Geotechnical Feasibility Appendix

Section 14 – Emergency Shoreline Protection

East Hagåtña, Guam

Alaska District, Pacific Ocean Division

05 August 2025
Status: ATR Backcheck
Submittal

US Army Corps of Engineers
Alaska District

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS, ALASKA DISTRICT
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CEPOA-EC-G-GM 05 August 2025

MEMORANDUM FOR

Civil Works Project Management (CEPOH-PPC), Nickolas Emilio

SUBJECT: Geotechnical Feasibility Report for the Section 14 Emergency Shoreline Protection, East Hagåtña, Guam.

1. Enclosed is the Geotechnical Feasibility Report for the Section 14 Emergency Shoreline Protection in East Hagåtña, Guam. Included with this report are a discussion of existing geotechnical information pertaining to the project and preliminary geotechnical analysis and recommendations.

2. Questions should be addressed to Justin Miller at 907-753-2577 or Amy Steiner at 907-753-2800.

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1 Introduction

The purpose of this report is to document the anticipated subsurface geotechnical conditions, provide analyses of anticipated site conditions as they pertain to the project described herein, and to introduce a preliminary geotechnical design and construction criteria for the proposed Section 14 Emergency Shoreline Protection in East Hagåtña, Guam. Information and assumptions in this report were developed through a site visit and it is intended for use by design engineers and planners to evaluate the feasibility of proposed flood barrier. Information in this report is not intended for use in construction contract documents.

2 Location and Project Description

Guam is the largest and southernmost island of the Mariana Islands located South-Southwest of Saipan. Located 3,950 miles west of Hawaii, Guam is the westernmost point of the United States. The island is approximately 30 miles long, 4 to 12 miles wide with an area of 210 square miles and 110 miles of shoreline. Hagåtña Bay is centrally located on the west coast of the island of Guam. The project area is located along Marine Corps Drive in Hagåtña Bay between the villages of Asan and Tamuning and spans approximately 2100 feet. Coastal flooding and erosion have been investigated by USACE and FEMA under the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973 with 8 reports written between 1979 and 1993, but no federally authorized projects exist in the study area. The approximate project location is shown in Figure 2-1.

Figure 2-1. Project Vicinity in Guam

An existing seawall runs the length of the project area. This wall was built to the elevation of the ground at the time of construction (1990's). However, since then, erosion of the sandy beach underneath the wall has resulted in many sections being critically undercut, and thus degrading the overall performance and functionality of the wall. Due to the continued exposure of the beach to elevated water levels and wave energy, this structure will continue to be susceptible to further damage. Figure 2-2 shows an example of the damage that exist along the seawall.

Figure 2-2. Undercutting of Existing Wall

3 Alternatives and Tentatively Selected Plan

The study team evaluated seven mitigation alternatives (Alternatives 1 through 7) in the process of recommending a TSP. The seven Alternatives considered are shown in the list below and described is the following sections. Alternative 2, a revetement, was selected as the recommended TSP.

- Alternative 1: No Action
- Alternative 2: Revetment:
- Alternative 3: Precast Concrete Seawall
- Alternative 4: Concrete Rubble Masonry Wall
- Alternative 5: Secant Pile Wall
- Alternative 6: Permeation Grouting
- Alternative 7: Beach Fill with Renourishment

3.1 Alternative 1: No Action

Alternative 1 consist of taking no action to repair the wall. The current wall is not founded on a solid foundation and is undermined by coastal forces. The current condition of the wall does not meet the coastal design requirements and is considered unstable.

3.2 Alternative 2: Revetment

Engineered revetments reduce the erosive power of the waves by dissipating the wave energy through the interstices of the armor units. It is anticipated that the revetment will be able to be constructed with concrete armor units, such as tribars, bearing on the limestone bench. Construction of the concrete armor unit revetment will consist of removing the existing wall and keying the concrete armor units into the limestone bench. The rock revetment would be constructed from the toe (-2.5 ft. MSL) up to the crest elevation (+8.5ft. MSL). The concrete armor unit revetment would be comprised of compacted fill as the foundation and base grade, a geotextile filter fabric, and a single layer of concrete armor units. To ensure stability of the structure and maintain economic feasibility, the armor unit sizes calculated for the depth limited wave height of 2.8 ft were used in the designs. The expected design life of this system (assuming proper installation and routine maintenance) is on the order of 50 years. The concrete armor units will need to be frabricated from a certified concrete precast company. Figure 3-1 shows an example a rock revetment.

Figure 3-1. Typical Concrete Armor Unit (tribar) Revetment

The typical cross section for rock revetment is shown in Figure 3-2.

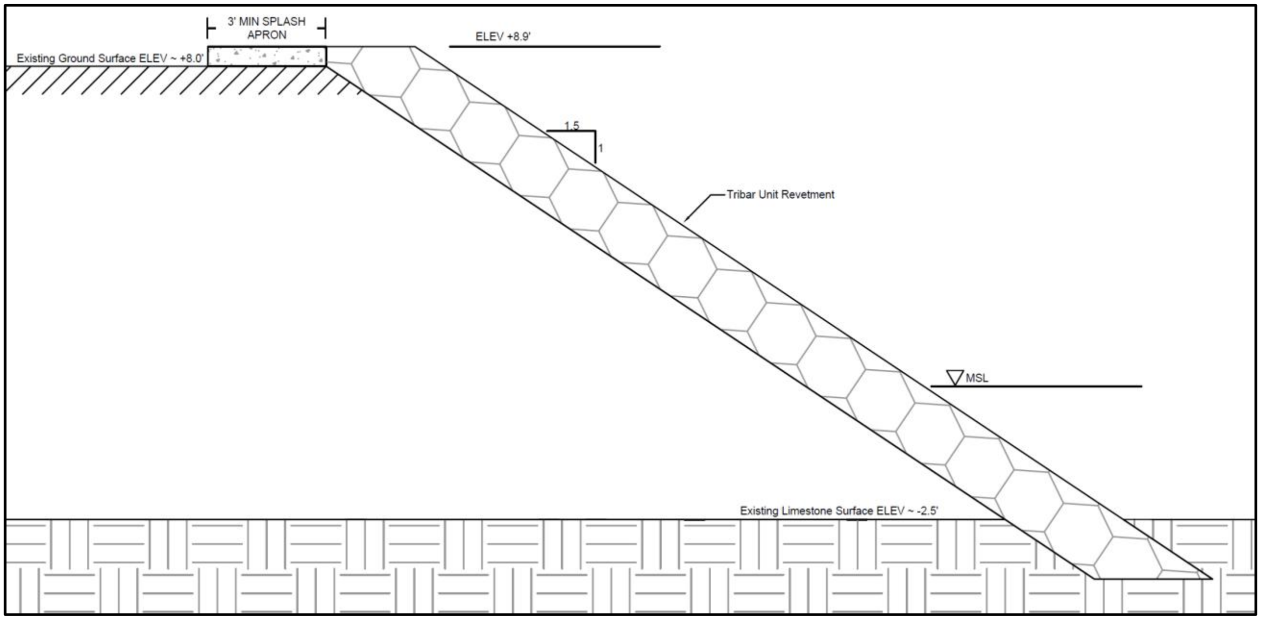

Figure 3-2. Typical Detail of a Tribar Revetment

3.3 Alternative 3: Tentatively Selected Plan (TSP): Precast Concrete Panel Wall

A precast concrete panel wall consists individual concrete panels that are installed throughout the length of the project. Construction of the precast concrete panel wall will consist of excavating approximately two to three feet of coastal soils and placing the individual wall panels on the limestone shelf. Following the construction of the precast concrete panel wall, the area should be regraded to the elevation of the existing ground surface. Figure 3-3 is an example of a precast concrete panel wall.

Figure 3-3. Typical Precast Concrete Panel Wall

The proposed precast concrete panel wall will act as a cantilever retaining wall. These types of cantilever retaining walls utilize the weight of the backfill to provide resistance to the lateral earth pressures. The typical cross section for a precast concrete panel wall is shown in Figure 3-4.

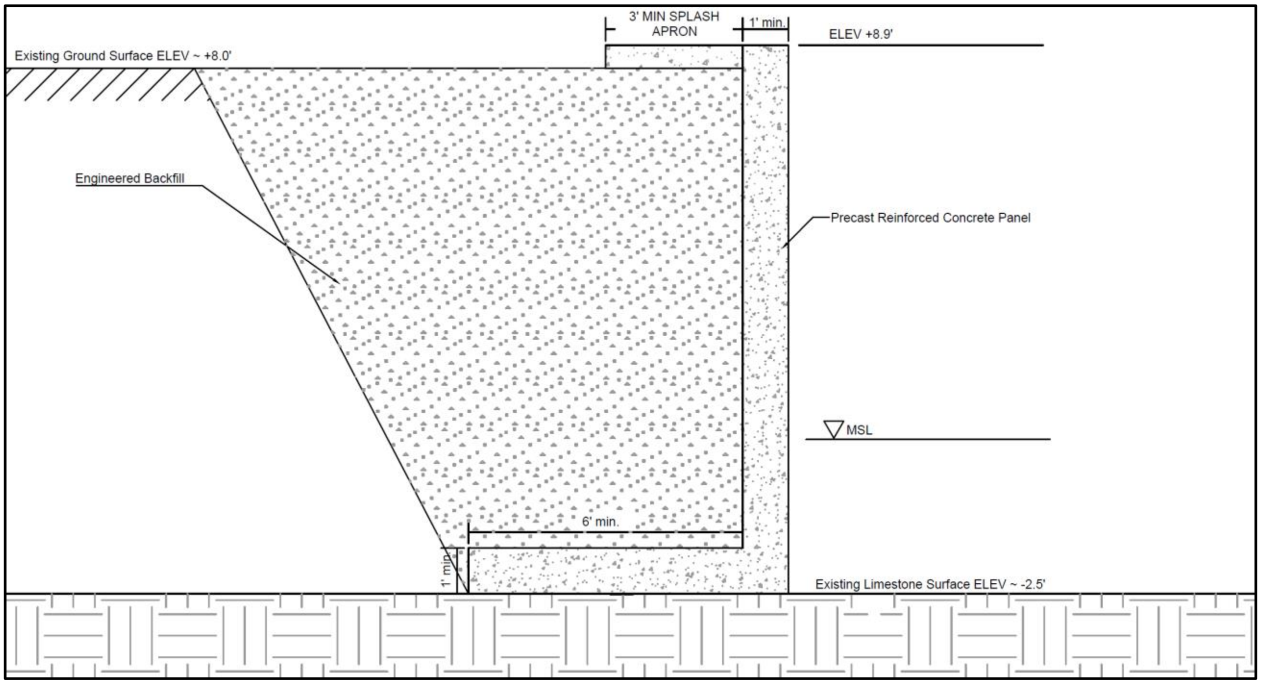

Figure 3-4. Typical Detail of a Precast Concrete Panel Wall

It is anticipated that precast concrete panel wall would be installed within the same footprint of the existing wall. Based on the proposed precast concrete panel cross-section illustrated in Figure 3-4, the final footprint would be approximately 7 feet with the total disturbed area being approximately 20 feet due to excavation and backfill of the existing soils. In addition to the approximately 20 feet of disturbed area, a minimal additional 30 feet will be needed landward of the disturbed area for the working platform of the construction equipment.

