Oregon International Port of Coos Bay

Proposed Section 204(f)/408
Channel Modification Project

Sub-Appendix 6

Channel Side Slope Analysis

June 2024 Draft

EXECUTIVE SUMMARY

The Oregon International Port of Coos Bay (OIPCB or Port) seeks to modify the Coos Bay, Oregon Federal Navigation Channel (FNC); the channel modifications assessed in this evaluation is referred to as the Proposed Alteration (PA). The PA consists of widening the channel to a nominal 450 feet (ft) and deepening it to -57 ft Mean Lower Low Water (MLLW) at the entrance to the Coos Bay FNC and to -45 ft MLLW from inside the entrance through approximately River Mile (RM) 8.2.

The purposes of this report are to:

- Evaluate the geotechnical stability of the initial proposed dredge cut slopes during and at the completion of capital dredging;
- Estimate the range of slope angles to which the side slopes will equilibrate following the conclusion of capital dredging;
- Estimate the anticipated duration for the side slope equilibration process;
- Estimate the range of dredge quantities associated with side slope equilibration for the PA for the purpose of calculating project costs and cost sharing;
- Estimate the physical zone of equilibration of the channel modifications following the equilibration process; and
- Evaluate the effect of the equilibrium side slopes on adjacent resources.

This report presents a geotechnical evaluation of the initial dredge cuts and an assessment of the future equilibrium side slopes that may result from the Coos Bay Channel Modification Project, including the long-term geotechnical stability of the side-slopes adjacent to infrastructure. After the completion of capital dredging, side slopes will continue to evolve until they reach a stable slope angle, after which sedimentation patterns will reach a future equilibrium state. Estimating the expected side slopes is critical for the purpose of predicting the total dredge volumes that may result from channel equilibration process and for the purpose of estimating potential effects to federal infrastructure and other resources. This analysis recognizes the inherent uncertainty in predicting the future equilibrium side slope, and therefore predicts a range of side slope outcomes. Three side slope conditions were estimate and applied for various applications as follows:

- For an assessment of existing (without project) conditions, channel side slopes were based on the *Median Measured* side slope.
- For an assessment of future (with project) conditions, channel side slopes were based on an estimated *Future Equilibrium* condition. Various methods to predict the future equilibrium condition were applied, based on the purpose of the analysis. Capital dredge volumes and costs were predicted based on a variety of analyses, and the morphologic processes specific to each reach. In order to predict project impacts to adjacent infrastructure and future increased O&M volumes, a more conservative future equilibrium condition was based on the median measured side slope, assumed to originate at the toe of the dredged channel slope (rather than from the current channel bottom, which in many reaches where sediment deposition occurs is a shoaled condition that is above the toe of the existing channel).

- The *Constructed Condition* exclusively refers to the immediate, post-construction condition.

The Port used the following general process for the side slope analysis. The process is illustrated in Figure ES-1:

1) Evaluate the geotechnical stability of the initial proposed dredge cut slopes during and at the completion of capital dredging. The evaluations began with 3:1 (Horizontal: Vertical) sediment slopes and 1:1 rock slopes and included the following evaluations:
    a. Evaluate the static slope stability of sediment and rock slopes.
    b. Evaluate the static liquefaction and flow failure potential of sediment slopes.
2) The Port utilized the geotechnically stable configuration evaluated in Step 1 above (i.e. the constructed condition) to estimate the range of slope angles to which the channel side slopes may equilibrate following dredging and included following evaluations:
    a. Estimate the anticipated duration for the side slope equilibration process.
    b. Estimate the range of dredge quantities associated with the channel side slope equilibration for the PA.
    c. Estimate the zone of equilibration of the equilibrated channel and effects of the equilibrium side slope on adjacent infrastructure.
3) The Port utilized the zone of the equilibration estimated in Step 2c above to evaluate the geotechnical global stability of the future equilibrium side slopes and adjacent infrastructure. The results of the evaluation were compared to the existing global stability at each adjacent infrastructure location. If the evaluation indicated a significant change to the global stability, the Port re-evaluated the global stability with mitigation in place.

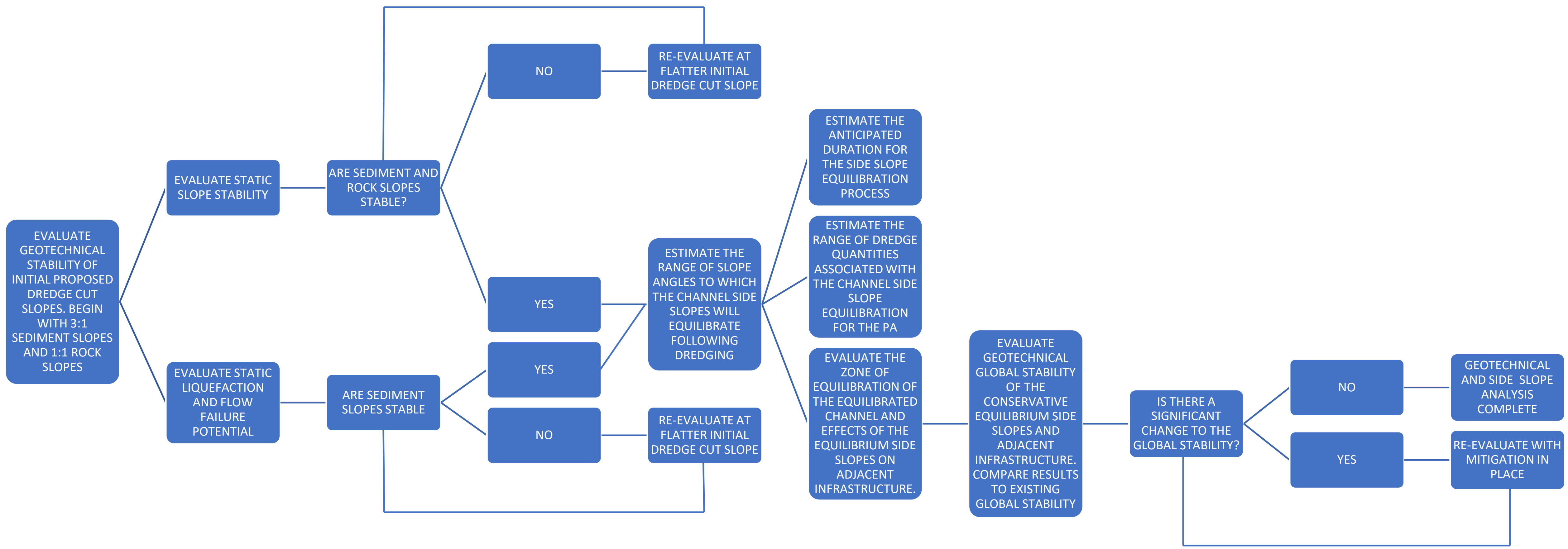

Figure ES-1
Side Slope Analysis Process Flow Chart

The geotechnical stability analyses indicate the proposed *constructed condition* dredge cuts of 3:1 (Horizontal to Vertical) for sediment and 1:1 for rock have acceptable (i.e., greater than 1.5) static slope stability factors of safety, except where relatively large cuts in zones of potential loose sand result in a relatively high risk of dredging-induced static liquefaction and flow failure. These zones have been identified within Reaches 1 through 3 and the left banks of Reaches 8 and 9. Reduced initial dredge cuts and stepped, top-down dredging techniques are recommended to lower the risk of static liquefaction and flow failure during dredging in these reaches. It should be noted that the rock stability analyses acknowledge the risk of sloughing on potentially adverse fractures and modifications to the rock cuts are proposed in limited areas where proposed improvements could be impacted.

The methodology used to estimate the long-term *future equilibrium* side slopes seeks to identify the various processes within the channel, and to isolate those processes that do and do not drive side slope equilibration. The methodology to estimate equilibrium side slopes consists of a sequence of three key assessments:

- Bound the range of side slope angles based on the previous findings/median measured slope angles within each reach;
- Identify particular portions of the channel that could be subject to equilibration; and
- Estimate the long-term future channel behavior based on quantitative and qualitative assessment of morphological processes.

The analysis segments the channel into nine reaches, each of which has similar geotechnical and hydrodynamic conditions.

The future equilibrium side slopes vary significantly throughout the channel, with the side slopes used to predict capital dredge volumes ranging from 3:1 to 22:1, and the more conservative, flatter side slopes used to predict future O&M dredge volumes and potential impacts to adjacent infrastructure ranging from 3:1 to 32:1. The largest volume of side slope material generated is expected to occur in the Entrance Channel, where waves and strong currents can rapidly mobilize and redistribute sediment. The large volume of side slope material is due to a deep cut in the sand at a relatively shallow angle. In the estuary, the presence of rock and the reduced hydrodynamic forcing are expected to limit future side slope equilibration caused by morphological processes.

Calculation of the capital dredge volume slopes varied by reach. In Reach 1, the future equilibrium slope is based on Raaijmakers' formula; in Reach 2, the value is based on the median slope angle; in Reach 3, the value is based on measured data from 2008-2016, which shows no erosion from the top of slopes; in Reach 4, the value is based on the median slope angle; in Reaches 5 through 7, the value is based on the Fredsoe equation; in Reaches 8 and 9, the value is based on the median slope angle or the bathymetry surveys showing no erosion from the top of slopes. Reach-specific methodologies are used because each reach undergoes different morphological processes.

