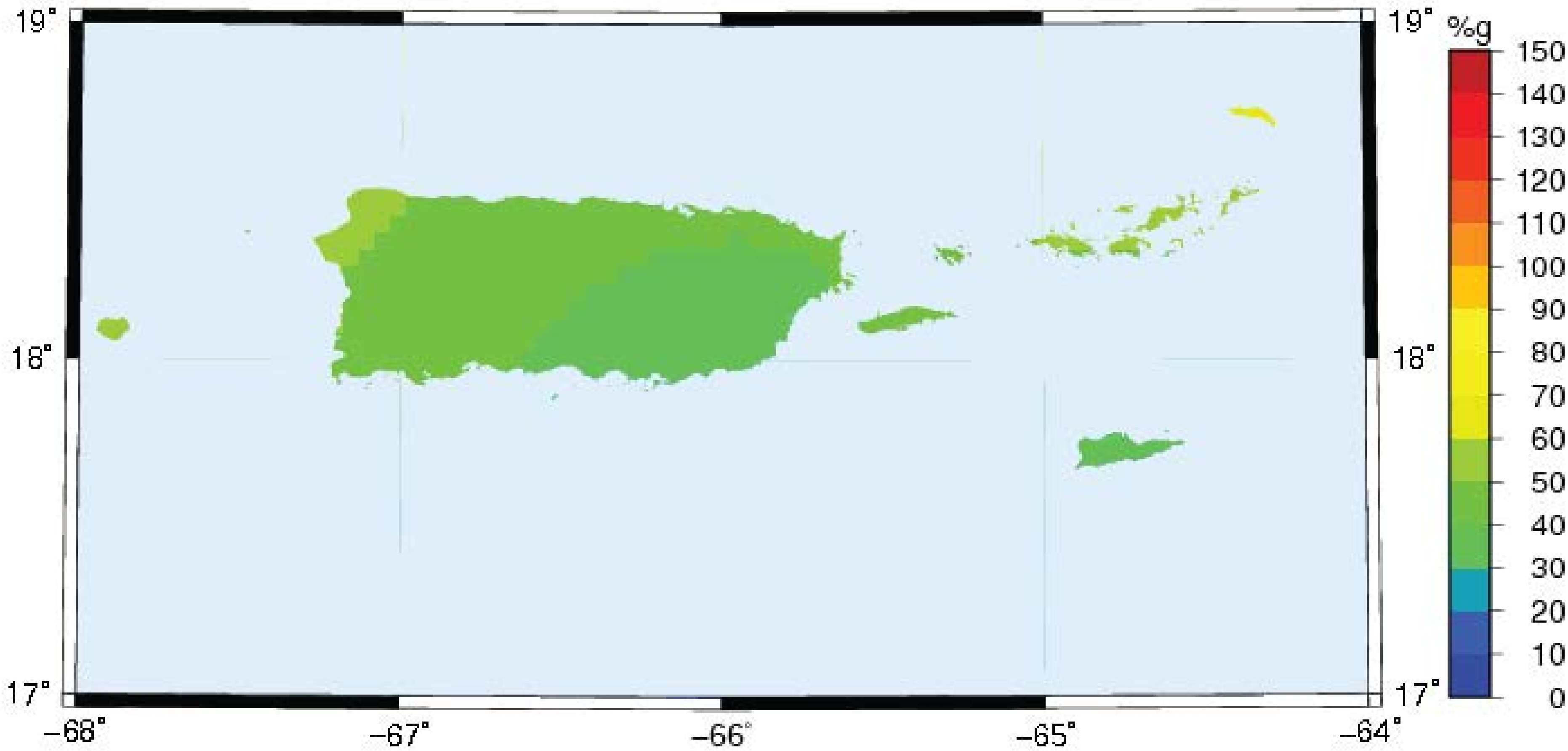
Mapped Ground Motion Hazard Values

Spectral Acceleration	Probability of Exceedance	File Format
0.2 second SA (5 Hz)	10% in 50 years	[PNG | PDF]
0.3 second SA (3.33 Hz)	10% in 50 years	[PNG | PDF]
1.0 second SA (1.0 Hz)	10% in 50 years	[PNG | PDF]
Peak Ground Acceleration	10% in 50 years	[PNG | PDF]
0.2 second SA (5 Hz)	2% in 50 years	[PNG | PDF]
0.3 second SA (3.33 Hz)	2% in 50 years	[PNG | PDF]
1.0 second SA (1.0 Hz)	2% in 50 years	[PNG | PDF]
Peak Ground Acceleration	2% in 50 years	[PNG | PDF]

Select Hazard Curves



PuertoRico



0.3 seconds SA, 10% in 50 years

Liquefaction Analysis using the Standard Penetration Test (SPT) 3-Aug-16
Computation of Safety Factor against Liquefaction using the method proposed by the Liquefaction Workshop, Youd, Idriss, et al. (2001)

READ THIS -----------> CELLS WITH RED LETTERS REQUIRE YOUR INPUT

Location: Rio Anton Ruiz Restoration Project

Design Earthquake Characteristics
MDE = **5.8** (Assumed from past regional quakes) Atmospheric Pressure (Pa) = 2090
phga = **0.35 g** (from USGS)
MSF = 1.93 Revised Idriss Scaling Factor (1995)

Boring No.: CB-AR-11

Elevation, top of hole: 3.57 MSL
Elevation, natural ground: 3.57 MSL
Elevation, groundwater: -1.4 MSL