3.4 Alternative 4: Concrete Rubble Masonry (CRM) Wall

A concrete rubble masonry wall consists of a CRM wall bearing on a reinforced concrete foundation. Construction of the CRM wall will consist of excavating approximately two to three feet of coastal soils and placing the reinforced concrete foundation on the limestone shelf. Following the construction of the reinforced concrete foundation, a CRM wall will be installed to the planned project heights. Following the construction of the CRM wall, the area should be regraded to the elevation of the existing ground surface. Figure 6 illustrates the surface of a CRM wall.

A concrete rubble masonry wall consists of a CRM wall bearing on a reinforced concrete foundation. Construction of the CRM wall will consist of excavating approximately two to three feet of coastal soils and placing the reinforced concrete foundation on the limestone shelf. The reinforced concrete foundation will need to be keyed into the limestone shelf for slip stability. Following the construction of the reinforced concrete foundation, a CRM wall will be installed to the planned project heights. Following the construction of the CRM wall, the area should be regraded to the elevation of the existing ground surface. Figure 3-5 illustrates the surface of a CRM wall.

Figure 3-5. Typical surface of a Concrete Rubble Masonry Wall

The proposed CRM wall will act as a gravity retaining wall. Gravity retaining walls use their own weight to resist the lateral earth pressures. The typical cross section for a CRM wall is shown in

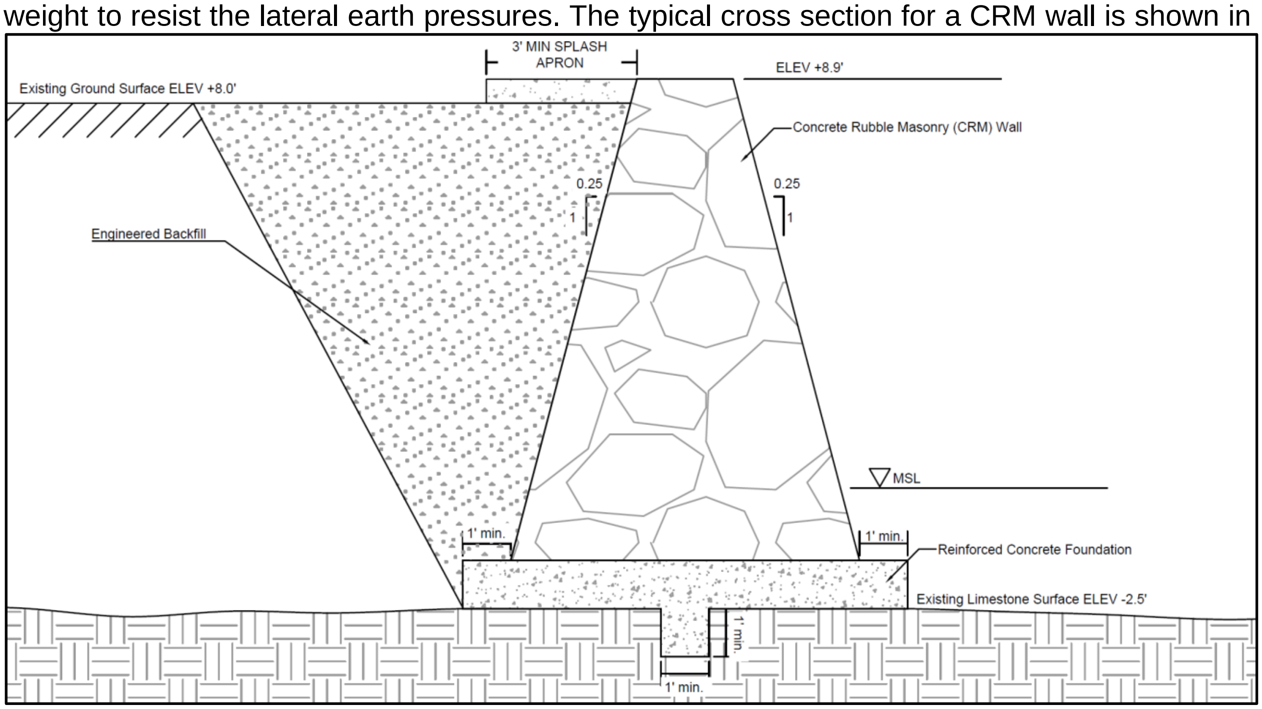

Figure 3-6.

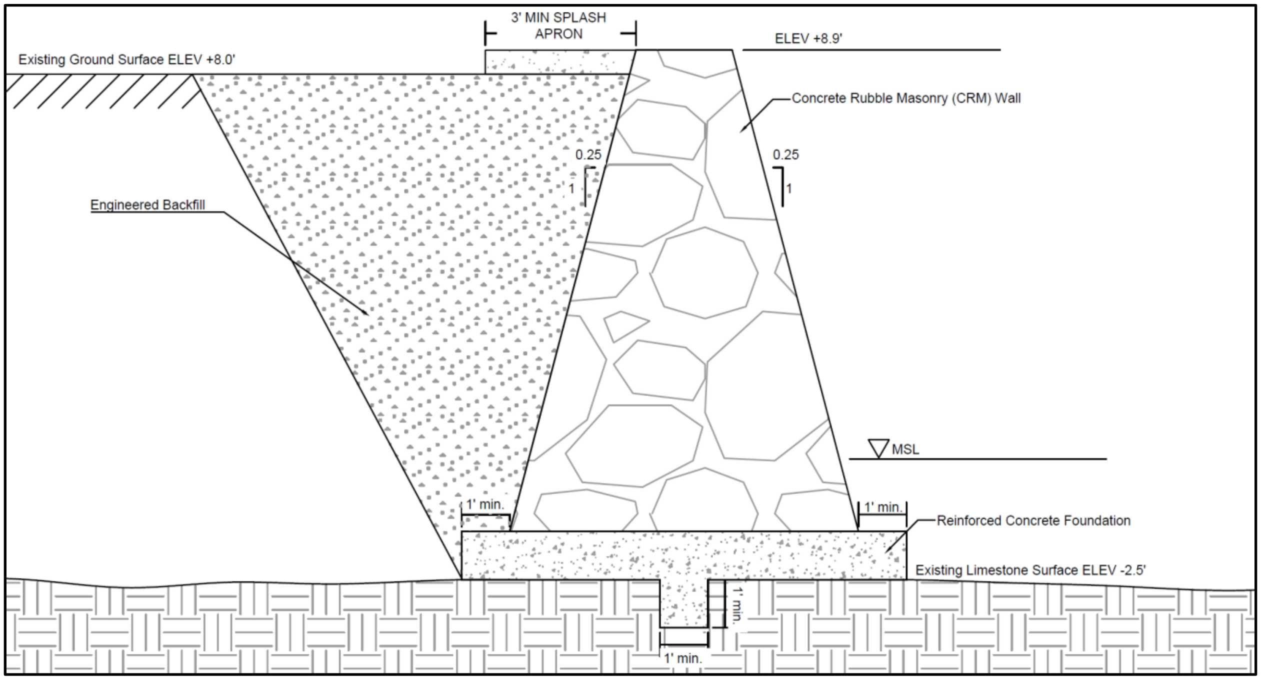

Figure 3-6. Typical surface of a Concrete Rubble Masonry Wall

It is anticipated that a CRM wall would be installed within the same footprint of the existing wall. Based on the proposed CRM cross-section illustrated in Figure 7, the final footprint would be approximately 9 feet with the total disturbed area being approximately 20 feet due to excavation and backfill of the existing soils. In addition to the approximately 20 feet of disturbed area, a minimal additional 30 feet will be needed landward of the disturbed area for the working platform of the construction equipment.

3.5 Alternative 5: Secant Pile Wall

Secant piling is a robust, rigid system which can be used to construct earth retention walls. The continuous wall is constructed by drilling overlapped concrete. A wide range of drilling techniques can be employed allowing secant pile walls to be constructed in variable ground conditions. The initial or "primary" piles are drilled into existing ground at the selected center spacing. The wall is completed by drilling structurally reinforced "secondary" piles which cut into and overlap with the adjacent primaries. Secant walls overlap individual piles which allows for flexible layouts accommodating linear or curved alignments with multiple corners. Vertical reinforcement is typically installed only in secondary piles and may be either a steel pile or rebar cage.

One benefit of constructing a secant pile wall is that the secant pile wall can be install behind the existing wall. This could add flexibility to the construction schedule, or a cost savings because the existing wall wouldn't necessarily have to be demoed. Figure 3-7 illustrates the look of an exposed secant pile wall.

Figure 3-7. Typical Exposed Secant Pile Wall

The proposed precast concrete panel wall will act as a cantilever retaining wall. These types of retaining walls utilize a rock socket to provide resistance to the lateral earth pressures The typical cross section for a secant pile wall is shown in Figure 3-8.