Calculation of the future equilibrium side slopes to estimate O&M dredge volumes and potential impacts to adjacent infrastructure was based on applying the median measured side slope at the toe of the future PA toe.

The reach locations and resultant side slopes for loose sediments are presented in Table ES-1.

Table ES-1
Range of Navigation Channel Side Slopes in Coos Bay

Reach	Location	Constructed Condition Left | Right	Future Equilibrium for Capital Dredge Volumes Left | Right	Future Equilibrium for O&M Volumes and Infrastructure Impact Analysis Left | Right
Reach 1	RM -1 – 0.1	4:1|4:1	9:1|9:1	16:1|29:1
Reach 2	RM 0.1 – 0.3	4:1|4:1	22:1|15:1	22:1|15:1
Reach 3	RM 0.3 – 0.9	4:1|4:1	4:1|4:1	22:1|15:1
Reach 4	RM 0.9 – 2.0	3:1|3:1	13:1|18:1	13:1|18:1
Reach 5	RM 2.0 – 4.5	3:1|3:1	9:1|9:1	11:1|11:1
Reach 6	RM 4.5 – 5.6	3:1|3:1	9:1|9:1	11:1|11:1
Reach 7	RM 5.6 – 6.4	3:1|3:1	13:1|13:1	15:1|32:1
Reach 8	RM 6.4 – 7.2	4:1|3:1	4:1|6:1	20:1|6:1
Reach 9	RM 7.2 – 8.2	5:1|3:1	5:1|3:1	20:1|3:1

The outer limits of the zone of equilibration of the future equilibrium side slopes have been determined for the PA. The footprint of the equilibrated side slopes beyond the constructed condition for the PA are shown in Table ES-2.

A review of side slope equilibration in Coos Bay and in the Columbia River indicates that the timescale of equilibration (years) in a sheltered estuary is equivalent to the depth (feet) by which the channel is deepened. Similarly, observations in unprotected trenches, such as the Entrance Channel, show that equilibration occurs within two months. In the inner channel, the timescale for the PA (8 ft deepening) is expected to be 8 years. The morphological models developed for and used in the assessment of side slopes in the estuary were calibrated to assume equilibration of the PA occurs over an 8-year time period. In the Entrance Channel, equilibration is expected to occur within the first two months following construction due to exposure to enhanced hydrodynamic forcing (i.e., waves and offshore currents). For planning purposes, 20% of the material in the Entrance Channel is expected to be removed during dredging and the remaining 80% occurs throughout the following year.

The side slope angles estimated herein have been used to refine dredged material quantities for the 90% design and used to estimate the footprint of side slope equilibration. These volumes and footprints associated with side slope equilibration are summarized in Table ES-2.

Table ES-2
Future Equilibrium Side Slope Volumes and Footprints

Plan	**Estimated Side Slope Volume of Capital Dredging** (million cubic yards)	**Estimated Side Slope Volume for Future Dredged Material Disposal Planning** (million cubic yards)	**Side Slope Footprint Beyond the Construction Daylight Line for EIS Impact Analysis** (acres)
PA	1.4	4.7	350

The analysis also determines the potential effects, and quantifies the potential risk, of future side slope equilibration on infrastructure within or adjacent to the channel, including the North Jetty, the South Jetty, the sunken wreck of the USACE Dredge *William T. Rossell* (the *Rossell*), the North Bay Marine Industrial Site T-dock located at approximately RM 5.55, the pile dikes located along the Jarvis Turn, the Southwest Oregon Regional Airport (SWORA), the relic trestle located downstream of approximately RM 2 and outfalls along the channel. According to the analysis, the most significant impact to the channel infrastructure is the potential for the future equilibrium side slopes, assumed to originate at the toe of the dredged channel, to undercut portions of the North Jetty. A rock apron has been proposed to mitigate the potential impacts of equilibrated slopes on the geotechnical stability of the North Jetty. On the other side of the channel, the South Jetty is not expected to be affected by side slope equilibration.

Projection of the median measured side slope from the toe of the PA channel shows intersection with the portions of the *Rossell*. However, if localized erosion did occur in the vicinity of the wreck, the vessel would be expected to settle vertically, and not to move towards the channel.

The results of the channel side slope analysis indicate that channel modification, combined with future side slope equilibration, is unlikely to affect the pile dikes. The navigation channel is naturally deep in this area, and only limited dredging is required for the PA. Because the dredge cut is expected to be shallow, future equilibration is expected to be limited. Projection of the future equilibrium side slope indicates that it will daylight more than 50 ft from the rock apron surrounding the pile dikes. Therefore, no effects are expected. However, in the event that future side slope equilibration was to cause any additional erosion in the vicinity of the pile dikes, the present state of the rock apron structure was investigated. Findings show that any additional mitigation beyond the existing rock apron is not necessary to protect the pile dikes from side slope equilibration. Also, no effects are expected at the airport due to the distance between the airport's embankment toe of slope and the daylight location of the equilibrated slopes.

RISK MANAGEMENT PLAN

Results of the investigations described in this Section 204(f)/408 Report, in the opinion of the OIPCB, show that all project effects on infrastructure and the natural environment have been managed and are minor and manageable. The Corps of Engineers, through their Section 408 and 404 reviews, will make the Federal determination whether the Proposed Alteration is environmentally acceptable and consistent with Federal policy. As is the case with the implementation of any navigation improvement project in such a dynamic physical environment and within an important and ecologically valuable estuary, there will be inherent residual risk and uncertainty associated project implementation. As such, Risk management will be a critical element of the project.

This sub-appendix presents a geotechnical evaluation of the initial dredge cuts and an assessment of the future equilibrium side slopes that may result from the Coos Bay Channel Modification Project, including the long-term geotechnical stability of the side-slopes adjacent to infrastructure. After the completion of capital dredging, side slopes will continue to evolve until they reach a stable slope angle, after which sedimentation patterns will reach a future equilibrium state. Estimating the expected side slopes is critical for the purpose of predicting the total dredge volumes that may result from channel equilibration process and for the purpose of estimating potential effects to federal infrastructure and other resources. This analysis recognizes the inherent uncertainty in predicting the future equilibrium side slope, and therefore predicts a range of side slope outcomes. Throughout the development of the Section 204(f)/408 Report, potential areas of residual risk regarding the potential for impacts from side slope equilibration area have been identified. While these potential impacts will be further evaluated in the EIS process, preliminary elements of risk identified as warranting quantitative risk management plan are summarized in Table ES-3.

Table ES-3
Risk Management Elements Related to Channel Side Slope Analysis

Issue or Concern	Primary Monitoring	Monitoring Tools	Frequency and Duration of Monitoring	Trigger(s) for Action	Possible Response Actions
North and South Jetty Stability	Bathymetric surveys	Bathymetric surveys to establish baseline Existing variability	Annually – 5-year period post construction. Periodic following major storm events.	Side slope equilibration and/or erosion beyond predicted limits and / or in close proximity to jetty structure	Temporarily suspend dredging operations; Add or enhance rock apron
Other Infrastructure Stability	Bathymetric surveys	Bathymetric surveys to establish baseline Existing variability	Annually – 5-year period post construction. Periodic following major storm events.	Side slope equilibration and/or erosion beyond predicted limits and / or in close proximity to jetty structure	Temporarily suspend dredging operations; Add or enhance rock apron or other protective measures

The Risk Management Plan will be developed based on USACE Risk Management guidance.

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ACRONYMS AND ABBREVIATIONS

2D	Two-dimensional
3D	Three-dimensional
3d HD	3D Hydrodynamic
ac	Acres
ADCP	Acoustic Doppler Current Profilers
AIS	Automatic identification system
AMD	Advanced Maintenance Dredging
ASA(CW)	Assistant Secretary of the Army for Civil Works
ATON	Aids to Navigation
BMPs	Best Management Practices
BOE	Basis of Estimate
BW	Boussinesq Wave
CBNS	Coos Bay North Spit
CDF	Confined Disposal Facility
CDIP	Coastal Data Information Program
CFR	Code of Federal Regulations
cfs	Cubic feet per second
CMOP	Coastal Margin Observation and Protection
CMS	Coastal Modeling System
CRA	Cost Risk Analysis
CSZ	Cascadia Subduction Zone
CWA	Clean Water Act
cy	Cubic yards
cy/yr	Cubic yards per year
CZMA	Coastal Zone Management Act
DBB	Design-Bid-Build
DDR	Design Documentation Report
DEA	David Evans and Associates, Inc.
DHI	Danish Hydraulic Institute
DMMP	Dredged Material Management Plan
DOGAMI	Oregon Department of Geology and Mineral Industries
DTM	Digital Terrain Model
EC	Engineering Circular
EIS	Environmental Impact Statement
ENSO	El Niño/Southern Oscillation
ER	Engineer Regulations
ERDC	Engineer Research and Development Center
ESA	Endangered Species Act
ETL	Engineer Technical Letter
FAA	Federal Aviation Administration
FERC	Federal Energy Regulatory Commission
FM	Flexible Mesh
FM HD	Flexible Mesh Hydrodynamic
FNC	Federal Navigation Channel
FR	Federal Register
ft	Foot or feet
FY	Fiscal Year