Soil Profile 5 0.3761 0.1948168 0.37

Soil Type	Plasticity Index (PI)	Depth (ft)	Elevation (ft)	Total Unit Wt (pcf)	Water Table Factor	Unit Wt Water (pcf)	Total Stress (psf)	Pore Pressure (psf)	Effective Stress (psf)	N_{field}	C_N	Clean $(N_1)_{60}$	NON-LIQUEFIABLE? $((N_1)_{60}>30?)$[1]	$CRR_{7.5}$[2]	r_d[3]	CSR	K_σ[4]	Use K_σ	K_α[5]	FS	FS using $K\sigma$	Depth below natural ground
SM		3	0.57	110	0	64	330	0	330	2	1.62	3		0.06	0.99	0.2259	2.092	1	1	0.51	1.07	3.0
SM		6	-2.43	110	1	64	660	192	468	3	1.55	5		0.07	0.99	0.3163	1.820	1	1	0.42	0.77	6.0
SM		7.5	-3.93	115	1	64	833	288	545	4	1.51	6		0.08	0.98	0.3417	1.713	1	1	0.45	0.77	7.5
SM		10.5	-6.93	115	1	64	1178	480	698	5	1.43	7		0.09	0.98	0.3747	1.551	1	1	0.46	0.71	10.5
SM		15	-11.43	115	1	64	1695	768	927	5	1.34	7		0.09	0.97	0.4014	1.384	1	1	0.41	0.57	15.0
SM		18.5	-14.93	115	1	64	2098	992	1106	3	1.27	4		0.06	0.96	0.4130	1.290	1	1	0.30	0.38	18.5
SM		19.5	-15.93	115	1	64	2213	1056	1157	5	1.25	6		0.08	0.95	0.4154	1.267	1	1	0.38	0.48	19.5
SM		21	-17.43	115	1	64	2385	1152	1233	2	1.23	2		0.06	0.95	0.4185	1.235	1	1	0.25	0.31	21.0
SM		22.5	-18.93	115	1	64	2558	1248	1310	2	1.20	2		0.05	0.95	0.4210	1.206	1	1	0.25	0.30	22.5
SM		30	-26.43	110	1	64	3383	1728	1655	2	1.10	2		0.05	0.93	0.4325	1.098	1	1	0.24	0.26	30.0

Minimum FS: 0.24 0.26

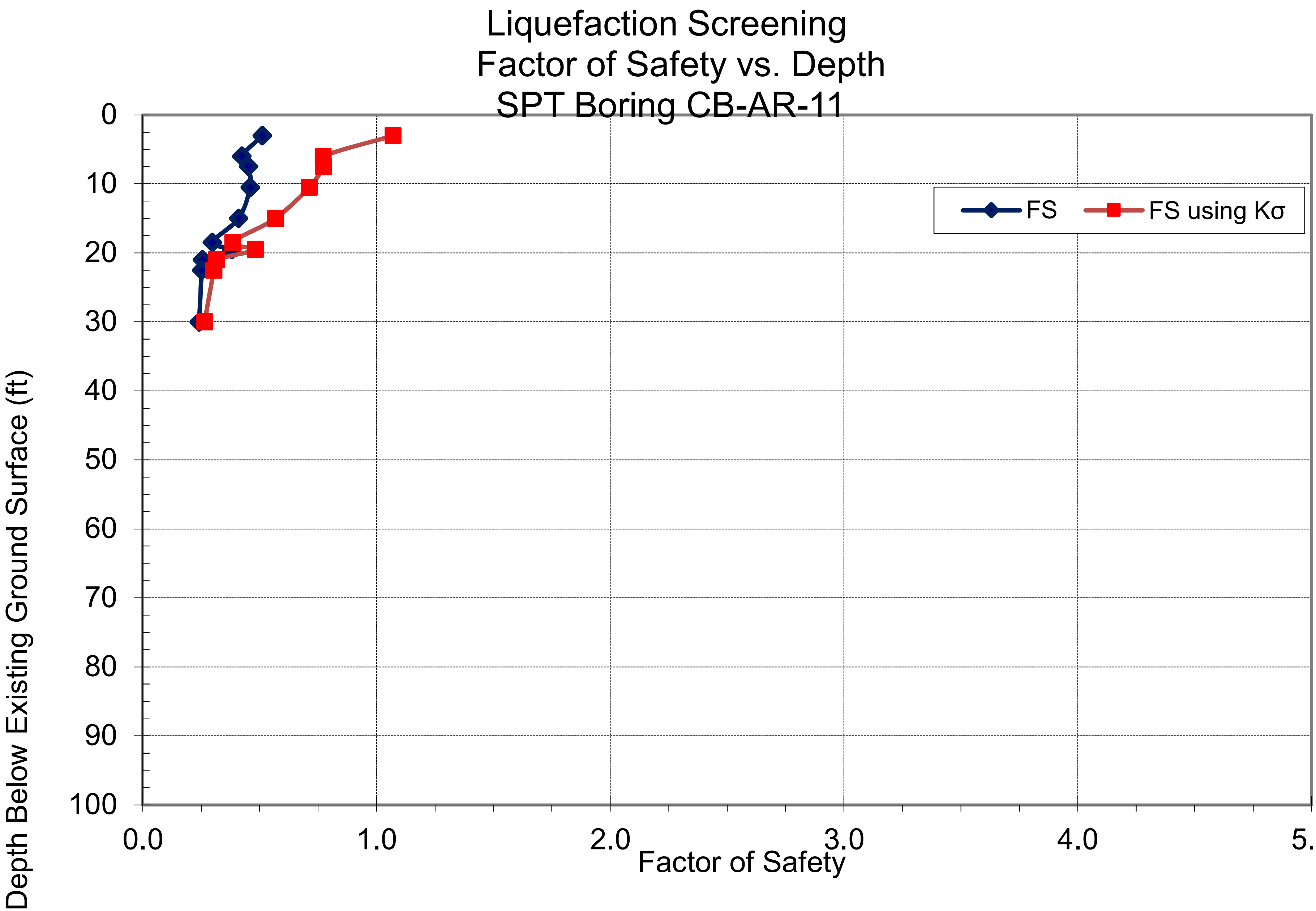
Notes:
1. Peat, sandstone, and soils with N1(60)>30 are considered non-liquefiable.
2. $CRR_{7.5}$ was determined assuming clean sand (conservative assumption)
3. Stress reduction coefficient (r_d): r_d formula, <30 feet below natural ground surface (: r_d=1-(0.00765*(depth*0.3048))
r_d formula, >30 feet below natural ground surface: r_d=1.174-(0.0267*(depth*0.3048)
4. Correction factor for high overburden stresses
5. Correction factor for static shear stress

OW=	Overwashed
CSR=	Cyclic Stress Ratio
CRR=	Cyclic Resistance Ratio
M=	MCE Magnitude
PHGA=	Peak Horizontal Ground surface Acceleration
g=	gravity
N_{field}	SPT blow counts measured in field
C_N	SPT correction factor
Clean $(N_1)_{60}$	
MSF	Magnitude Scaling factor
K_σ	Correction for overburden
K_α	Correction for shear stress
FS	Factor of Safety

References:

Acceler8, *Design Criteria Memorandum 6: Geotechnical Seismic Evaluation of CERP Dam Foundations*, 16 May 2005.