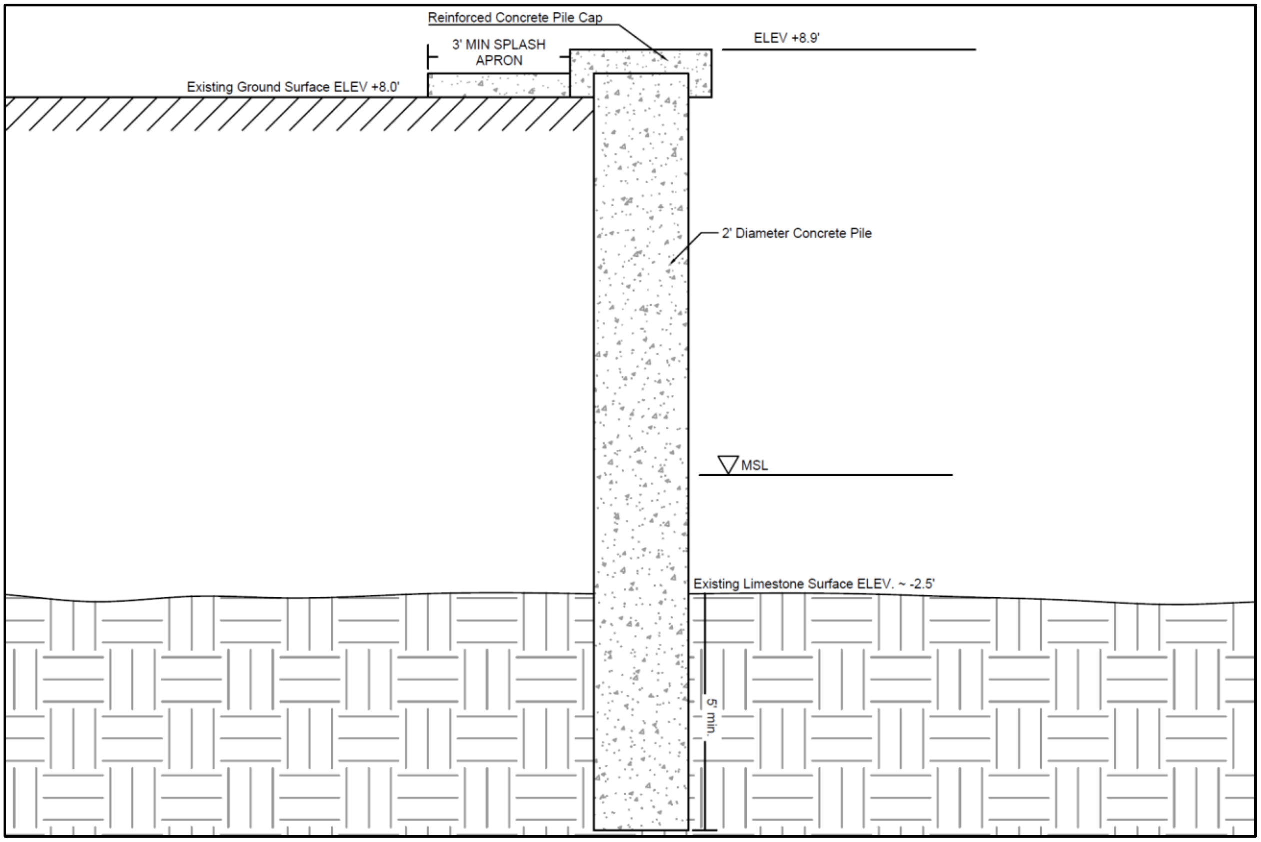

Figure 3-8. Typical Detail of a Secant Pile Wall

It is anticipated that precast concrete panel wall would be installed landward of the existing wall. Based on the secant pile wall cross-section illustrated in Figure 3-8, the final footprint would be approximately 3 feet with the total disturbed area being approximately 5 feet. In addition to the approximately 5 feet of disturbed area, a minimal additional 30 feet will be needed landward of the disturbed area for the working platform of the construction equipment.

3.6 Alternative 6: Permeation Grouting (Interim Risk Reduction Measure)

Permeation grouting could be an interim risk reduction measure to stabilize the existing seawall while a permanent solution is planned and implemented. Permeation grouting consists of injecting a flowable grout into granulated soils conditions to fill cracks or voids and form a solid cemented mass. Permeation grouting transforms granular soils into sandstone-like masses by filling the voids with low viscosity, non-particulate grout. Sands with low fines content are best suited for this technique. Typically, a sleeve port pipe is first grouted into a pre-drilled hole. The chemical grout is injected under pressure through the ports. The grout permeates the soil and hardens, creating a sandstone-like mass. The grouted soil has increased strength, stiffness, and reduced permeability. Permeation grouting offers the advantages of being easily performed where access and space are limited, and where no structural connection to the foundation being underpinned is required. A common application of permeation grouting is to provide both excavation support and underpinning of existing structures adjacent to an excavation. It can typically be accomplished without disrupting normal facility operations. Figure 12 illustrates exposed permeation grouting in sandy soils.

Figure 3-9. Exposed Permeation Grouting in Sandy Soils

The typical detail for permeation grout is shown in Figure 3-10.

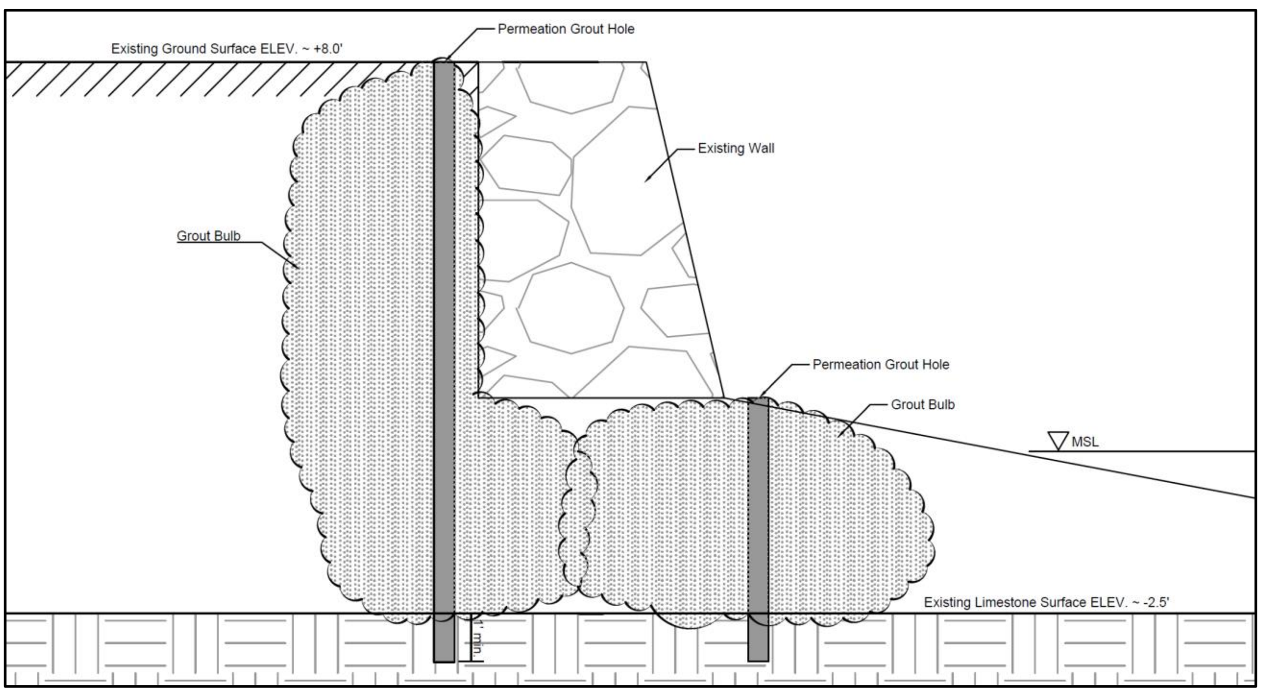

Figure 3-10. Permeation Grout Typical Detail

One benefit of using permeation grouting to stabilize the existing wall is that this method eliminates the cost for demoing the existing wall. A full analysis will need to be evaluated in order to accurately determine the recommended hole spacing. It is anticipated that a five-foot diamond grid pattern of permeation grout holes will be adequate to repair and support the existing wall. The grout holes need to be extended a minimum of one foot into the existing limestone shelf.

It is anticipated permeation grouting would be installed both landward and seaward of the existing wall. Based on the permeation grouting cross-section illustrated in Figure 3-10, the final footprint would be approximately 2 feet landward and 2 feet seaward of the existing wall. In addition to the approximately 2 feet landward and 2 feet seaward of the existing wall, a minimal additional 30 feet will be needed landward of the disturbed area for the working platform of the construction equipment.

3.7 Alternative 7: Beach Fill with Renourishment

Beach nourishment is the adding of sediment onto or directly adjacent to an eroding beach. This "soft structural" response allows sand to shift and move with waves and currents. A wide, nourished beach system absorbs wave energy, protects upland areas from flooding, and mitigates erosion. The beach provides a buffer between storm waves and landward areas, and it can prevent destructive waves from reaching the dunes and upland developments. When sediment is naturally moved offshore from a nourished beach, it causes waves to break farther from the shoreline, which weakens their energy before reaching the shore.

4 Geotechnical Investigations

There has been one geotechnical investigation that was performed by USACE in 1981 for the Agat Small Boat Harbor Project Report near Nimitz Beach Park approximately 10 miles south of the project site. This geotechnical investigation consisted of 11 borings that were drilled to depths between 18.4 and 21.3 feet below ground surface (bgs). Subsurface conditions consisted of unconsolidated clastic sediments, coral limestone, and coral limestone breccia that did not have a consistent stratigraphic sequence. Unconsolidated clastic sediments contained material ranging from calcareous clay/silt to freshly broken, angular gravel, cobbles, and boulders. The coral limestone hardness ranged from easily friable (by hand) to hard, with micro to macro scale voids that contribute to a porosity between 16% and 30%. Compressive strength of the harder limestone was estimated to be 300psi or greater. Due to the distance from the project area and difference in observed conditions, a comprehensive geotechnical investigation will still need to be performed during PED.

5 Regional Geology

Guam is divided across a major fault into two distinct physiographic provinces. To the north is a low-relief limestone plateau with precipitous coastal cliffs standing approximately 200 to 400 feet above sea level. To the south is a deeply dissected west-facing volcanic cuesta with an uplifted limestone unit on its eastern flank, contemporaneous with the cliff-forming unit in the north wand standing approximately 200 feet above sea level. The northern plateau is the detrital Miocene-Pliocene Barrigada Limestone, which extends to the surface in the interior but elsewhere grades laterally and upward into the Pliocene-Pleistocene Mariana Limestone–a reef and lagoonal deposit that dominates the northern plateau. Minor outcrops of Miocene argillaceous Janum Limestone and Holocene reef Merizo Limestone are exposed in coastal areas. A geologic map of the project vicinity is shown in Figure 5-1.