ACRONYMS AND ABBREVIATIONS

gpm	Gallons per minute
GRI	Geotechnical Resources, Inc.
HCSS	Heavy Construction Systems Specialists
HOWL	Highest Observed Water Level
HRA	Habitat Restoration Area
HSE	Health, safety and environment
IG	Infragravity
ILS	Instrument Landing System
in.	Inches
IWP	Industrial Waste Pond
JCLNG	Jordan Cove LNG Export Facility
lf	Linear feet
LiDAR	Light Detection and Ranging
LNG	Liquefied natural gas
LNGC	Liquefied natural gas carrier
LOA	Length Overall
LSB	Log-spiral Bay
LST	Longshore Transport
M&N	Moffatt & Nichol
MCR	Mouth of the Columbia River
MCX	Mandatory Center of Expertise
mcy	Million cubic yards
MHHW	Mean Higher High Water
MHW	Mean High Water
mi	Miles
MLLW	Mean Lower Low Water
MLW	Mean Low Water
mm	Millimeters
MMR	Major Maintenance Report
MOF	Material Offloading Facility
MPRSA	Marine Protection, Research, and Sanctuaries Act
MSL	Mean Sea Level
MTL	Mean Tide Level
MTO	Material takeoffs
NAIP	National Agricultural Imagery Program
N	SPT Blow Counts
NAVD88	North American Vertical Datum of 1988
NDBC	National Data Buoy Center
NED	National Economic Development
NEPA	National Environmental Policy Act
NGDC	National Geodetic Data Center
NM	Nautical Mile
NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Service
NRC	National Research Council
NTPro	Navi Trainer Pro 5000

ACRONYMS AND ABBREVIATIONS

NW	Northwest
O&M	Operations and Maintenance
OCMP	Oregon Coastal Management Program
ODEQ	Oregon Department of Environmental Quality
ODLCD	Oregon Department of Land Conservation and Development
ODMDS	Ocean Dredged Material Disposal Site
ODSL	Oregon Department of State Lands
OESA	Oregon Endangered Species Act
OGMT	Oregon Gateway Marine Terminal
OIPCB or Port	Oregon International Port of Coos Bay
OPC	Opinion of probable costs
OPRD	Oregon Parks and Recreation Department
OSU	Oregon State University
PA	Proposed Alteration
Pcf	Pounds per Cubic Foot
Psf	Pounds per Square Foot
POT	Peak-Over-Threshold
PRG	Project Review Group
PRG	Project Review Group
psi	pounds per square inch
PSU	Practical salinity unit
QC	Quality control
RAO	Response Amplitude Operators
RFP	Roseburg Forest Products
RM	River mile
RMS	Root-mean-squared
ROD	Record of Decision
SDPP	South Dunes Power Plant
SEF	Sediment Evaluation Framework
SELFE	Semi-implicit Eulerian-Lagrangian Finite Element
SHPO	Oregon State Historic Preservation Office
SL	Screening levels
SLC	Sea level change
SLR	Sea-level Rise
SMMP	Site Management/Monitoring Plan
SOORC	Southern Oregon Ocean Resource Commission
SPT	Standard Penetration Test
SSE	Safe Shutdown Earthquake
SW	Spectral Wave
SWORA	Southwest Oregon Regional Airport
TCX	Technical expertise
the “Project”	Coos Bay Section 204(f) Channel Modification Project
TIN	Triangular irregular networks
TSP	Tentatively Selected Plan
U.S.	United States
USACE	U.S. Army Corps of Engineers
USBLM	U.S. Bureau of Land Management

ACRONYMS AND ABBREVIATIONS

USC	United States Code
USCG	U.S. Coast Guard
USDA	U.S. Department of Agriculture
USEPA	U.S. Environmental Protection Agency
USFWS	U.S. Fish and Wildlife Service
USGS	U.S. Geological Survey
USGS	U.S. Geological Survey
VFR	Visual flight rules
WIIN	Water Infrastructure Improvements for the Nation
WNW	West-northwest
WOP	Without Project
WRDA	Water Resources Development Act
WRRDA	Water Resources Reform and Development Act
WSP	Western Snowy Plover
WSW	West-southwest

1. INTRODUCTION

The Oregon International Port of Coos Bay (OIPCB or Port) is home to the second largest deep-draft coastal harbor between San Francisco and the Puget Sound, based on the tonnage of cargo transported through the Port. Access to the Port's facilities is provided by the Coos Bay Federal Navigation Channel (FNC), a federal channel that was first dredged in the early 1900s. The channel was last improved in 1998, when the channel was deepened by 2 feet (ft) from 35 ft to 37 ft. Since 1998, vessels calling at the Port have substantially increased in size.

This report presents the results of a geotechnical and morphological study describing the side slope angles that are expected to be stable during and after non-Federal dredging, and how the side slopes will equilibrate in the years following capital dredging. The results of this analysis serve as a basis for capital and long-term dredge quantity estimates, sizing of dredge material disposal areas, hydrodynamic modeling, cost sharing, and assessment of effects.

1.1 Overview

The OIPCB proposes a Pacific Coast Intermodal Port (PCIP) project at Coos Bay, Oregon. The PCIP consists of integrated elements that would link freight arriving by container ship to the Port to Class 1 rail networks in Oregon. The in-water component of the project includes the deepening and widening of the existing FNC for deep-draft container vessels. In support of that work, the Port is conducting economic, engineering, and environmental studies preparatory to improving the Federal navigation project. These investigations are being conducted under the authority granted by Section 204 of the Water Resources Development Act (WRDA), 1986, as modified by Section 1014 of the Water Resources Reform and Development Act (WRRDA), 2014. This action will require approval by the U.S. Army Corps of Engineers under Section 14 of the Rivers and Harbors Appropriation Act of 1899, 33 United States Code 408, to modify the Federal navigation project. The Section 204/408 Report and Environmental Impact Statement (EIS) will propose modifications to the Coos Bay Navigation Channel in Coos County, Oregon, to accommodate larger deep draft vessels and provide local, state, and federal economic benefits. The USACE, Portland District is presumed to be the lead federal agency for the EIS in cooperation with the U.S. Department of Transportation's Federal Rail Administration.

1.2 Study Area Description

Coos Bay is located in Coos County, Oregon, on the southern Oregon coast, about 200 miles (mi) south of the mouth of the Columbia River (MCR) and 445 mi north of San Francisco Bay. It is the navigational approach to Charleston, Empire, North Bend, Glasgow, Coos Bay, and Eastside (Figure 1-1 and Figure 1-2). The bay is formed by the junction of Isthmus Slough, Catching Slough, Coos River, Kentuck Slough, Haynes Inlet, South Slough, and Winchester Creek, and is located at the foot of the Coast Range. Deep-draft navigation is limited to the lower 15 mi of the estuary.

The surface area of the Coos Bay estuary is about 12,000 acres (ac) (about 19 square mi). Tidelands, located from River Mile (RM) 0 through 15 comprise 20 percent to 30 percent of the estuary area. The inlet to the estuary, referred to as the Entrance Channel, is fully exposed to waves.

The Coos Bay estuary drains directly into the Pacific Ocean. The nearshore zone adjacent to the Entrance Channel is composed of fine- to medium-grained sediments and intermittent rock outcroppings. The coastal shelf within 8 mi of the inlet has a roughly 100:1 (Horizontal:Vertical) slope. Cape Arago, a headland that limits sediment transport and marks the southern boundary of the littoral cell, is located 2.5 mi south of the inlet.

The topography of the lower Coos River area is a combination of rugged mountain terrain, extensive sand dunes adjacent to the ocean, and relatively flat pasture land along the river. The terrain of the area is quite rugged, because the mountains are relatively young, denoted by the typical narrow, sinuous valleys and steep side slopes. Relief varies from sea level to just under 3,000 ft; however, most of the land lies between 500 ft and 1,500 ft in elevation.

Geotechnical investigations indicate the subsurface conditions in the channel typically vary from relatively clean sand to siltstone and sandstone sedimentary rock. The sedimentary rock is present near the mudline from about RM 2 to RM 6 and at Guano Rock from about RM 0.7 to RM 0.9.