Yould, T.L. and Idriss, I.M., et al., *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils,* ASCE, Journal of Geotechnical and Geoenvironmental Engineer, October 2001.



Areas in the graph plotted with points or lines are those that were initially assesed as liquefiable soils. Areas in white represent areas of soils that met the criteria for non-liquefiable soils.

Liquefaction Induced Settlement

Soil Type	Depth (ft)	Thickness (ft)	N_{field}	Clean $(N_1)_{60}$	CSR	ε_v (%)	ΔH	
SM	3	3	2	3	0.23	5.0	0.15	
SM	6	1.5	3	5	0.32	4.0	0.06	
SM	7.5	3	4	6	0.34	3.5	0.11	
SM	10.5	4.5	5	7	0.37	3.0	0.14	
SM	15	3.5	5	7	0.40	3.0	0.11	
SM	18.5	1	3	4	0.41	4.5	0.05	
SM	19.5	1.5	5	6	0.42	3.5	0.05	
SM	21	1.5	2	2	0.42	7.5	0.11	
SM	22.5	7.5	2	2	0.42	7.5	0.56	
SM	30		2	2	0.43	**Total**	**1.33**	**feet**
							15.93	**inches**

Sorce: Geotechnical Earthquake Engineering by Steven L. Kramer, Sec 9.6 Effects of Liquefaction, 9.6.3.2 Settlement of Saturated Sands, Figure 9.53 page 405

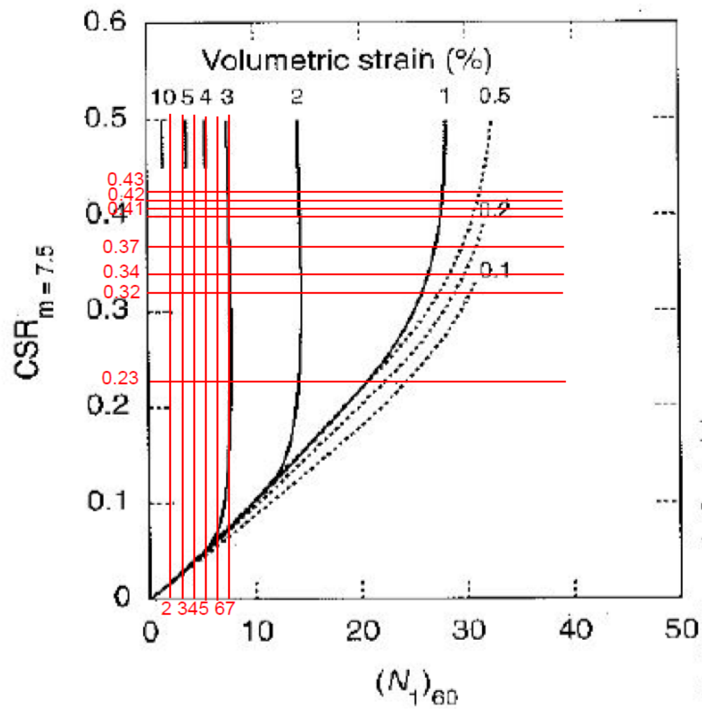


Figure 9.53 Chart for estimation of volumetric strain in saturated sands from cyclic stress ratio and standard penetration resistance. (After Tokimatsu and Seed, 1987. Evaluation of settlements in sand due to earthquake shaking, *Journal of Geotechnical Engineering*, Vol. 113, No. 8. Reprinted by permission of ASCE.)