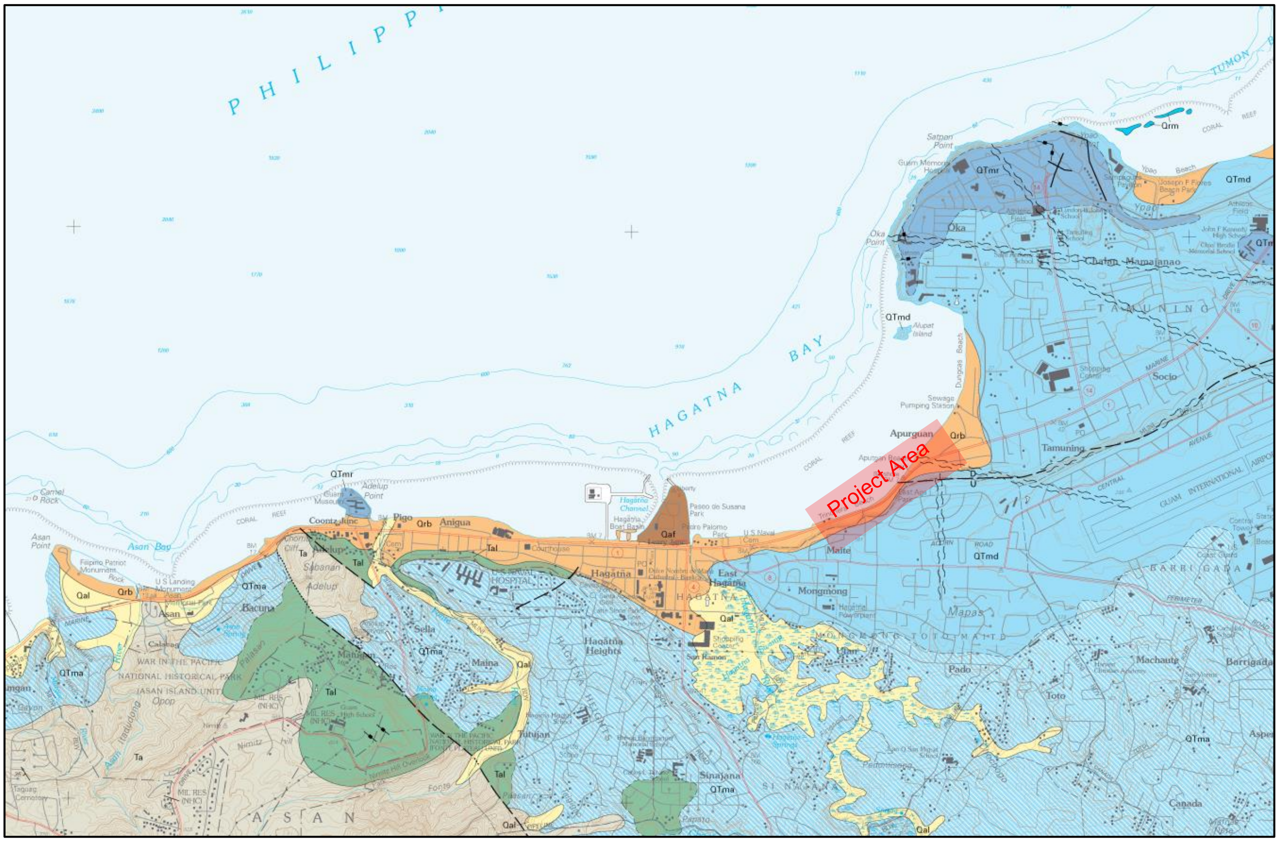

Figure 5-1. Geologic Map of Hagåtña Quadrangle, Guam

6 Geotechnical Design Considerations for TSP

It is anticipated that the proposed revetment can be constructed successfully for the planned project. However, it is important that prudent consideration be given to certain subsurface conditions and construction aspects. These include foundation soils, stability, seismic concerns, and settlement. This engineering analysis is based on information gathered during the March 2022 site visit. The following sections are based on anticipated conditions and need to be reevaluated following a formal subsurface site investigation.

6.1 Anticipated Soil Profile

Based on conditions encountered during the site visit, it is anticipated that the soils in near the proposed location of the coastal revetment will typically consist of 8 to 10 feet of unconsolidated marine sediments (gravels and sands) overlying limestone bedrock. The anticipated soil profile must be confirmed by a geotechnical drilling program.

6.2 Anticipated In Situ Soil Properties

The soil properties used to design the revetment profile are summarized in Table 1. Typical unit weights from Table 5-2 (Coduto, 2001) and Effective internal friction values were estimated in accordance with Table 3-1 of EM 1110-1-1905, *Bearing Capacity of Soils* (1992). The soil properties in Table 1 are anticipated soil properties and will need to be reevaluated following a formal subsurface site investigation.

Table 6-1.: Anticipated Design Foundation Soil Properties

Interpreted Geology	[1]Approximate Depth (ft)	[2] Engineering Property	Unified Soil Classification Symbol	[3]Unit Weight (pcf)	[3] Friction Angle (degrees)
Alluvial Soils	0 – 8	Loose to Medium Dense	SP, SW	90 – 120 (110)	< 29 (27)
Limestone	8 - 50	Hard / Unweathered	Bedrock	140 – 160 (140)	38 - 55 (45)

[1] Depth is indicated as below the existing ground surface.
[2] Engineering properties are anticipated and should be considered approximate.
[3] Ranges of applicable values are presented, recommended value is shown in parentheses

6.3 Preliminary TSP Cross-Section

The preliminary cross-section for the breakwater is shown in Figure 6-1. During the engineering analyses, each soil layer was assumed to be homogeneous and uniform in composition.

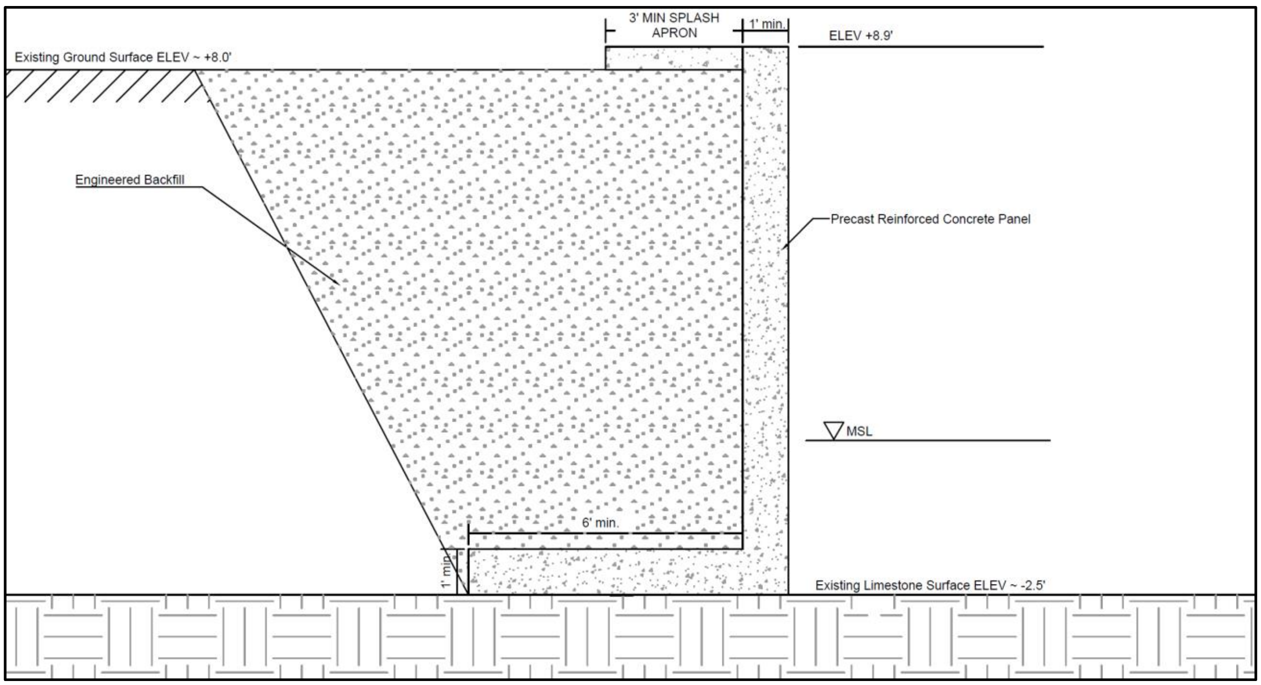

Figure 6-1. Preliminary Seawall Cross-Section

Table 6-2. Seawall Design Parameters

Design Parameter	Drained	Undrained
Friction Angle of Soil Behind Walls, φ’	32	26
Active Earth Pressure Coefficient, K_a (backfill angle = 0)	0.31	0.39
Passive Earth Pressure Coefficient, K_p (backfill angle = 0)	3.25	2.56

6.4 Design Factors of Safety

Appropriate factors of safety must be to ensure adequate performance of the project throughout its design life. Three important considerations in determining appropriate factors of safety include: uncertainties in the conditions being analyzed, the consequences of failure, and the acceptable performance. Table 6-3 provides applicable factors of safety and source documents, which include procedures for performing the analysis.

Table 6-3. Applicable Factors of Safety

Reference	Analysis Condition	Minimum Factor of Safety
EM 1110-1-1904	Settlement Analysis	Conducted to min. crest elevation
EM 1110-1-1905	Bearing Capacity	2.5
EM-1110-2-1902	Slope Stability, End of Construction	1.3
EM-1110-2-1902	Slope Stability, Long Term	1.5
EM-1110-2-1902	Slope Stability, Earthquake Loading	>1.0

6.5 Tide Conditions

The tides at East Hagåtña are generally diurnal with two highs and two lows occurring daily. Tide levels, referenced to Mean Lower Low Water (MLLW), are shown in Table 6-4. Water level data is from the National Oceanic and Atmospheric Administration (NOAA) online database.

Table 6-4. Tidal data for the East Hagåtña Shoreline Protection Project Referenced to MLLW

Tide	* Elevation (feet)
Mean Higher High Water (MHHW)	+2.34
Mean High Water (MHW)	+2.22
Mean Tide Level	+1.41
Mean Low Water (MLW)	+0.60
Mean Lower Low Water (MLLW)	0.00
* Source: NOAA National Ocean Surface	

6.6 Seismic Design Parameters

East Hagåtña, Guam is in a seismic region of the Southwest Pacific where large magnitude earthquakes occur. Structures shall be designed to meet or exceed seismic requirements in ER 1110-2-1806 “Earthquake Design and Evaluation for Civil Works Projects.” It is unnecessary to analyze the liquefaction settlement due to seismicity as the structure will be founded in rock.

The proposed structure is assigned a Seismic Design Category D per Section 11.6-1 of American Society of Civil Engineers (ASCE) 7-22, since the mapped spectral response acceleration parameter at 1-second period, S_1, is less than 0.75 and the short-period response acceleration parameter, S_{Ds}, is greater than 0.50 at the project site. Seismic data for Agat, Guam was determined using the probabilistic seismic hazard maps of Alaska provided by the U.S. Geological Survey (USGS) and the ASCE 7 Hazard Tool and is shown Table 6-5 using a 2% probability of exceedance in 50 years. The specified design ground motions are for Site Class C. Seismic design ground motion parameters are provided for ASCE 7-22.

Table 6-5. Seismic Design Ground Motion Parameters

Parameter	ASCE 7-22
Site Class	C
Site-Specific PGA_M	0.99
S_1	0.65
S_{D1}	0.62
S_s	3.03
S_{Ds}	2.14

The proposed facility is assigned a Risk Category I in accordance with Table 2-2 of the UFC 3-301-01 Structural Engineering (2023) since the structure poses a low hazard to human life in the event of failure.