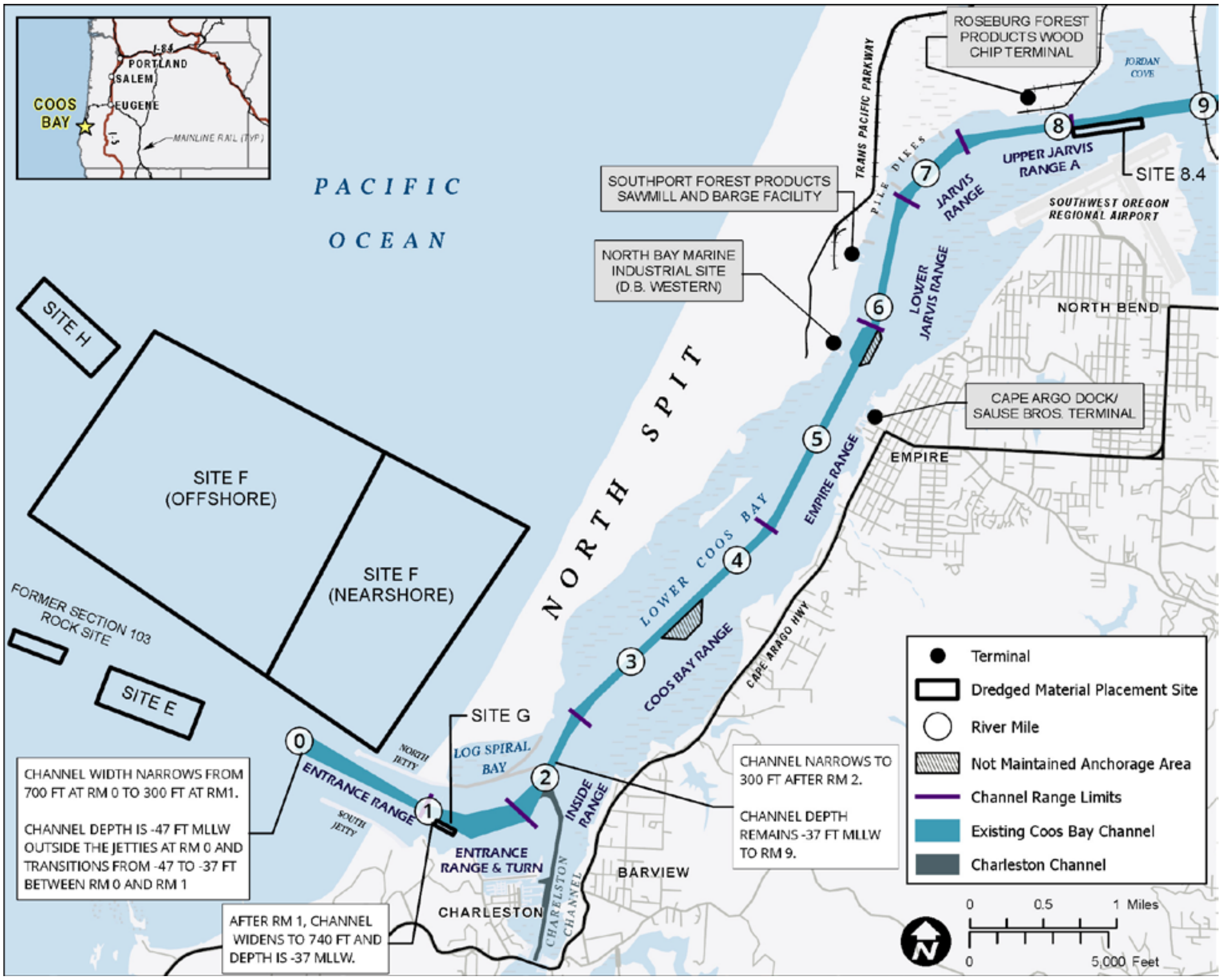


Figure 1-1
Coos Bay Project Vicinity Map, Lower Bay

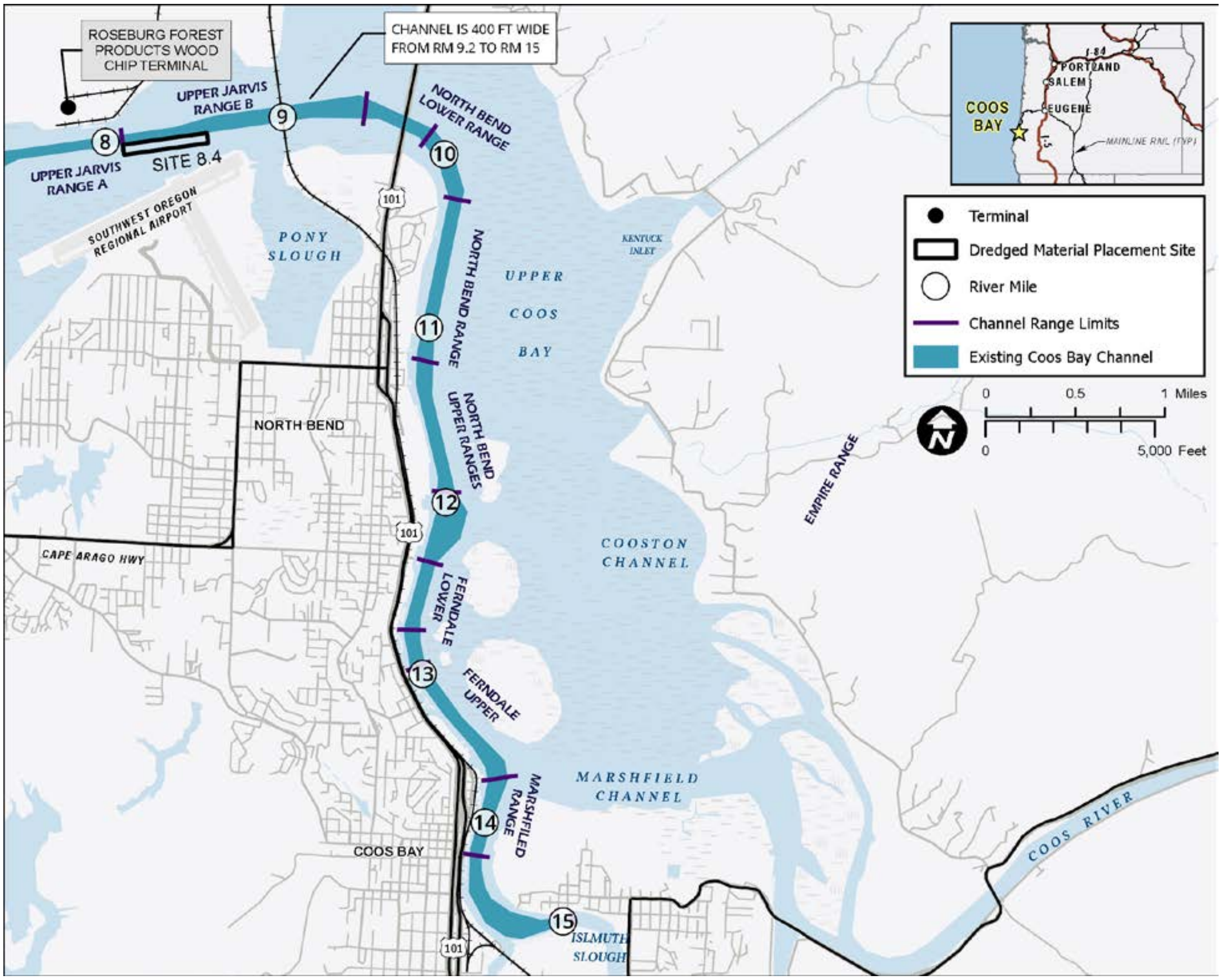


Figure 1-2
Coos Bay Project Vicinity Map, Upper Bay

1.3 Existing Navigation Project

The Coos Bay Federal Navigation Project was first authorized by the Rivers and Harbors Appropriation Act of March 3, 1899, and has been subsequently modified in 1919, 1937, 1951, 1952, 1979, and 1998. The 1979 project represents the completion of the 1970 authorized which allowed the USACE to deepen and maintain the Entrance Channel at -45 ft Mean Lower Low Water (MLLW) and the inner channel to -35 ft MLLW. The most recent project modification was authorized in the fiscal year (FY) 1996 Energy and Water Development Appropriations Act, Public Law 104-46, which provided for deepening the channel by 2 ft to -47 ft MLLW from the ocean entrance to Guano Rock at RM 1, and to -37 ft MLLW from RM 1 to RM 15. Public Law 104-46 also provided for deepening the turning basin at RM 12 by 2 ft and expanding it by 100 ft, from 800 ft by 1,000 ft to 900 ft by 1,000 ft.

The U.S. Army Corps of Engineers (USACE) Federal Navigation Project consists of the following federally authorized elements:

- North Jetty (9,600 ft long) and South Jetty (3,900 ft long), located on either side of the Entrance Channel, including the two relic structures that extend from the root of the North Jetty, one of which extends into Log-spiral Bay (LSB) and the other of which extends into the estuary.
- An Entrance Channel with an authorized depth of -47 ft MLLW, which decreases from a width of 700 ft at RM 0 to a width of 300 ft at RM 1.
- An inner channel (from RM 1 to RM 15) that has an authorized depth of -37 ft MLLW, a width of 300 ft from RM 1 to RM 9, and a width of 400 ft from RM 9 to RM 15.
- Two (2) turning basins, both of which are 1,000 ft long. The first is located at RM 12, and has a width of 900 ft. The other, located at RM 14, has a width of 730 ft. Both have a depth of -37 ft MLLW, consistent with the channel depth.
- Five (5) pile dikes between RM 6.4 and RM 7.3 in the main channel.
- Continuation of the main channel beyond RM 15 (in the Isthmus Slough) with a width of 150 ft and a depth of -22 ft MLLW.
- A 150-ft-wide Charleston Access Channel that has a depth that varies from –17 to -14 ft MLLW.
- A breakwater and bulkhead at Charleston.
- Charleston Small Boat Basin (10 feet deep) constructed by USACE in 1956 and maintained by the OIPCB.
- Advanced maintenance dredging (AMD) of the channel extends offshore to RM -0.55, where the width of maintenance is 1,060 ft. Authorized AMD is 5 ft of depth in the Entrance Channel (RM 0.55 to RM 1) and 1 ft of depth upstream of RM 1.

The USACE maintains the above elements to provide navigational access to Coos Bay. USACE maintenance of the main navigation channel and jetty features provides ongoing deep-draft navigation access to Coos Bay.

1.4 Description of the 2023 Proposed Alteration (2023 PA)

To accommodate larger deep draft vessels and provide local, state, and federal economic benefits, the Port proposes navigation channel improvements to the Coos Bay Navigation Channel. These proposed channel improvements are hereinafter referred to as the 2023 Proposed Alteration (2023 PA) and they are summarized as follows:

- *Coos Bay Inside Range*: the channel from RM 1.3 to RM 2.8 on the red side of the channel was widened. The range heading of the Coos Bay Inside Range was changed by 1° from 28.0° - 208.0° to 27.0° - 207.0°.