Attachment C

Structural Plates and Analyses

ATTACHMENT C

STRUCTURAL CALCULATIONS

D1 – SSP WEIR

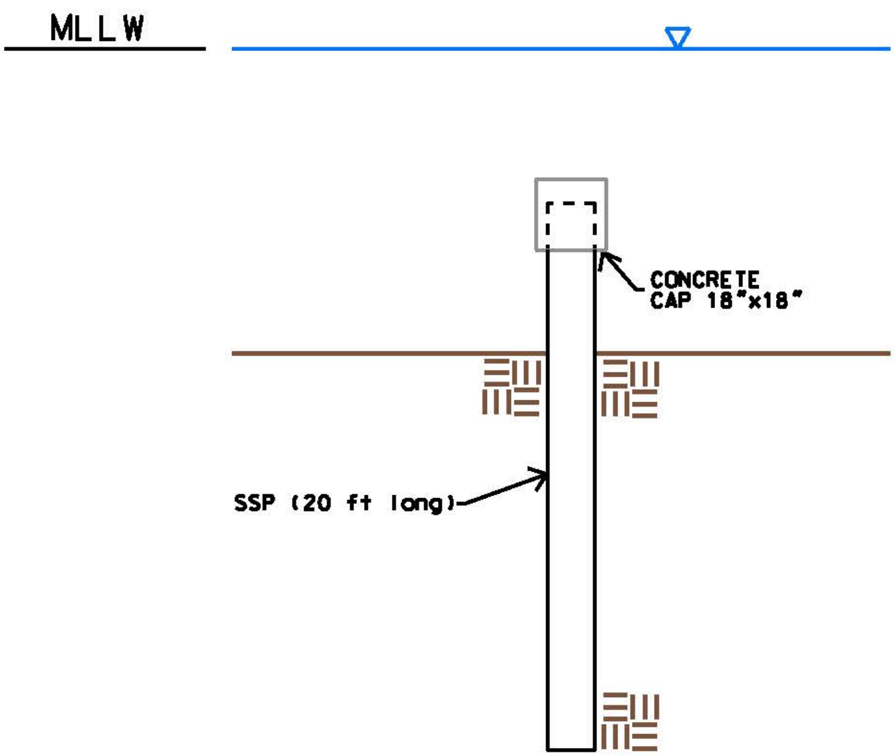
D1.1 – WEIR DESIGN
D1.2 – CWALSHT OUTPUT
D1.3 – SUPPLEMENTAL INFORMATION

US Army Corps
of Engineers®
Jacksonville District

ATTACHMENT C

STRUCTURAL CALCULATIONS

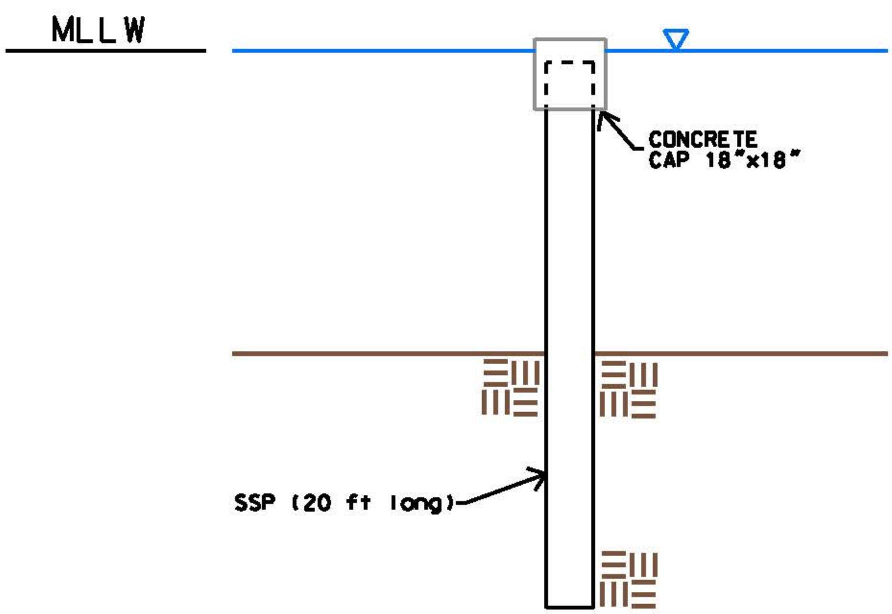
D1.1 – WEIR DESIGN



NOTES

1. SECTION SHOWN AT NOTCH. USE 20-FT SHEETS WITH CONCRETE CAP AS SHOWN.
2. SHEET PILE MAY BE LEFT UNCOATED. ASSUME MARINE GRADE ASTM A690 MARINE GRADE STEEL.
3. SSP OPTIONS FOR COST ESTIMATING:
   - PZC 13
   - PZ 22
   - AZ 13-700

RIO ANTON RUIZ
SSP WEIR AT NOTCH



NOTES

1. SECTION SHOWN OUTSIDE NOTCH. USE 20-FT SHEETS WITH CONCRETE CAP AS SHOWN
2. SHEET PILE MAY BE LEFT UNCOATED. ASSUME MARINE GRADE ASTM A690 MARINE GRADE STEEL.
3. SSP OPTIONS FOR COST ESTIMATING:
   - PZC 13
   - PZ 22
   - AZ 13-700

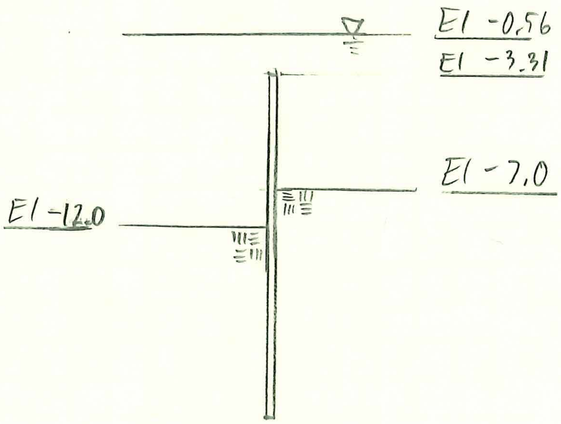
RIO ANTON RUIZ SSP
WEIR OUTSIDE NOTCH

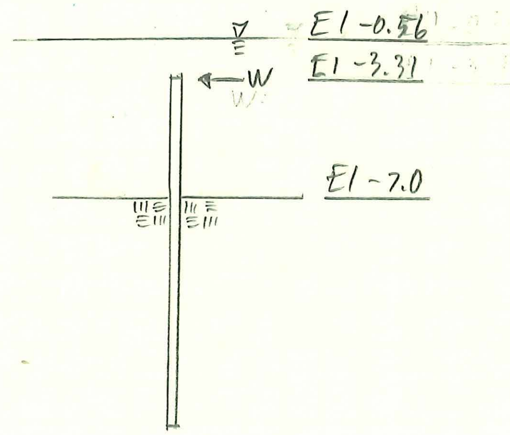
Sheet Pile Weir

checked by A. Hendricks

Sheet Pile Design

Simplified Load Cases





Load Case 1
Scour Condition
Usual; FS 1.5

Load Case 2
Boat Impact
Usual; FS 1.5
w = 0.75 kips / ft (see 2/3)

CWALSHT Results (Stability)

	Load Case 1		Load Case 2
Tip El	-19.06	Tip El	-17.91
Mmax	1,079 ft-lbs	Mmax	4,960 ft-lbs
Defl	1.10E8 lb·in³	Defl	5.17E8 lb·in³

Max sheet Length Required

-19.06 ft - 3.31 ft = 15.75 ft → use 20-foot long sheets

S_x required (A572 Gr 50)

$$S_x = \frac{M_{max}}{f_b} = \frac{4{,}960 \text{ ft}\cdot\text{lbs} \times \left(\frac{12 \text{ in}}{1 \text{ ft}}\right)}{(50{,}000 \text{ psi} / 2)} = 2.4 \text{ in}^3$$

Use PZC 13

$S_x = 24.2 \text{ in}^3 >> 2.4 \text{ in}^3$

checked by A. Hendricks

Sheet Pile Design

Load due to Small Watercraft

Assumptions

Watercraft length: 20 ft

Approach Velocity: 10 ft/s ≈ 6 knots

Impact at top of weir

Impact is spread over 2-foot width

Watercraft Weight

$W = 12 L^2$ (UFC 4-152-07; 6-3.5.1.1)

$= 12\,(20\text{ ft})^2$

$= 4{,}800\text{ lbs}$

Watercraft Impact Force

$v = 10\text{ ft/s}$ $\quad a = \frac{v}{1.0\text{ s}} = 10\text{ ft/s}^2$

$g = 32.2\text{ ft/s}^2$

$F = \frac{W a}{g}$

$= \frac{4{,}800\text{ lbs} \times 10\text{ ft/s}^2}{32.2\text{ ft/s}^2}$

$= 1.5\text{ kips}$

Line Load on Sheet Pile

$w = \frac{F}{2} = 0.750\text{ kips/ft} = 750\text{ lbs/ft}$

checked by A. Hendricks

Sheet Pile Design

Corrosion of Sheet Pile

See attached LBFoster information sheets

For 50-year design life (sea water; temperate; submerged)

1.75 mm loss of thickness

$1.75\ mm \times 2\ sides = 3.5\ mm \approx 0.138\ in$

round 0.138 in up to 0.1875 in in thickness reduction table

$S_x = 14.76\ in^3 \gg 2.4\ in^3$

Sheet may be left uncoated. Section loss is not expected to compromise structural adequacy.