7 Preliminary Geotechnical Analysis of TSP

The following sections are based on information gathered during the March 2022 site visit and assumptions on the subsurface conditions. These sections should only be as a check of the feasibility of the tentatively selected plan and are not adequate for a formal design analysis. A formal subsurface site investigation needs to be performed in order to evaluate and check the accuracy of the assumptions.

7.1 Bearing Capacity Analysis

A preliminary bearing capacity analysis was performed in order to ensure the foundation soil/rock has a bearing capacity that is suitable for the seawall. The allowable bearing pressure for the limestone bedrock was taken from Table 1806.1 from the NYC Building Code (2022). This limestone was assumed to be “soft rock” (a with a maximum allowable pressure of 16 ksf. The seawall loading is calculated as:

$$Q_{revetment} = 150pcf \cdot (11ft) = 1.65ksf$$

Since the seawall is founded in the limestone bedrock, it is assumed that all the load from the seawall will be supported by the limestone. Based on the assumptions above, the maximum allowable pressure of the limestone is greater than the calculated seawall loading pressure, so the seawall is assumed to be stable with respect to bearing capacity.

7.2 Global Slope Stability Analysis

A preliminary slope stability analysis was performed for the open cell pile wall. Geostudio Slope/W was used to determine the global slope stability factor of safety for the open cell piling seawall. The backfill was analyzed using Mohr-Coulomb whereas the limestone bedrock was analyzed as undrained. The undrained assumption is conservative in this case as it ignores any residual strength that the limestone bedrock has. Only circular slip surfaces were considered for this analysis. The model also assumed that all the sand on the beach would be eroded away (and would not provide passive pressure). This model is very conservative as it is essentially the worst case scenario. The calculated factor of safety for the Slope/W model was 14.4 which well exceeds the required factors of safety per EM 1110-2-1902. Model results can be found in Appendix C.

7.2.1 Seismic Stability Analysis

Seismic stability of the seawall will be accounted for and designed during the preconstruction engineering and design phase. It is recommended that a liquefaction analysis also be performed in conjunction with the seismic stability analysis. Data collected during future geotechnical investigations will help to determine the materials parameters to be used in the seismic stability and liquefaction analyses. Ground motion parameters to be used during PED can be found in Attachment B.

7.3 Settlement Analysis

The seawall will be founded in competent rock, so settlement is not expected and is not necessary to be evaluated.

8 Future Geotechnical Site Investigation Recommendations

It is recommended that a geotechnical site investigation consisting of a geophysical survey and geotechnical drilling be conducted during the preconstruction engineering and design (PED) phase of the project. The geophysical survey should include techniques to map the top of bedrock and to correlate the rock quality parameters. The geotechnical drilling program will include drilling between 5 and 10 test borings along the centerline of the proposed seawall a minimum of 10 feet into the limestone bedrock. Laboratory testing of the sediment material will consist of gradations, Atterberg limits, moisture contents, and direct shear tests. Laboratory testing of the encountered rock include recovery, rock quality designation (RQD), unit weight, unconfined compression test (USC), tensile testing, Mohs hardness, and CERCHAR Abrasively Index (CAI). It is also recommended that a geophysical survey (e.g., seismic refraction) be conducted to map the top of bedrock, as the depth to bedrock may not be consistent/planar across the entire site. Seismic wave velocities from the geophysical surveys may also be used to infer bedrock ripability for pile driving and/or excavation. The main goal of a geotechnical site investigation and geophysical survey would be to properly characterize proposed foundation material and identify any geological conditions that would require special considerations during PED. Geotechnical and geophysical information would also be used to establish accurate cost estimates.

9 Geotechnical Engineering Evaluation

Given the information gathered during the March 2022 site visit and the stated geotechnical assumptions, there are no anticipated height or width limitations on designing or constructing the proposed emergency shoreline protection. There is also no special foundation requirements needed to address concerns of slope stability, bearing capacity, or settlement of the structure

10 References

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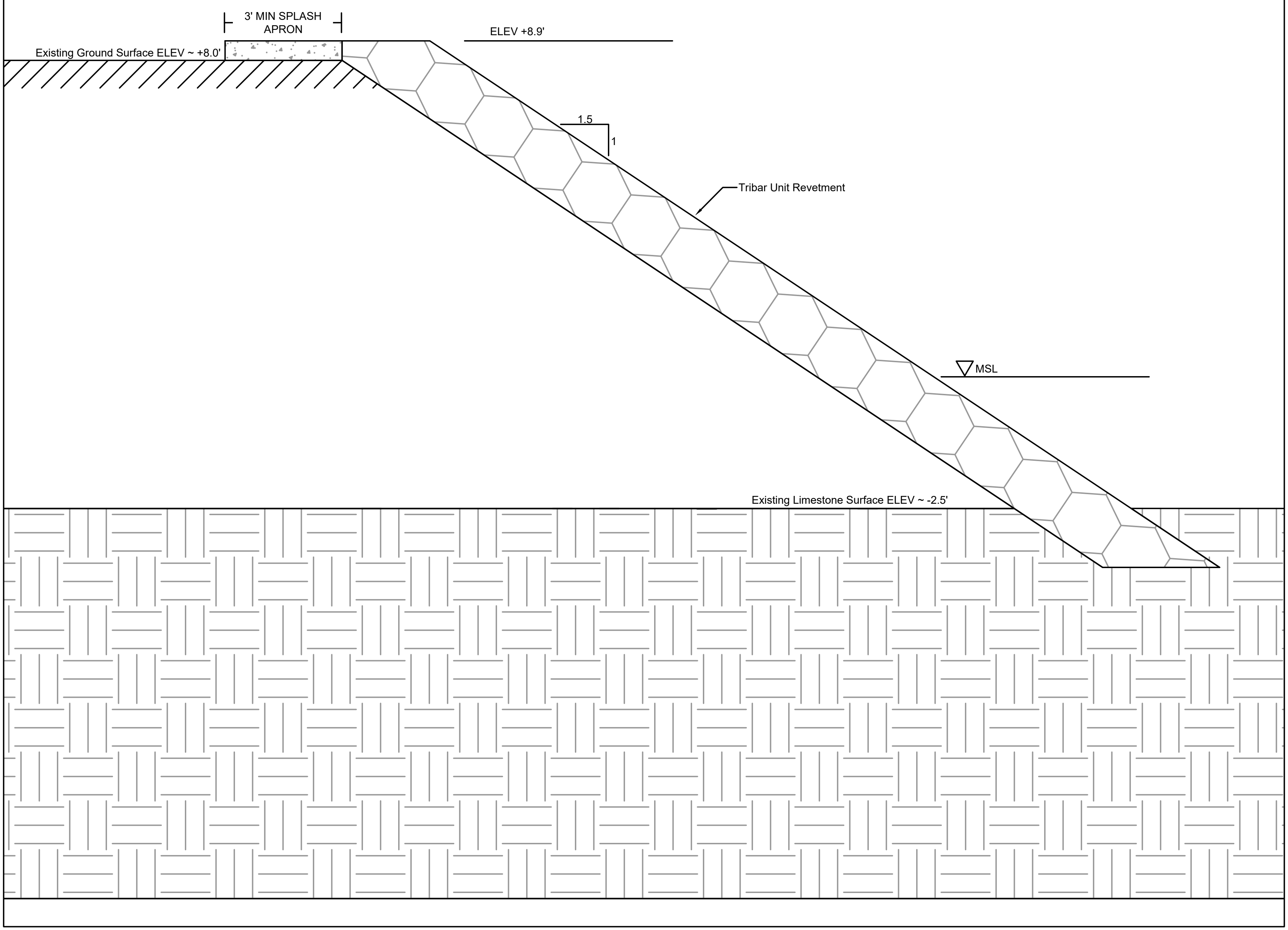
APPENDIX A
ALTERNATIVES DESIGN CROSS SECTIONS

U.S. ARMY CORPS OF ENGINEERS

TRIBAR REVETMENT TYPICAL DETAIL



SCALE: NTS
DATE: APRIL 2022
DRAWN/RVW: JJM/JJR
FIGURE: 1 of 5

REVETMENT TYPICAL DETAIL
CAP Section 14 East Hagåtña
East Hagåtña, Guam

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Geotechnical and Materials

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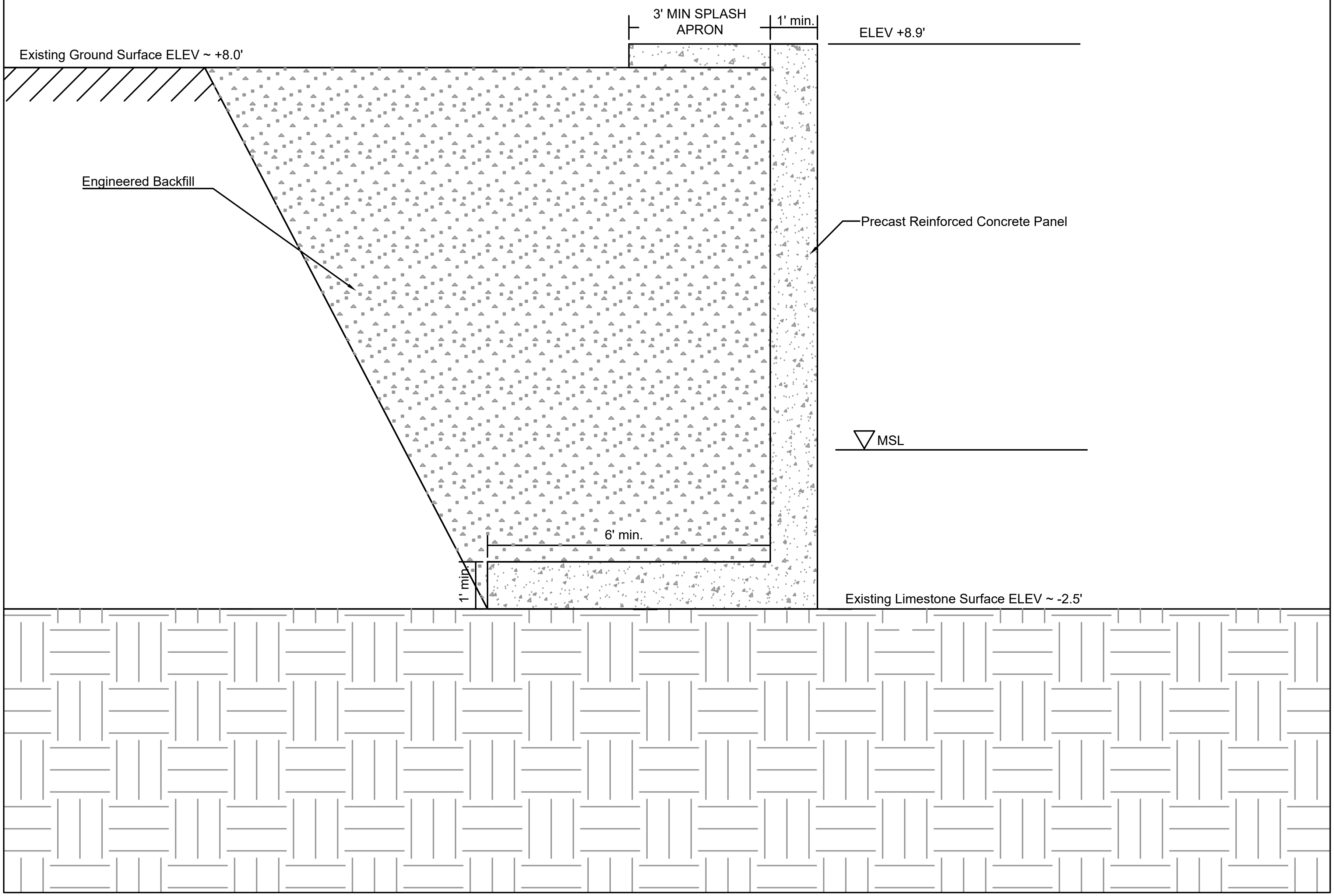
PRECAST WALL TYPICAL DETAIL