- *Bend Widener at RM 4.0*: a bend widener was included in the 2023 PA to add an additional 50 ft on the green side in the turn from Coos Bay Range to Empire Range.

- *Post Panamax Generation 3 (PPX3) Containership Turning Basin at RM 5.0*: a larger turning basin at the container facility is needed to accommodate the PPX3 containership. Based on the vessel's dimension, the proposed turning basin is 2,000 feet long (parallel to the channel) and 1,600 feet wide. The turning basin's design bottom elevation is -45 ft MLLW, the same as the 2023 PA channel.

- *Capesize Turning Basin at RM 8.0*: a Capesize turning basin was added at RM 8.0 to replace the turning basin that was removed at RM 7.5. Operationally, this turning basin will be used by inbound empty vessels. Therefore, the turning basin's design bottom elevation is -37 ft MLLW. The deeper navigation channel (450-ft wide at -45 ft MLLW) continues through the length of the turning basin.

The above improvements are shown in Table 1-1 and Table 1-2; no dredging is proposed beyond the boundaries in these tables. The project vicinity is represented graphically in Figure 1-3. In this figure, the channel is labeled by RM. Figure 1-3 also shows the location of the adjacent federal infrastructure: the two jetties that run parallel to the channel from RM 0 to RM 1 and the pile dikes located along the north bank of the channel from RM 6.4 to RM 7.5.

Table 1-1
Channel Footprint for Existing Authorized Project and 2023 PA

Range(s) and RM	Existing Conditions	2023 PA
	Offshore Extent	
Offshore Limit including Advanced Maintenance Dredging	RM -0.55[1]	RM -1
Offshore Limit of Navigation Channel	RM 0[1]	RM -0.9

Range(s) and RM	Existing Conditions	2023 PA
	Channel Width (ft)	
Offshore Inlet Offshore Limit of Navigation Channel to RM 0.3	700 narrowing to 550	1,280 narrowing to 600
Entrance Range RM 0.3 to 1.0	550 narrowing to 300	600
Entrance Range RM 1.0 to 2.0 and Turn	Varies up to 740	Varies up to 1,140
Inside Range RM 2.0 to 2.5	300	650 narrowing to 550
Coos Bay Range RM 2.5 to 4.3	300	450
Empire Range RM 4.3 to 5.9	300	450
Post Panamax Generation 3 Turning Basin RM 4.7 to 5.6	None	2,000 x 1,600
Lower Jarvis Range RM 5.9 to 6.8	300	450
Jarvis Turn RM 6.8 to 7.3	400	500
Upper Jarvis Range RM 7.3 to 8.2	300	450
Capesize Turning Basin RM 7.6 to 8.0	None	1,400 × 1,025

Notes:

1. The authorized FNC starts at RM 0. However, advanced maintenance dredging (AMD) occurs further offshore, typically from the channel entrance to RM -0.55. The channel width at RM -0.55 is approximately 960 ft.

Table 1-2
Channel Depth for Existing Authorized Project and 2023 PA

	Navigation Bottom Elevation (ft, MLLW)		Advance Maintenance Dredging[1] (ft)	
Range(s) and RM	**Existing Conditions**	**2023 PA**	**Existing Conditions**	**2023 PA**
Offshore Inlet Offshore Limit of Navigation Channel to RM 0.3	-47	-57	5	6
Entrance Range RM 0.3 to 1.0	-47 decreasing to -37[2]	-57 decreasing to -45[3]	Varies 5 to 1[4]	Varies 6 to 1[5]
Entrance Range and Turn RM 1.0 to 2.0	-37	-45	1	1
Inside Range RM 2.0 to 2.5	-37	-45	1	1
Coos Bay Range RM 2.5 to 4.3	-37	-45	1	1
Empire Range RM 4.3 to 5.9	-37	-45	1	1
Post Panamax Generation 3 Turning Basin RM 4.7 to 5.6	None	-45	None	1
Lower Jarvis Range RM 5.9 to 6.8	-37	-45	1	1
Jarvis Turn RM 6.8 to 7.3	-37	-45	1	1
Upper Jarvis Range RM 7.3 to 8.2	-37	-45	1	1
Capesize Turning Basin RM 7.6 to 8.0	None[6]	-37[6]	None	1

Notes:

1. Capital dredging consists of the navigation depth plus AMD plus a rock buffer plus a portion of overdepth.
2. For the existing channel, the navigation depth decreases from a depth of -47 to -37 ft MLLW between RM 0.4 and RM 0.7. The channel is dredged farther offshore to obtain AMD depth.
3. For the 2023 PA, the navigation depth decreases by 12 ft between RM 0.3 (depth of -57 ft MLLW) and RM 1.0 (depth of -45 ft MLLW).
4. AMD of 5 ft starts at the offshore daylight line, approximately RM -0.6, and continues to RM 0.7.
5. AMD of 6 ft starts at the offshore daylight line. The AMD will be 1 ft in areas near Guano Rock (RM 0.7 to RM 1).
6. Under the Existing Conditions, there is no formal turning basin; vessels that visit Roseburg Forest Products turn in existing deeper water at this location. Under the 2023 PA, incoming vessels will enter the channel and turn under ballast load, so it is not necessary to dredge beyond a depth of -37 ft MLLW.

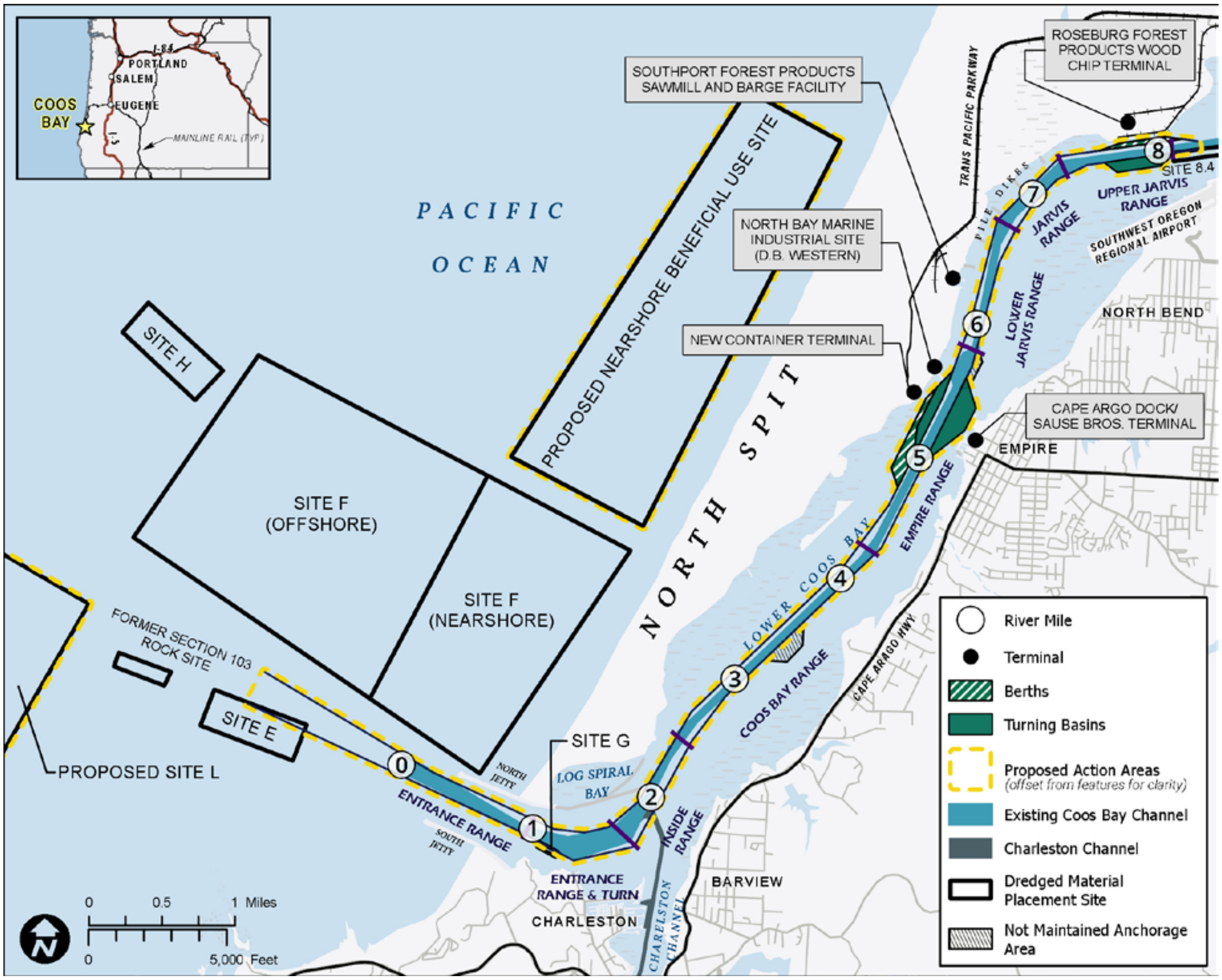


Figure 1-3
Summary of the 2023 Proposed Alteration

1.5 Previous Studies

From 2016 to 2019, the Port evaluated alternatives for modifications to the Coos Bay Federal Navigation Project in support of a previous proposal. In support of that effort, M&N prepared 19 substantial works of engineering and design, economics, modeling, and construction planning. The USACE, Portland District comprehensively reviewed and evaluated the entirety of the Port's proposals as reflected in their Main Report and all appendices (OIPCB 2019).