ATTACHMENT C

STRUCTURAL CALCULATIONS

D1.2 – CWALSHT OUTPUT

```
 PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                         BY CLASSICAL METHODS
 DATE: 19-JULY-2016                                        TIME: 13:24:01

                           ****************
                           *  INPUT DATA  *
                           ****************

       I.--HEADING
      'RIO ANTON RUIZ SSP WEIR
      'LOAD CASE 1: SCOUR CONDITION
      'USUAL; FS = 1.5; Q-CASE

      II.--CONTROL
         CANTILEVER WALL DESIGN
         FACTOR OF SAFETY FOR ACTIVE PRESSURES  = 1.00
         FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

     III.--WALL DATA
         ELEVATION AT TOP OF WALL    = -3.31 FT.

      IV.--SURFACE POINT DATA

           IV.A.--RIGHTSIDE
              DIST. FROM     ELEVATION
              WALL (FT)         (FT)
                   0.00        -7.00
                  50.00        -7.00

           IV.B.--LEFTSIDE
              DIST. FROM     ELEVATION
              WALL (FT)         (FT)
                   0.00       -12.00
                  50.00       -12.00

       V.--SOIL LAYER DATA

          V.A.--RIGHTSIDE
              LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
              LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

                ANGLE OF          ANGLE OF                          <-SAFETY->
  SAT.    MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.   WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)   (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
110.00  105.00    28.00    0.00     0.00    0.00                  DEF  DEF

          V.B.--LEFTSIDE
              LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE  = DEFAULT
              LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

                ANGLE OF          ANGLE OF                          <-SAFETY->
  SAT.    MOIST  INTERNAL  COH-     WALL    ADH-    <--BOTTOM-->  <-FACTOR->
 WGHT.   WGHT.  FRICTION  ESION  FRICTION  ESION   ELEV.  SLOPE  ACT. PASS.
 (PCF)   (PCF)    (DEG)   (PSF)    (DEG)   (PSF)   (FT)  (FT/FT)
110.00  105.00    28.00    0.00     0.00    0.00                  DEF  DEF

      VI.--WATER DATA
         UNIT WEIGHT          = 62.50 (PCF)
         RIGHTSIDE ELEVATION = -3.31 (FT)
         LEFTSIDE ELEVATION  = -3.31 (FT)
         NO SEEPAGE
```

VII.--VERTICAL SURCHARGE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 19-JULY-2016 TIME: 13:24:04

****
* SOIL PRESSURES FOR *
* CANTILEVER WALL DESIGN *
****

I.--HEADING
'RIO ANTON RUIZ SSP WEIR
'LOAD CASE 1: SCOUR CONDITION
'USUAL; FS = 1.5; Q-CASE

II.--SOIL PRESSURES

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

	NET	<---LEFTSIDE--->		<------NET------> (SOIL + WATER)		<--RIGHTSIDE--->	
ELEV. (FT)	WATER (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
-3.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
-4.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
-5.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
-6.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
-7.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
-7.3	0.0	0.0	0.0	5.3	29.5	5.3	29.5
-8.3	0.0	0.0	0.0	22.5	124.7	22.5	124.7
-9.3	0.0	0.0	0.0	39.6	219.8	39.6	219.8
-10.3	0.0	0.0	0.0	56.8	315.0	56.8	315.0
-11.3	0.0	0.0	0.0	73.9	410.2	73.9	410.2
-12.0	0.0	0.0	0.0	85.7	475.8	85.7	475.8
-12.3	0.0	29.5	5.3	61.6	500.0	91.1	505.3
-13.0	0.0	95.2	17.1	7.7	553.9	102.9	571.0
-13.1	0.0	104.6	18.8	0.0	561.6	104.6	580.4
-13.3	0.0	124.7	22.5	-16.5	578.0	108.2	600.5
-14.3	0.0	219.8	39.6	-94.5	656.1	125.4	695.7
-15.3	0.0	315.0	56.8	-172.5	734.1	142.5	790.8
-16.3	0.0	410.2	73.9	-250.5	812.1	159.7	886.0
-17.3	0.0	505.3	91.1	-328.5	890.1	176.8	981.2
-18.3	0.0	600.5	108.2	-406.5	968.1	194.0	1076.3
-19.3	0.0	695.7	125.4	-484.6	1046.2	211.1	1171.5
-20.3	0.0	790.8	142.5	-562.6	1124.2	228.3	1266.7
-21.3	0.0	886.0	159.7	-640.6	1202.2	245.4	1361.8
-22.3	0.0	981.2	176.8	-718.6	1280.2	262.6	1457.0