SCALE: NTS
DATE: MARCH 2024
DRAWN/RVW: JJM/JJR
FIGURE: 3 of 5

PRECAST CONCRETE PANEL TYPICAL DETAIL
CAP Section 14 East Hagåtña
East Hagåtña, Guam

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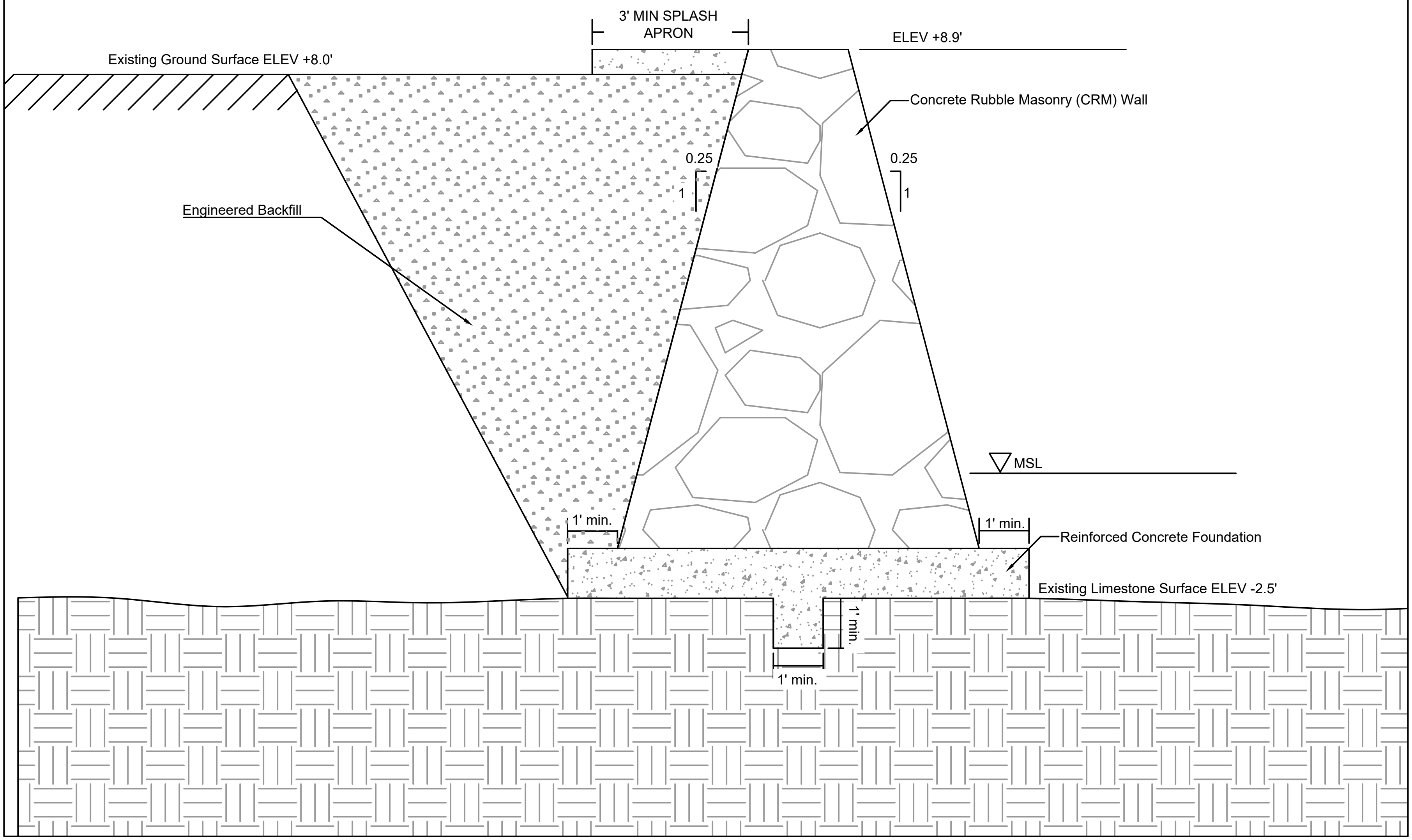
HYBRID CRM WALL
TYPICAL DETAIL



SCALE: NTS
DATE: MARCH 2024
DRAWN/RVW: JJM/JJR
FIGURE: 2 of 5

HYBRID CRM WALL TYPICAL DETAIL
CAP Section 14 East Hagåtña
East Hagåtña, Guam

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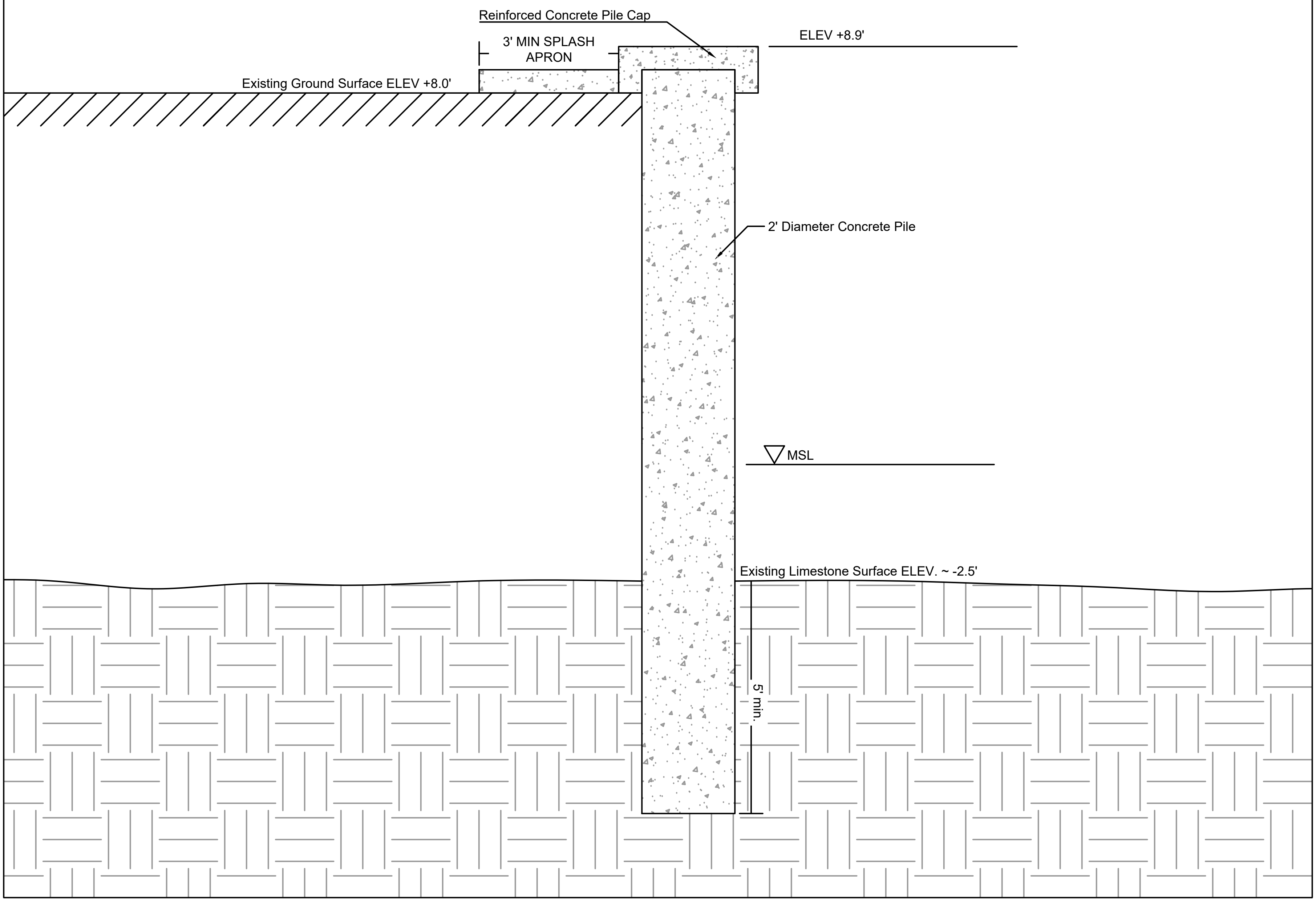
SECANT PILE WALL
TYPICAL DETAIL