1.6 Objective

This report supports the design of the Coos Bay Channel Modification Project by describing the results of an assessment of the side slopes adjacent to the Federal Navigation Channel (FNC) Project as modified to reflect the PA. The purposes of this report are to:

- Evaluate the geotechnical stability of the initial proposed dredge cut slopes during and at the completion of capital dredging;
- Estimate the range of slope angles to which the side slopes will equilibrate following the conclusion of capital dredging;
- Estimate the anticipated duration for the side slope equilibration process;
- Estimate the range of dredge quantities associated with side slope equilibration for the PA for the purpose of calculating project costs and cost sharing;
- Estimate the physical zone of equilibration of the channel modifications following the equilibration process; and
- Evaluate the effect of the future equilibrium side slopes on adjacent resources.

1.7 Report Organization

Report organization is as follows:

- **Section 2** describes the environmental conditions at Coos Bay, focusing on the substrate (i.e., sediment and rock) and the hydrodynamics (currents and waves). The purpose of this section is to divide the channel into "reaches" that are geotechnically and hydrodynamically similar; these reaches then form the basis for the analysis of side slopes at Coos Bay.
- **Section 3** introduces the geotechnical methodology to assess the stability of the initial dredge cut, definition of equilibrium side slopes, the methodology used to estimate equilibrium side slopes and the various morphological processes that occur within Coos Bay, and the geotechnical methodology to assess the stability of the equilibrium side slopes adjacent to key infrastructure. For the processes that drive side slope equilibration, methodologies to predict this equilibration are included.
- **Section 4** applies the methodology described in Section 3 to the channel reaches described in Section 2. Section 4.1 specifically describes the geotechnical conclusions and recommendations resulting from the geotechnical stability assessment of the initial dredge cut. Section 4.2 describes the side slope estimates developed within Coos Bay. This includes an overview of the data used, and a reach-by-reach breakdown of the methodologies introduced above to estimate the range of side slopes and the expected

equilibrium side slopes. Section 4.3 provides a summary table of the side slope analysis results.

- **Section 5** explains how the calculated side slopes were used to develop a 3D model that could be used for further analyses. The slope angles were calculated to fit the bulk properties of each reach. However, due to discontinuities in the bathymetric data and heterogeneities in the subsurface such as rock outcroppings, the slopes presented above cannot be applied uniformly within each reach. The discussion presented in this section explains how the slope angles were used to develop a 3D model that could be used for further analyses.
- **Section 6** describes using the 3D model to calculate shoaling volumes as a result of slope equilibration.
- **Section 7** defines the zone of equilibration and summarizes the potential effects to infrastructure, such as, the North and South Jetties, the *Rossell*, and the pile dikes.
- **Attachment A** presents the global slope stability locations and output files from the geotechnical stability assessment of the future equilibrium side slopes adjacent to key infrastructure.
- **Attachment B** presents the charts used to support the static liquefaction analyses; an example calculation using the Van den Ham assessment; an evaluation of the sensitivity of the Raaijmakers, Stoutjesdijk et al., and Van den Ham assessments to variations in the assumed parameters.

2. ENVIRONMENTAL CONDITIONS

The geotechnical and morphological processes that govern side slope equilibrium depend on the geological and hydrodynamic properties of the channel. The sediment characteristics and the presence of underlying rock define the channel's ability to remain stable under different forcing conditions. The primary forcing mechanism in the channel are the currents at the channel bed, which are driven by local hydrodynamics and surface waves.

2.1 Coos Bay Sediment

Most of the sediment within the Coos Bay estuary, up to RM 8, is of marine origin, largely sand and silts. The Coos River watershed consists of terrain that is composed primarily of sandstone and siltstone. The resulting sediment yields are primarily alluvial sand, silt, and clay, which make up the bottom material of the upper bay, upstream of the potential project location. Towards the mouth of the bay, the sediment characteristics shift to sand and shell fragments of a marine origin.

Various studies have been performed to quantify and characterize sediment throughout the channel. These studies include USACE (2005), SHN (2007), USACE (2009), USACE (2015), and the studies included in Sub-Appendix 5 – Geotechnical Data. Sediment characterization reports are summarized in the Level 1 Site History Evaluation Report (OIPCB 2018), which provides a complete description of sediment and rock within the channel; sampling results are summarized in Figure 2-1. In this figure, labels correspond to median grain size (d_{50}, μm) and the percent of fines (silts and clays); for some measurements, only one of these metrics is available.

These studies show that sediment throughout the channel is generally medium grained sand. Borings occasionally indicated the presence of silt and peat; however, these borings did not show a consistent pattern of these materials in the channel. For the morphological portion of the analysis, the future equilibrium side slope calculations assume that fine- to medium-grained sand (d_{50} of 0.2 to 0.5 mm) is present throughout, as the equations used for these analyses are based on sediment only (Section 3.2.4). The future equilibrium side slopes are based on measured data and therefore include any peat present in those slopes (Section 3.2.1). The geotechnical portion of the analysis does assume the presence of the peat layer as described in Section 3.

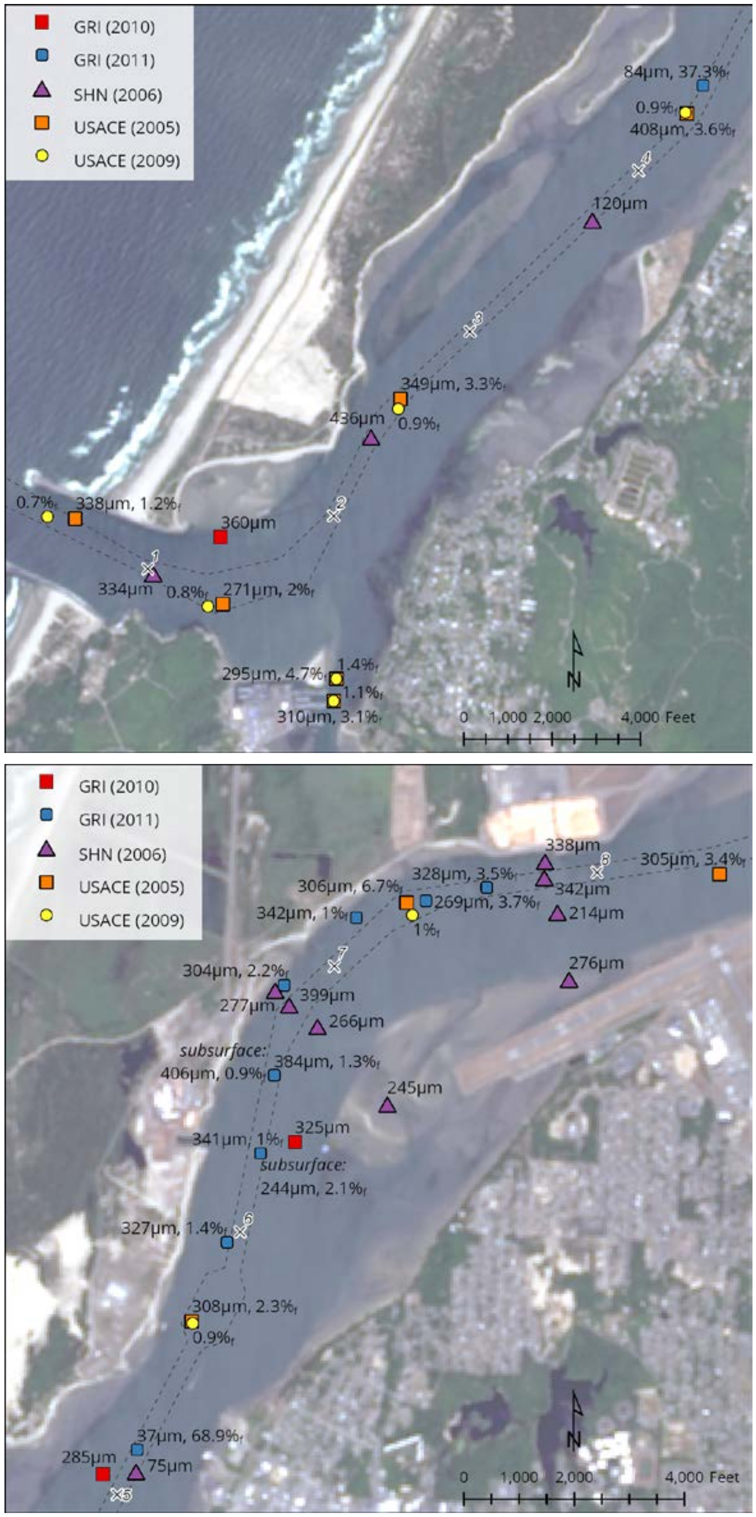


Figure 2-1
Sediment Sampling Results in Coos Bay: Entrance Channel (Lower) and Jarvis Turn (Upper)

2.2 Rock

Geophysical surveys and geotechnical explorations have indicated that the channel modification will include dredging in both rock and sediment. These studies are detailed in Appendices 2 and 5. Because rock is not subject to the same geotechnical and morphological processes as sediment, side slopes cut into rock are expected to be stable; side slope equilibration would begin above the top of the bedrock cut. Similarly, in instances where the side slopes overlie rock with less than 5 ft of sand overburden, significant side slope equilibration is not anticipated.