-23.3	0.0	1076.3	194.0	-796.6	1358.2	279.7	1552.2
-24.3	0.0	1171.5	211.1	-874.7	1436.2	296.9	1647.3
-25.3	0.0	1266.7	228.3	-952.7	1514.3	314.0	1742.5
-26.3	0.0	1361.8	245.4	-1030.7	1592.3	331.1	1837.7
-27.3	0.0	1457.0	262.6	-1108.7	1670.3	348.3	1932.8
-28.3	0.0	1552.2	279.7	-1186.7	1748.3	365.4	2028.0
-29.3	0.0	1647.3	296.9	-1264.8	1826.3	382.6	2123.2
-30.3	0.0	1742.5	314.0	-1342.8	1904.4	399.7	2218.4
-31.3	0.0	1837.7	331.1	-1420.8	1982.4	416.9	2313.5
-32.3	0.0	1932.8	348.3	-1498.8	2060.4	434.0	2408.7
-33.3	0.0	2028.0	365.4	-1576.8	2138.4	451.2	2503.9
-34.3	0.0	2123.2	382.6	-1654.8	2216.4	468.3	2599.0
-35.3	0.0	2218.4	399.7	-1732.9	2294.4	485.5	2694.2
-36.3	0.0	2313.5	416.9	-1810.9	2372.5	502.6	2789.4
-37.3	0.0	2408.7	434.0	-1888.9	2450.5	519.8	2884.5
-38.3	0.0	2503.9	451.2	-1966.9	2528.5	536.9	2979.7
-39.3	0.0	2599.0	468.3	-2044.9	2606.5	554.1	3074.9
-40.3	0.0	2694.2	485.5	-2123.0	2684.5	571.2	3170.0
-41.3	0.0	2789.4	502.6	-2201.0	2762.6	588.4	3265.2
-42.3	0.0	2884.5	519.8	-2279.0	2840.6	605.5	3360.4

```
PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                          BY CLASSICAL METHODS
DATE: 19-JULY-2016                                          TIME: 13:24:04

                       ****************************
                       *  SUMMARY OF RESULTS FOR  *
                       *  CANTILEVER WALL DESIGN  *
                       ****************************

    I.--HEADING
     'RIO ANTON RUIZ SSP WEIR
     'LOAD CASE 1: SCOUR CONDITION
     'USUAL; FS = 1.5; Q-CASE

   II.--SUMMARY

        RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

        LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

   WALL BOTTOM ELEV. (FT)     :      -19.06
         PENETRATION (FT)     :        7.06

   MAX. BEND. MOMENT (LB-FT)  :  1.0787E+03
         AT ELEVATION (FT)    :      -15.69

   MAX. SCALED DEFL. (LB-IN^3):  1.1003E+08
         AT ELEVATION (FT)    :       -3.31

             NOTE:  DIVIDE SCALED DEFLECTION MODULUS OF
                    ELLASTICITY IN PSI TIMES PILE MOMENT
                    OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
                    IN INCHES.
```

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHOREDOR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 19-JULY-2016 TIME: 13:24:04

****
* COMPLETE OF RESULTS FOR *
* CANTILEVER WALL DESIGN *
****

I.--HEADING
'RIO ANTON RUIZ SSP WEIR
'LOAD CASE 1: SCOUR CONDITION
'USUAL; FS = 1.5; Q-CASE

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
-3.31	0.0000E+00	0.	1.1003E+08	0.00
-4.31	7.2298E-12	0.	1.0056E+08	0.00
-5.31	-1.5643E-10	0.	9.1094E+07	0.00
-6.31	-3.4379E-10	0.	8.1627E+07	0.00
-7.00	-4.0431E-10	0.	7.5095E+07	0.00
-7.31	8.5148E-02	1.	7.2160E+07	5.32
-8.31	6.4255E+00	15.	6.2694E+07	22.47
-9.31	3.5231E+01	46.	5.3243E+07	39.61
-10.31	1.0365E+02	94.	4.3858E+07	56.76
-11.31	2.2883E+02	159.	3.4660E+07	73.91
-12.00	3.5727E+02	214.	2.8532E+07	85.75
-12.31	4.2746E+02	237.	2.5868E+07	61.56
-13.00	6.0151E+02	261.	2.0213E+07	7.73
-13.10	6.2739E+02	261.	1.9438E+07	0.00
-13.31	6.8243E+02	260.	1.7824E+07	-16.46
-14.31	9.2095E+02	204.	1.0957E+07	-94.48
-15.31	1.0650E+03	71.	5.6670E+06	-172.50
-16.31	1.0365E+03	-141.	2.1928E+06	-250.51
-17.31	7.5755E+02	-430.	4.7364E+05	-328.53
-17.55	6.4464E+02	-511.	2.7834E+05	-347.26
-18.31	2.2248E+02	-512.	1.9849E+04	345.89
-19.06	0.0000E+00	0.	0.0000E+00	1026.34

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF
ELLASTICITY IN PSI TIMES PILE MOMENT
OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
IN INCHES.

III.--WATER AND SOIL PRESSURES

		<-------------SOIL PRESSURES-------------->			
		<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	WATER PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
-3.31	0.	0.	0.	0.	0.
-4.31	0.	0.	0.	0.	0.
-5.31	0.	0.	0.	0.	0.
-6.31	0.	0.	0.	0.	0.
-7.00	0.	0.	0.	0.	0.

-7.31	0.	0.	0.	5.	30.
-8.31	0.	0.	0.	22.	125.
-9.31	0.	0.	0.	40.	220.
-10.31	0.	0.	0.	57.	315.
-11.31	0.	0.	0.	74.	410.
-12.00	0.	0.	0.	86.	476.
-12.31	0.	30.	5.	91.	505.
-13.00	0.	95.	17.	103.	571.
-13.10	0.	105.	19.	105.	580.
-13.31	0.	125.	22.	108.	601.
-14.31	0.	220.	40.	125.	696.
-15.31	0.	315.	57.	143.	791.
-16.31	0.	410.	74.	160.	886.
-17.31	0.	505.	91.	177.	981.
-17.55	0.	528.	95.	181.	1004.
-18.31	0.	601.	108.	194.	1076.
-19.06	0.	696.	125.	211.	1172.
-20.31	0.	791.	143.	228.	1267.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 10-JUNE-2016 TIME: 9:24:57

****
* INPUT DATA *
****

I.--HEADING
'Rio Anton Ruiz SSP Weir
'Load Case 2: Boat Impact
'Usual; FS = 1.5; Q-Case

II.--CONTROL
CANTILEVER WALL DESIGN
FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

III.--WALL DATA
ELEVATION AT TOP OF WALL = -3.31 FT.

IV.--SURFACE POINT DATA

IV.A.--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
0.00	-7.00
50.00	-7.00

IV.B.--LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
0.00	-7.00
50.00	-7.00

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
110.00	105.00	28.00	0.00	0.00	0.00			DEF	DEF

V.B.--LEFTSIDE
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE = DEFAULT
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = DEFAULT

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
110.00	105.00	28.00	0.00	0.00	0.00			DEF	DEF

VI.--WATER DATA
UNIT WEIGHT = 62.50 (PCF)
RIGHTSIDE ELEVATION = -3.31 (FT)
LEFTSIDE ELEVATION = -3.31 (FT)
NO SEEPAGE

VII.--VERTICAL SURCHARGE LOADS

NONE

VIII.--HORIZONTAL LOADS

VIII.A.--HORIZONTAL LINE LOADS

ELEVATION (FT)	LINE LOAD (PLF)
-3.31	750.00

VIII.B.--HORIZONTAL DISTRIBUTED LOADS

NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 10-JUNE-2016 TIME: 9:25:10