SCALE: NTS
DATE: MARCH 2024
DRAWN/RVW: JJM/JJR
FIGURE: 4 of 5

SECANT PILE WALL TYPICAL DETAIL
CAP Section 14 East Hagåtña
East Hagåtña, Guam

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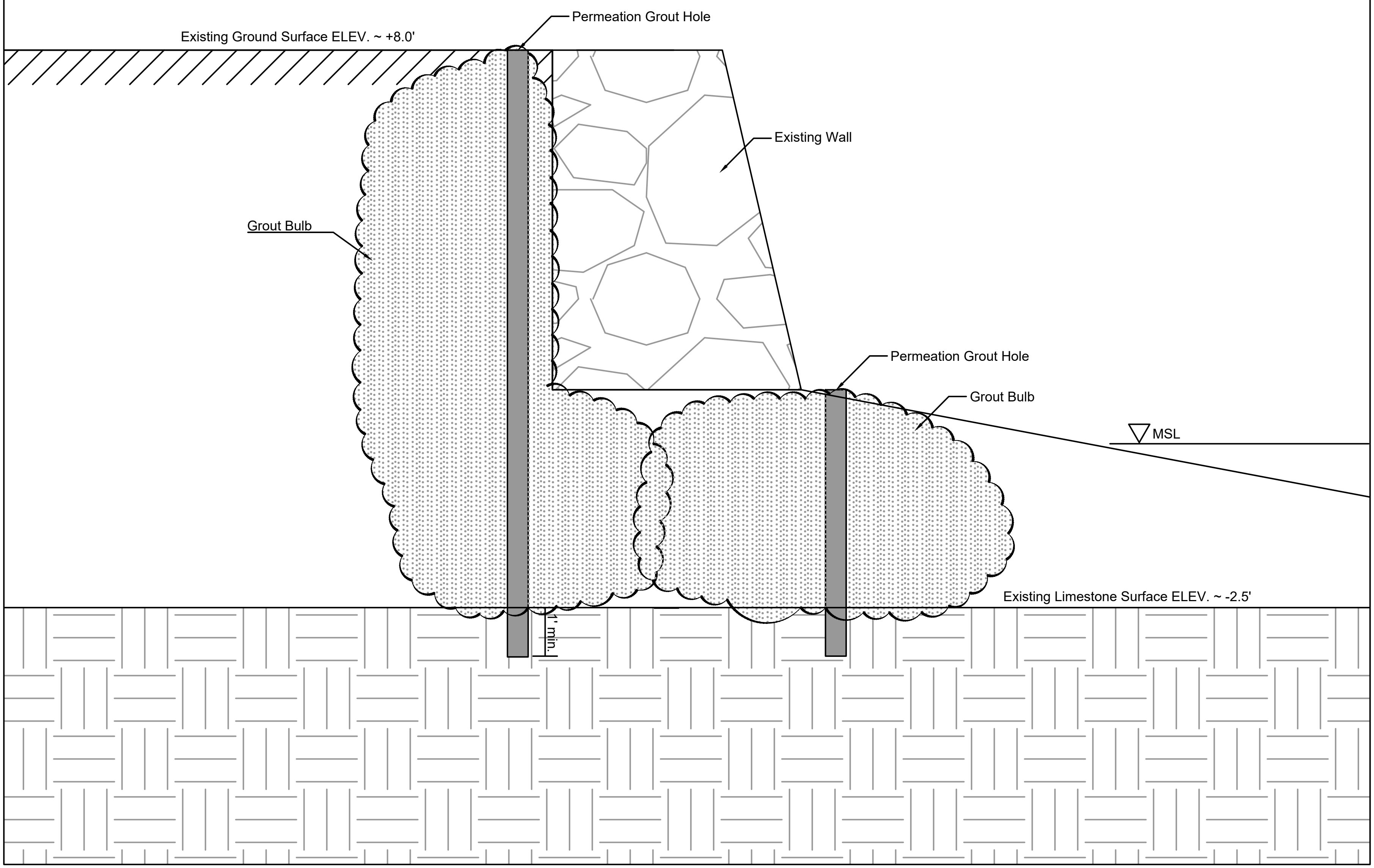
PERMEATION GROUT TYPICAL DETAIL



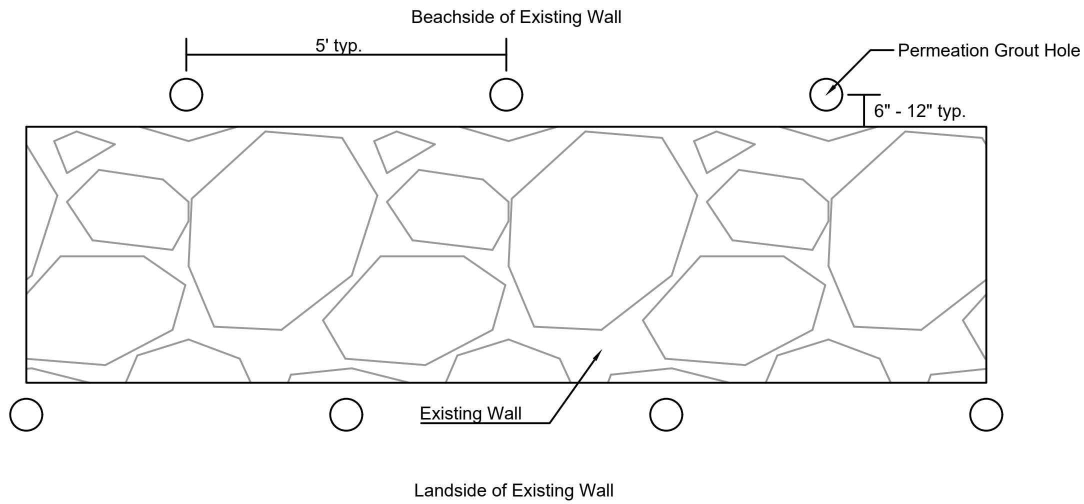
SCALE: NTS
DATE: APRIL 2022
DRAWN/RVW: JJM/JJR
FIGURE: 5 of 5

PERMEATION GROUTING TYPICAL DETAIL
CAP Section 14 East Hagåtña
East Hagåtña, Guam

ALASKA DISTRICT
CORPS OF ENGINEERS
Geotechnical and Materials

U.S. ARMY
CORPS OF ENGINEERS

U.S. ARMY CORPS OF ENGINEERS



PERMEATION GROUTING TYPICAL DETAIL
CAP Section 14 East Hagåtña
East Hagåtña, Guam

SCALE: NTS
DATE: APRIL 2022
DRAWN/RVW: JJM/JJR
FIGURE: 1 of 1

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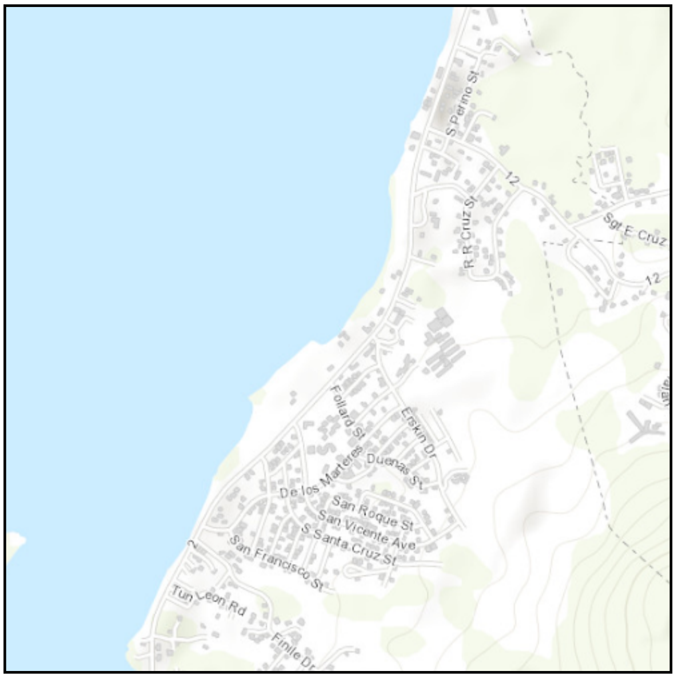
APPENDIX B

SEISMIC DESIGN PARAMETERS

ASCE Hazards Report

Address:
No Address at This Location

Standard:	ASCE/SEI 7-22	**Latitude:**	13.388132
Risk Category:	I	**Longitude:**	144.659088
Soil Class:	C - Very Dense Soil and Soft Rock	**Elevation:**	5.37961543133404 ft (NAVD 88)



ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

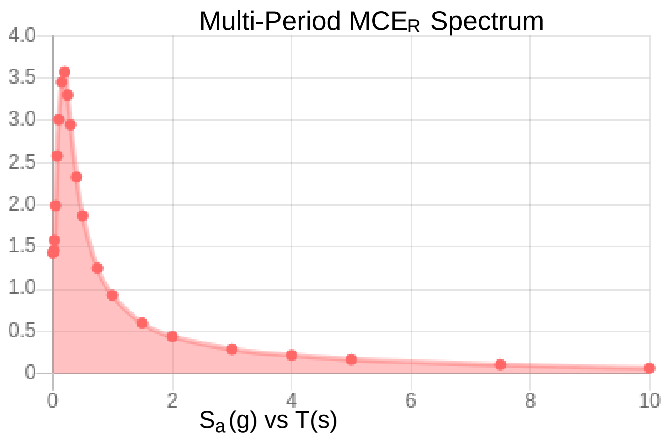
Seismic

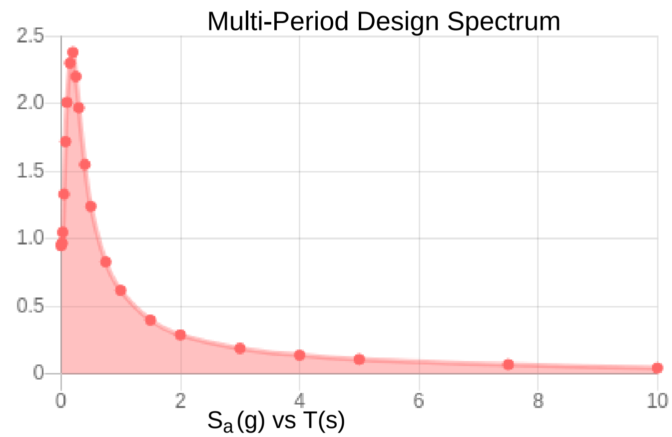
Site Soil Class:	C - Very Dense Soil and Soft Rock

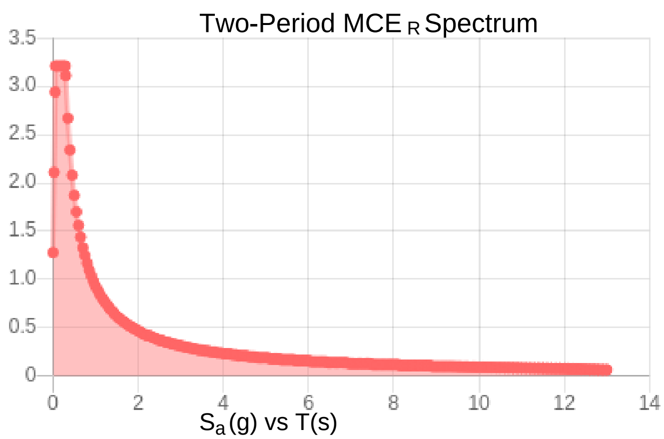
Results:

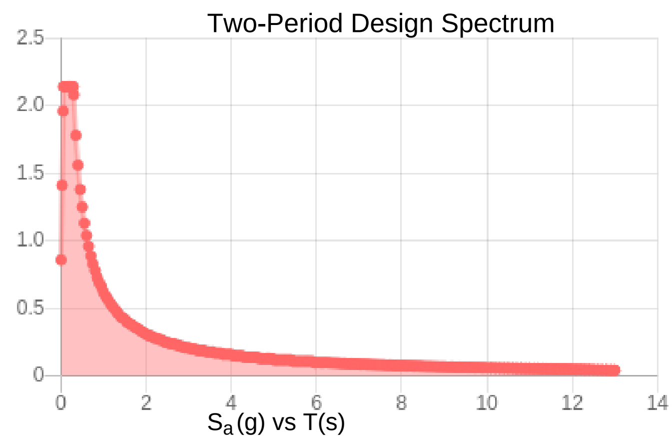
PGA_M :	0.99	T_L :	12
S_{MS} :	3.21	S_S :	3.03
S_{M1} :	0.93	S_1 :	0.65
S_{DS} :	2.14	V_{S30} :	530
S_{D1} :	0.62		

Seismic Design Category: D









MCE$_R$ Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: **Thu Feb 29 2024**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

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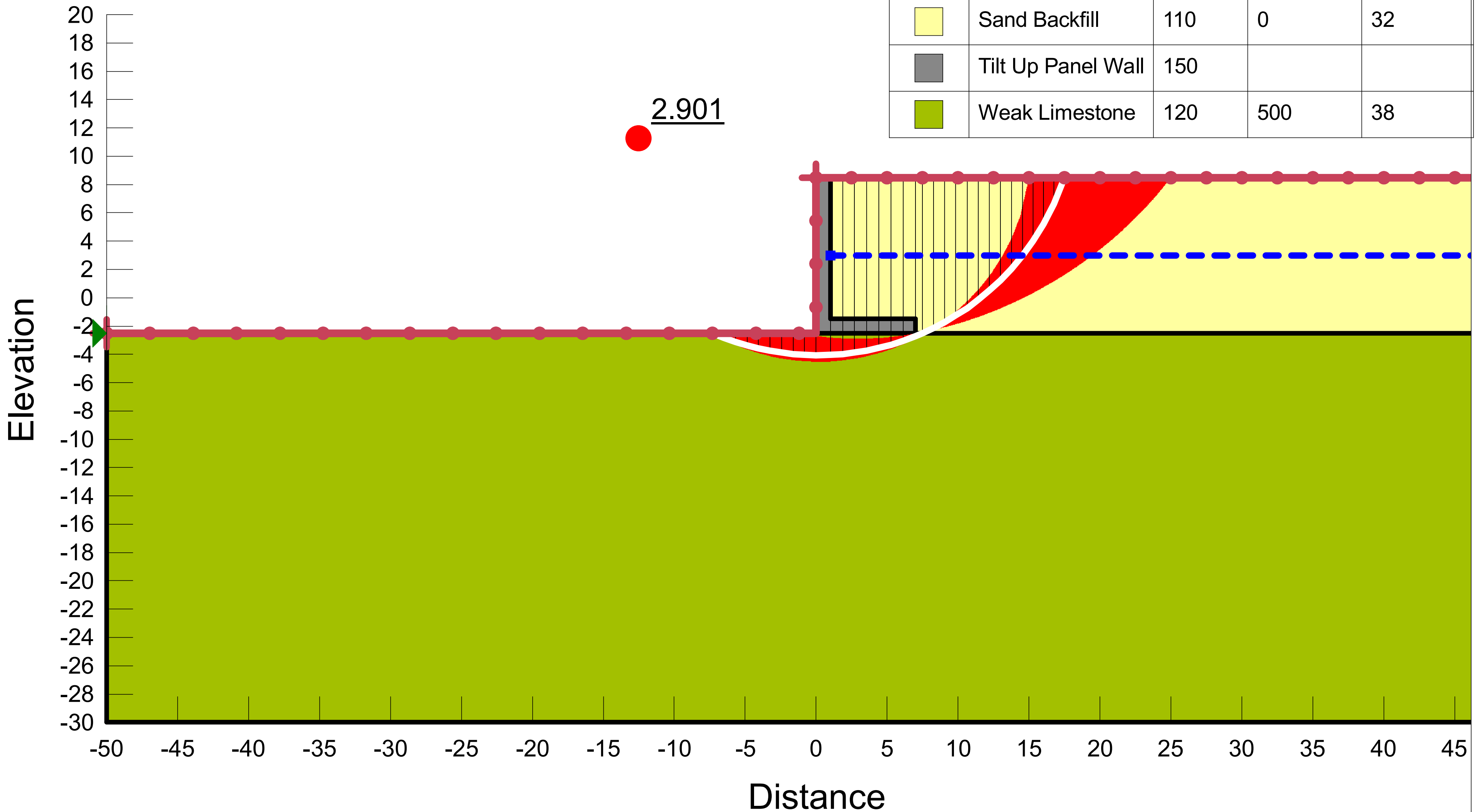
APPENDIX C
GLOBAL STABILITY RESULTS

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Sand Backfill	110	0	32
	Tilt Up Panel Wall	150		
	Weak Limestone	120	500	38



US Army Corps of Engineers
Alaska District

Slope Stability			
Project:	Section 14 Emergency Shoreline Protection	Date:	05/13/2025 11:30:26 AM
Location:	East Hagåtña, Guam	Factor of Safety:	2.901
Engineer:	Justin M. Miller, P.E.	Scale:	1:125

US Army Corps of Engineers®
Alaska District

Precast Panel Retaining Wall Factor of Safety

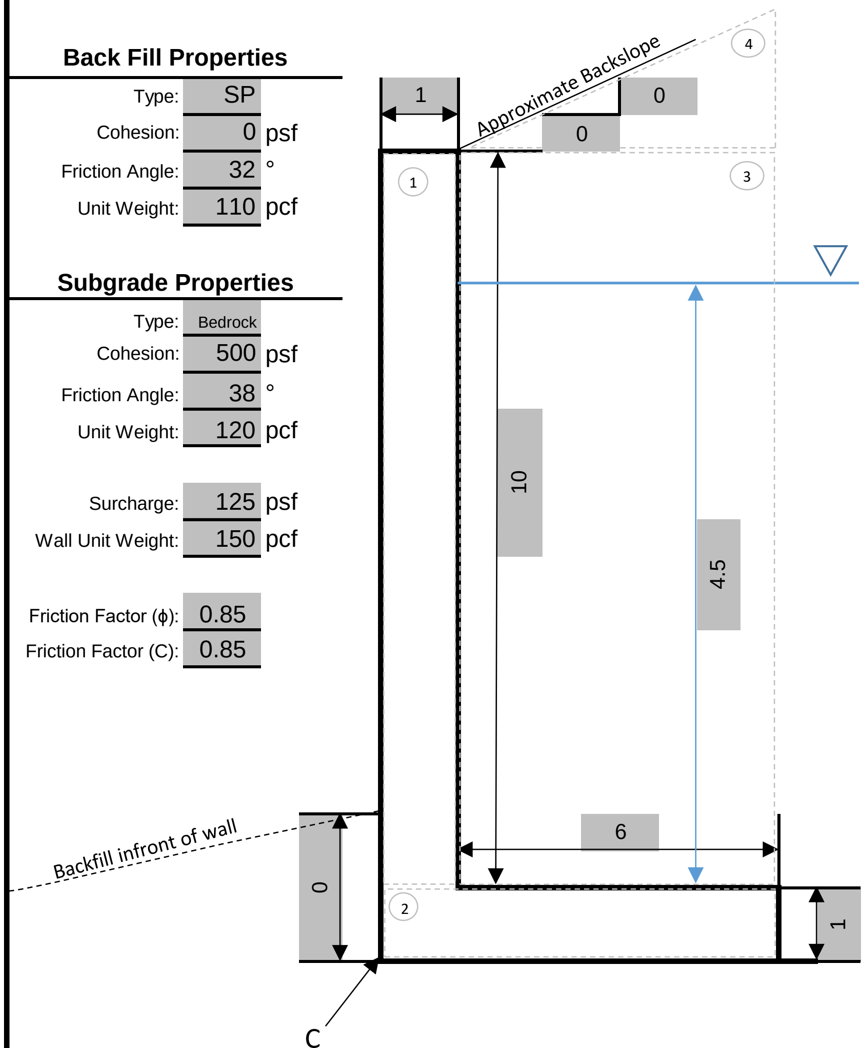
Project:	Section 14 Emergency Shoreline Protection	Date:	5/13/2025
Location:	East Hagåtña, Guam	Engineer:	Justin M. Miller, P.E.

Back Fill Properties

Type:	SP	
Cohesion:	0	psf
Friction Angle:	32	°
Unit Weight:	110	pcf

Subgrade Properties

Type:	Bedrock	
Cohesion:	500	psf
Friction Angle:	38	°
Unit Weight:	120	pcf
Surcharge:	125	psf
Wall Unit Weight:	150	pcf
Friction Factor (ϕ):	0.85	
Friction Factor (C):	0.85	



K_a:	0.30726	Alpha:	0 °	P_a:	2904.96 lb/ft
K_p:	4.20375	H':	11.000 ft	P_p:	0.00 lb/ft

Overturning Resisting Factors

Section	Area (ft^2)	Weight (lb/ft)	Moment Arm (ft)	Moment about C (lbft/ft)
1	10.00	1500.0	0.50	750.0
2	7.00	1050.0	3.50	3675.0
3	60.00	4915.2	4.00	19660.8
4	0.00	0.0	0.00	0.0

$$q_u = c'_{subgrade} N_c F_{cd} F_{ci} + q N_q F_{qd} F_{qi} + \frac{1}{2}\gamma_{subgrade} B' N_\gamma F_{\gamma d} F_{\gamma i}$$

N_C:	77.5	N_q:	61.5	N_γ:	82.3
F_{cd}:	1.11	F_{qd}:	1.06413	$F_{\gamma d}$:	1
F_{ci}:	0.58331	F_{qi}:	0.58331	$F_{\gamma i}$:	0.194

	General Stability		Construction Excavation
q_u:	33145.3 psf	q_u:	33145.3 psf
q_{toe}:	2745.8 psf	q_{toe}:	2745.8 psf
q_{heel}:	-362.9 psf	q_{heel}:	-362.9 psf

$$FS_{Overturn} = \frac{M_1 + M_2 + M_3 + M_4 + M_5}{P_a cos\alpha \left({H'}/{3}\right) - M_v}$$

$$FS_{Bearing} = \frac{q_u}{q_{toe/heal}}$$

$$FS_{Sliding} = \frac{\sum V \tan\left(k_1 \varphi'_{subgrade}\right) + B\left(k_2 c'_{subgrade}\right) + P_p}{P_a cos\alpha}$$

	General Stability		Construction Excavation
$FS_{Overturn}$:	2.26	$FS_{Overturn}$:	2.26
$FS_{Sliding}$:	2.81	$FS_{Sliding}$:	2.81
$FS_{Bearing}$:	12.07	$FS_{Bearing}$:	12.07
FS_{Global}:	2.90	FS_{Global}:	2.90