Several portions of the channel are directly underlain by rock. The previous channel deepening project in 1996 dredged directly into rock from RM 2 through RM 5.2 across the channel and along the side slopes. Along this stretch the side slopes are governed by this rock cut. The geophysical investigations performed by DEA in 2008 and 2016 show that the rock surface directly beneath the channel and the side slopes extends as far upstream as RM 5.5; in these areas, a portion of dredging will consist primarily of rock.

The following geotechnical investigations were performed, and are described in the Sub-appendix 5 – Geotechnical Data:

- GRI in 2010, 2016 and 2023
- Geo Recon International in 1989, 1997, and 2010
- USACE in 1974, 1994a, 1995, 1996

The depth of the top of rock can be seen in Figure 2-2; rock contour maps are also provided in Sub-appendix 8 – Construction Drawings. In the Entrance Channel (offshore of RM 0.9), the top of the rock is located within 5 to 10 ft of the authorized depth of the existing channel. Equilibration may be somewhat limited in this area due to the presence of rock. The rock surface deepens at the Entrance Turn (RM 0.9 to RM 2). From RM 2 to RM 5.5, the rock surface is located at a depth equivalent to the existing channel bottom. Rock is present between RM 5.5 and RM 6.5, though it is deeper than the authorized channel depth. Accordingly, side slope adjustment of the adjacent sediment may occur in this reach.

Capital dredging under the PA will increase the portion of channel that is directly underlain by rock. Therefore, while many of the measured slope angles can be used to estimate how cuts in sediment would respond to dredging, the equilibrium side slopes of the PA in these areas will be governed by the rock surface.

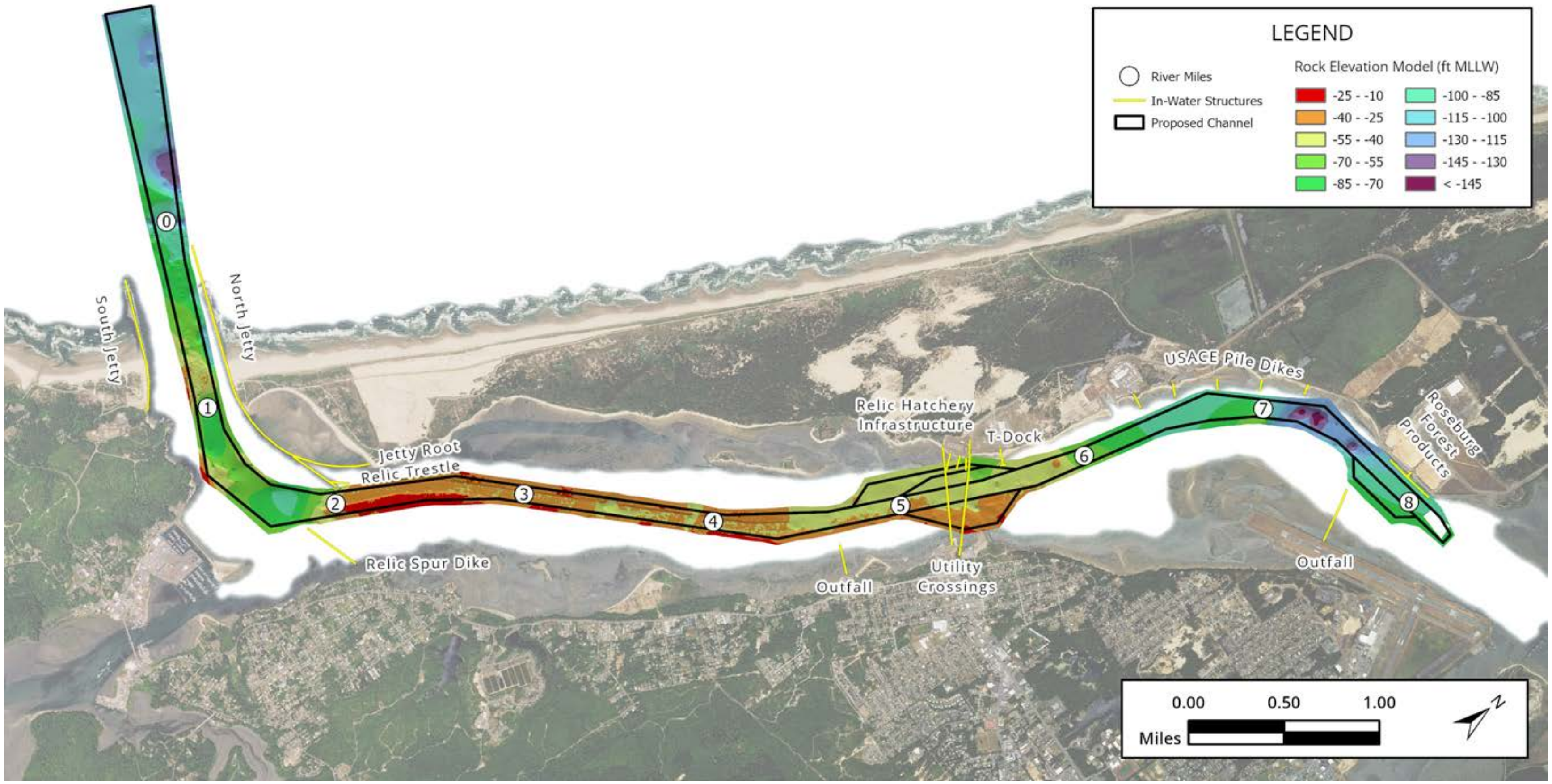


Figure 2-2
Depth of Rock in Coos Bay

2.3 Hydrodynamic Conditions

The Coos Bay channel is in an estuary at the mouth of the Coos River on the Pacific Ocean. As such, circulation within the estuary is driven by a complex combination of forces including riverine input, runoff, tides, waves, currents, and differences in water density.

Offshore of the jetties (through RM 0.1), the hydrodynamics are dominated by winds, tides, waves, and longshore currents, as described in Sub-appendix 4 (*Offshore and Ocean Entrance Dynamics Report*).

Most offshore waves originate from a westerly and northwesterly direction (prevailing direction). The winter storms have two directional peaks: the majority of waves approach from west to west-northwest, and there is a secondary peak from the southwest. The west to west-northwest waves are long period swell waves with periods on the order of 16 to 20 seconds generated by distant storms, while the southwest waves originate from nearby storms with periods generally less than 15 seconds. This southwest peak accounts for the highest storm waves. Extreme waves are regularly observed at Coos Bay; during the winter months, offshore wave heights exceed 10 ft more than 40% of the time. As the largest waves approach the jetty, they refract towards the jetty tips and become depth limited.

Shore parallel currents, which run perpendicular to the channel orientation, are present offshore of the jetties. Far offshore, the current velocity is generally on the order of 1 ft per second (fps), and rarely exceeds 2 fps (USACE 2012a). Immediately offshore of the jetties, tidal currents become significant; these currents are parallel with the channel between the jetties, and generally curve as they exit the jetties. Nearshore velocities perpendicular to the channel can reach 3 fps (USACE 2012a). Sediment is flushed from the channel during ebb currents and may be drawn into the channel during flood currents.

Between the jetties, (RM 0.3 to RM 1), the sediment transport processes are dominated by waves and tidal currents. Waves are able to penetrate well into the entrance; waves up to 10 ft have been observed in the vicinity of RM 1. Figure 2-3 shows where wave crests were present inside the jetties on December 10, 2005. In this figure, the waves are most prominent along the South Jetty, breaking along the inner portion of the jetty and near Guano Rock.

Figure 2-3
Wave Penetration into the Entrance Channel

The hydrodynamics of the entrance turn (RM 0.9 to RM 2.1) continues to be dominated by tidal currents, albeit of a slightly smaller magnitude than at the jetties. Hydrodynamically, this segment differs from the region between the jetties due to the nearly 90-degree turn (the Entrance Turn). In the vicinity of a river bend such as this, deposition typically occurs on the convex (inner) bank, while erosion and undercutting typically occur on the concave (outer) bank. In addition, smaller waves are able to penetrate upstream to approximately RM 2.5.