****
* SOIL PRESSURES FOR *
* CANTILEVER WALL DESIGN *
****

I.--HEADING

'Rio Anton Ruiz SSP Weir
'Load Case 2: Boat Impact
'Usual; FS = 1.5; Q-Case

II.--SOIL PRESSURES

RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

		<---LEFTSIDE--->		<------NET------> (SOIL + WATER)		<--RIGHTSIDE--->	
ELEV. (FT)	NET WATER (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
-3.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
-4.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
-5.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
-6.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
-7.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
-7.3	0.0	29.5	5.3	-24.2	24.2	5.3	29.5
-8.0	0.0	95.2	17.1	-78.0	78.0	17.1	95.2
-8.3	0.0	124.7	22.5	-102.2	102.2	22.5	124.7
-9.3	0.0	219.8	39.6	-180.2	180.2	39.6	219.8
-10.3	0.0	315.0	56.8	-258.2	258.2	56.8	315.0
-11.3	0.0	410.2	73.9	-336.3	336.3	73.9	410.2
-12.3	0.0	505.3	91.1	-414.3	414.3	91.1	505.3
-13.3	0.0	600.5	108.2	-492.3	492.3	108.2	600.5
-14.3	0.0	695.7	125.4	-570.3	570.3	125.4	695.7
-15.3	0.0	790.8	142.5	-648.3	648.3	142.5	790.8
-16.3	0.0	886.0	159.7	-726.4	726.4	159.7	886.0
-17.3	0.0	981.2	176.8	-804.4	804.4	176.8	981.2

-18.3	0.0	1076.3	194.0	-882.4	882.4	194.0	1076.3
-19.3	0.0	1171.5	211.1	-960.4	960.4	211.1	1171.5
-20.3	0.0	1266.7	228.3	-1038.4	1038.4	228.3	1266.7
-21.3	0.0	1361.8	245.4	-1116.4	1116.4	245.4	1361.8
-22.3	0.0	1457.0	262.6	-1194.5	1194.5	262.6	1457.0
-23.3	0.0	1552.2	279.7	-1272.5	1272.5	279.7	1552.2
-24.3	0.0	1647.3	296.9	-1350.5	1350.5	296.9	1647.3
-25.3	0.0	1742.5	314.0	-1428.5	1428.5	314.0	1742.5
-26.3	0.0	1837.7	331.1	-1506.5	1506.5	331.1	1837.7
-27.3	0.0	1932.8	348.3	-1584.6	1584.6	348.3	1932.8
-28.3	0.0	2028.0	365.4	-1662.6	1662.6	365.4	2028.0
-29.3	0.0	2123.2	382.6	-1740.6	1740.6	382.6	2123.2
-30.3	0.0	2218.4	399.7	-1818.6	1818.6	399.7	2218.4
-31.3	0.0	2313.5	416.9	-1896.6	1896.6	416.9	2313.5
-32.3	0.0	2408.7	434.0	-1974.6	1974.6	434.0	2408.7
-33.3	0.0	2503.9	451.2	-2052.7	2052.7	451.2	2503.9
-34.3	0.0	2599.0	468.3	-2130.7	2130.7	468.3	2599.0
-35.3	0.0	2694.2	485.5	-2208.7	2208.7	485.5	2694.2
-36.3	0.0	2789.4	502.6	-2286.7	2286.7	502.6	2789.4
-37.3	0.0	2884.5	519.8	-2364.7	2364.7	519.8	2884.5

```
PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                          BY CLASSICAL METHODS
DATE: 10-JUNE-2016                                          TIME: 9:25:11

                       ****************************
                       *  SUMMARY OF RESULTS FOR  *
                       *  CANTILEVER WALL DESIGN  *
                       ****************************

    I.--HEADING
     'Rio Anton Ruiz SSP Weir
     'Load Case 2: Boat Impact
     'Usual; FS = 1.5; Q-Case

   II.--SUMMARY

        RIGHTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

        LEFTSIDE SOIL PRESSURES DETERMINED BY SWEEP SEARCH WEDGE METHOD.

   WALL BOTTOM ELEV. (FT)     :      -17.91
         PENETRATION (FT)     :       10.91

   MAX. BEND. MOMENT (LB-FT)  :  4.9599E+03
         AT ELEVATION (FT)    :      -11.38

   MAX. SCALED DEFL. (LB-IN^3):  5.1667E+08
         AT ELEVATION (FT)    :       -3.31

             NOTE:  DIVIDE SCALED DEFLECTION MODULUS OF
                    ELLASTICITY IN PSI TIMES PILE MOMENT
                    OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
                    IN INCHES.
```

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHOREDOR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 10-JUNE-2016 TIME: 9:25:11

****
* COMPLETE OF RESULTS FOR *
* CANTILEVER WALL DESIGN *
****

I.--HEADING
'Rio Anton Ruiz SSP Weir
'Load Case 2: Boat Impact
'Usual; FS = 1.5; Q-Case

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN^3)	NET PRESSURE (PSF)
-3.31	0.0000E+00	750.	5.1667E+08	0.00
-4.31	7.5000E+02	750.	4.4661E+08	0.00
-5.31	1.5000E+03	750.	3.7785E+08	0.00
-6.31	2.2500E+03	750.	3.1169E+08	0.00
-7.00	2.7675E+03	750.	2.6822E+08	0.00
-7.31	2.9996E+03	746.	2.4941E+08	-24.19
-8.00	3.5045E+03	711.	2.0938E+08	-78.02
-8.31	3.7208E+03	683.	1.9231E+08	-102.20
-9.31	4.3397E+03	542.	1.4162E+08	-180.22
-10.31	4.7784E+03	323.	9.8410E+07	-258.24
-11.31	4.9589E+03	25.	6.3418E+07	-336.26
-12.31	4.8032E+03	-350.	3.6946E+07	-414.28
-13.31	4.2331E+03	-803.	1.8714E+07	-492.30
-13.34	4.2120E+03	-816.	1.8342E+07	-494.33
-14.31	3.2281E+03	-1158.	7.7317E+06	-207.84
-15.31	2.0152E+03	-1219.	2.2971E+06	86.33
-16.31	8.8864E+02	-985.	3.5717E+05	380.51
-17.31	1.4260E+02	-458.	7.6092E+03	674.68
-17.91	0.0000E+00	0.	0.0000E+00	851.18

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF
ELLASTICITY IN PSI TIMES PILE MOMENT
OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
IN INCHES.

III.--WATER AND SOIL PRESSURES

		<-------------SOIL PRESSURES-------------->			
		<----LEFTSIDE----->		<---RIGHTSIDE---->	
ELEVATION (FT)	WATER PRESSURE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
-3.31	0.	0.	0.	0.	0.
-4.31	0.	0.	0.	0.	0.
-5.31	0.	0.	0.	0.	0.
-6.31	0.	0.	0.	0.	0.
-7.00	0.	0.	0.	0.	0.
-7.31	0.	30.	5.	5.	30.
-8.00	0.	95.	17.	17.	95.
-8.31	0.	125.	22.	22.	125.

-9.31	0.	220.	40.	40.	220.
-10.31	0.	315.	57.	57.	315.
-11.31	0.	410.	74.	74.	410.
-12.31	0.	505.	91.	91.	505.
-13.31	0.	601.	108.	108.	601.
-13.34	0.	603.	109.	109.	603.
-14.31	0.	696.	125.	125.	696.
-15.31	0.	791.	143.	143.	791.
-16.31	0.	886.	160.	160.	886.
-17.31	0.	981.	177.	177.	981.
-17.91	0.	1076.	194.	194.	1076.
-19.31	0.	1172.	211.	211.	1172.

ATTACHMENT C

STRUCTURAL CALCULATIONS

D1.3 – SUPPLEMENTAL INFORMATION

The following soil parameters are provided for the design of a steel sheet pile wall. The short-term, long-term and seismic conditions should all be analyzed, and the most critical condition used for design purposes. These parameters are based on boring log CB-AR-11, laboratory testing results and typical soil data from the area available through USDA web soil survey and per Table 3-1 from EM 1110-2-2504.

RIO ANTON RUIZ RESTORATION PROJECT SOIL PARAMETERS														
Elevation (NAVD 88)			Depth (ft)			USCS	γ_{sat} (pcf)	γ' (pcf)	Undrained (Q)		Drained (S)		Seismic (0.80R)	
									ϕ (deg.)	c (psf)	ϕ' (deg.)	c' (psf)	ϕ' (deg.)	c' (psf)
3.5	to	-2.5	0	to	-6	FILL SM	110	105	28	0	28	0	26	0
-2.5	to	-19	-6	to	-22.5	SM	115	110	30	0	30	0	28	0
-19	to	-30	-22.5	to	-33.5	SM	110	105	29	0	29	0	25	0

GERDAU AMERISTEEL

Eurocode 3:

Design of Steel Structures

Part 5: Piling

(ENV 1993-5)

Loss of Thickness

(mm)

LBFoster Piling

TABLE 1. Loss of Thickness Due to Corrosion for Steel Sheet Pilings (Ref 4) [A]

Soil, with or without groundwater:	DESIGN LIFE: 5 years	25 years	50 years	75 years	100 years
Undisturbed natural soils	0.00 mm	0.30 mm	0.60 mm	0.90 mm	1.20 mm
Polluted natural soils and industrial grounds	0.15 mm	0.75 mm	1.50 mm	2.25 mm	3.00 mm
Aggressive natural soils (swamp, marsh, peat...)	0.20 mm	1.00 mm	1.75 mm	2.50 mm	3.25 mm
Non-compacted and non-aggressive fills [B] (clay, schist, sand, silt...)	0.18 mm	0.70 mm	1.20 mm	1.70 mm	2.20 mm
Non-compacted and aggressive fills [B] (ashes, slag...)	0.50 mm	2.00 mm	3.25 mm	4.50 mm	5.75 mm
Water [C]:					
Common fresh water (river, ship canal-,...) in the zone of high attack (water line)	0.15 mm	0.55 mm	0.90 mm	1.15 mm	1.40 mm
Very polluted fresh water (sewage, industrial effluent-,...) in the zone of high attack (water line)	0.30 mm	1.30 mm	2.30 mm	3.30 mm	4.30 mm
Sea water in temperate climate in the zone of high attack (low water and splash zones)	0.55 mm	1.90 mm	3.75 mm	5.60 mm	7.50 mm
Sea water in temperate climate in the submerged zone or tidal zone	0.25 mm	0.90 mm	1.75 mm	2.60 mm	3.50 mm

A. Values are provided for general guidance only. Local knowledge may lead to the use of other values for design. The values given for 5 and 25 years are based on measurements, whereas other values are extrapolated.

B. In compacted fills, these corrosion losses should be divided by two.

C. The highest corrosion rate is usually found at the splash zone of marine environments or at the low water level in tidal waters. However, in most cases, the highest bending stresses occur in the submerged zone.

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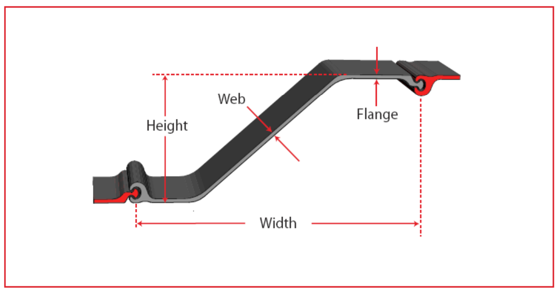

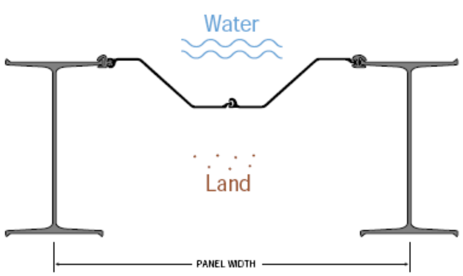

LBFoster Piling

Summary of Calculated Section Modulus and Moment of Inertia for Thickness Reduction from 0.000" – 0.250"

Thickness Reduction (in.)	Section Modulus (in^3 / ft)				Moment of Inertia (in^4 / ft)			
	PZ27	PZC13	PZC18	PZC26	PZ27	PZC13	PZC18	PZC26
0.0000	31.80	24.17	33.50	48.38	187.3	151.9	255.5	428.1
0.0625	27.96	21.10	29.25	43.74	168.28	131.75	222.12	385.73
0.1250	24.07	17.96	24.89	39.08	144.12	111.79	188.23	343.42
0.1875	20.10	14.76	20.49	34.41	119.72	91.31	154.32	301.3
0.2500	16.10	11.49	16.05	29.74	95.39	70.72	120.38	259.48

* Reference: Richard Hartman, Ph.D., P.E.