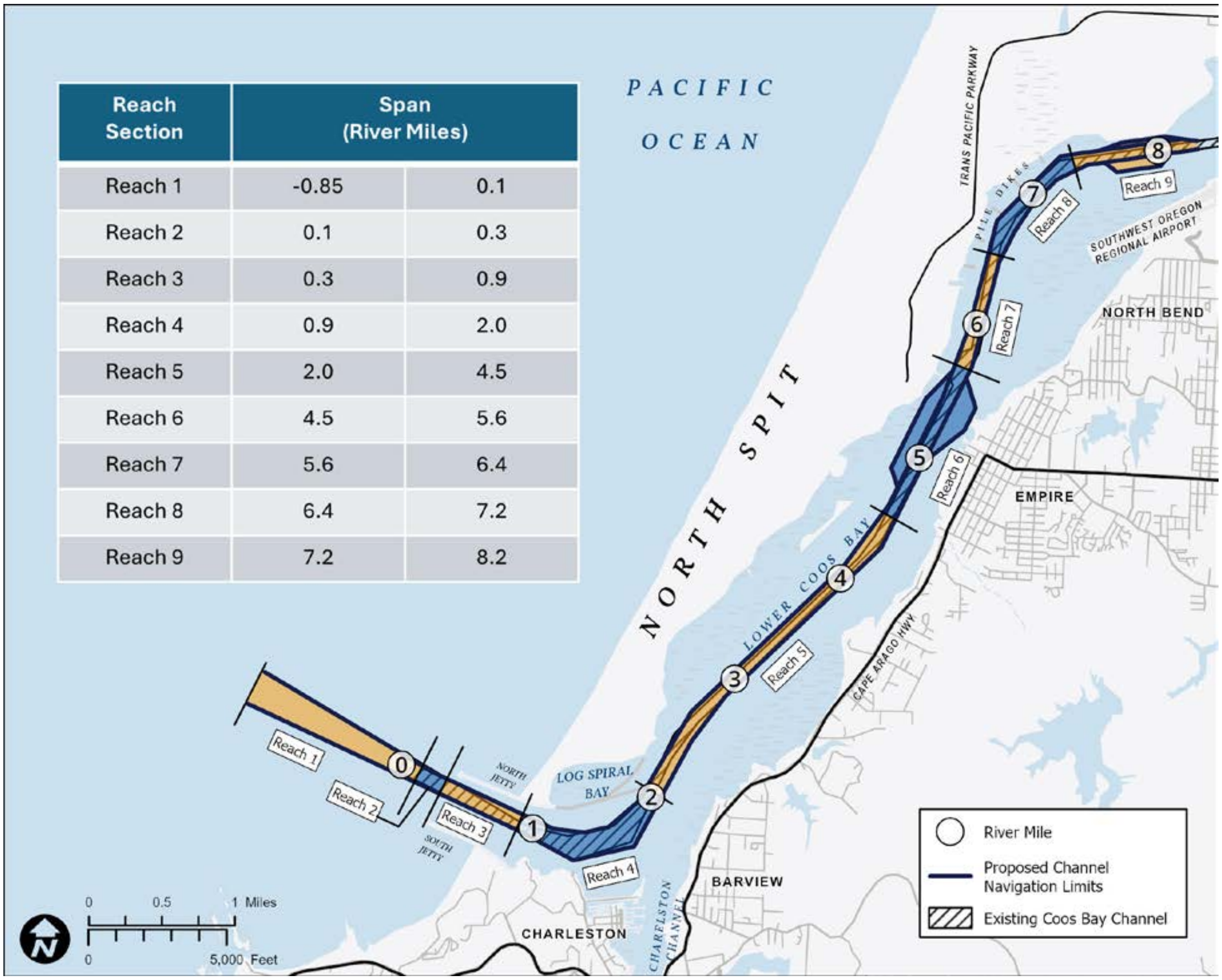
Upstream of RM 2.1, the hydrodynamics are dominated by tidal and river currents. Currents are fairly consistent along this length, with median currents ranging from 1.5 to 2 fps, depending on the local bathymetry. The estuary widens upstream RM 5.5; the wider area can carry an equivalent volume of water with a smaller velocity. Therefore, current velocities are greater downstream of RM 5.5. As noted above, small waves penetrate into the downstream portion of this reach (up to RM 2.5).

Jarvis Turn (RM 6.5 to 8.4) is a wide, approximately 80-degree turn. Along this turn, current velocities are greater at the outer bank than at the inner bank, leading to patterns of erosion and deposition, respectively.

2.4 Channel Reach Designations

As explained above, the hydrodynamics and geology vary along the length of the Coos Bay channel. This analysis separates the channel into nine reaches, each of which is hydrodynamically/ geologically distinct. Side slope equilibration and stability is generally a function of the local hydrodynamics and geology. Therefore, because the hydrodynamic and geologic properties are generally consistent within each reach, the geotechnical and morphological processes that drive side slope equilibration are similarly expected to be consistent (see further discussion in Section 3.2.2). These reaches can be seen in Figure 2-4 and are described below:

- Reach 1 is defined as RM -1 through RM 0.1. It is located offshore of the jetty heads, where the morphology is dominated by longshore currents that run perpendicular to the channel alignment, carrying suspended sediment and bed load.
- Reach 2 is defined as RM 0.1 to RM 0.3, just between the two jetty tips. This reach is influenced by a cross current and by flow over the North Jetty relic head; this latter feature creates a scour hole on the north side of the channel.
- Reach 3 extends from RM 0.3 to RM 0.9 and is located between the jetties. A combination of tidal currents and the gyre described above transport offshore sediment into this reach, leading to significant shoaling.
- Reach 4 encompasses the Entrance Turn, from RM 0.9 to RM 2.0. The curved nature of this reach leads to a velocity distribution where the currents are generally faster on the outer bend than on the inner bend, which would lead to patterns of erosion and deposition, respectively. However, this reach is very deep (deeper than -60 ft MLLW in some areas), and patterns of erosion and shoaling have not been observed from a review of past bathymetry surveys.
- Reach 5 extends from RM 2.0 to RM 4.5, where the navigation channel is completely underlain by rock. Therefore, this reach is not expected to undergo side slope equilibration.
- Reach 6 extends from RM 4.5 to RM 5.6, where tidal currents flow parallel to the channel. In this reach, side slope equilibration occurs due to the gravity-driven downward component of bed load transport, which moves sediment from the top of the slope to the toe of the slope.
- Reach 7 extends from RM 5.6 to RM 6.4. It is similar to Reach 6 in as much as tidal currents are parallel to the channel. Once again, side slope equilibration occurs due to sediment moving from the top of slope to the toe of the slope due to the gravity component of transport. The channel is wider, which reduces the currents.
- Reach 8 extends from RM 6.4 to RM 7.2 and includes Jarvis Turn. Along Jarvis Turn, the channel migrates northwards (towards the outer bank), which leads to erosion of the North Spit and shoaling adjacent to the airport. This erosion, while still ongoing, appears to have been reduced by the installation of pile dikes; the pile dikes are described in detail in Sections 4.2.9 and 7.7.
- Reach 9 extends from RM 7.2 to RM 8.2. The inner bank is depositional, similar to Reach 8. The outer bank on the North Spit is expected to be maintained by the existing Roseburg Forestry Products.



Reach Section	Span (River Miles)	
Reach 1	-0.85	0.1
Reach 2	0.1	0.3
Reach 3	0.3	0.9
Reach 4	0.9	2.0
Reach 5	2.0	4.5
Reach 6	4.5	5.6
Reach 7	5.6	6.4
Reach 8	6.4	7.2
Reach 9	7.2	8.2

Figure 2-4
Reach Designations Used in side slope Analysis

3. METHODOLOGY FOR CHANNEL SIDE SLOPE ANALYSIS

This section presents the methodologies for the individual evaluations completed as part of the channel side slope analysis. As described previously, the methodology for the channel side slope analysis is based on the following objectives:

- Evaluate the geotechnical stability of the initial proposed dredge cut slopes during and at the completion of capital dredging;
- Estimate the range of slope angles to which the side slopes will equilibrate following the conclusion of capital dredging;
- Estimate the anticipated duration for the side slope equilibration process;
- Estimate the range of dredge quantities associated with the channel side slope equilibration for the PA for the purpose of calculating project costs and cost sharing;
- Estimate the physical zone of equilibration of the channel modifications following the equilibration process; and
- Evaluate the effect of the equilibrium side slopes on adjacent resources.

This report presents a geotechnical evaluation of the initial dredge cuts and an assessment of the future equilibrium side slopes that may result from the Coos Bay Channel Modification Project, including the long-term geotechnical stability of the side-slopes adjacent to infrastructure. After the completion of capital dredging, side slopes will continue to evolve until they reach a stable slope angle, after which sedimentation patterns will reach a future equilibrium state. Estimating the expected side slopes is critical for the purpose of predicting the total dredge volumes that may result from channel equilibration process and for the purpose of estimating potential effects to federal infrastructure and other resources. This analysis recognizes the inherent uncertainty in predicting the future equilibrium side slope, and therefore predicts a range of side slope outcomes. Three side slope conditions were estimated and applied for various applications as follows:

- For an assessment of existing (without project) conditions, channel side slopes were based on the *Median Measured* side slope.
- For an assessment of future (with project) conditions, channel side slopes were based on an estimated *Future Equilibrium* condition. Various methods to predict the future equilibrium condition were applied, based on the purpose of the analysis. Capital dredge volumes and costs were predicted based on variety of analyses, and the morphologic processes specific to each reach. In order to predict project impacts to adjacent infrastructure and future increased O&M volumes, a more conservative future equilibrium condition was based on the median measured side slope, assumed to originate at the toe of the dredged channel slope (rather than from the current channel bottom, which in many reaches where sediment deposition occurs is a shoaled condition that is above the toe of the existing channel).
- The *Constructed Condition* exclusively refers to the immediate, post-construction condition.

The Port used the following general process for the channel side slope analysis. The process is illustrated in Figure 3-1.

1) Evaluate the geotechnical stability of the initial proposed dredge cut slopes during and at the completion of capital dredging. The evaluations began with 3:1 (Horizontal: Vertical) sediment slopes and 1:1 rock slopes and included the following evaluations:
    a. Evaluate the static slope stability of sediment and rock slopes.
    b. Evaluate the static liquefaction and flow failure potential of sediment slopes.
2) The Port utilized the geotechnically stable configuration evaluated in Step 1 above (i.e. the constructed condition) to estimate the range of slope angles to which the channel side slopes may equilibrate following dredging and included the following evaluations:
    c. Estimate the anticipated duration for the side slope equilibration process.
    d. Estimate the range of dredge quantities associated with the channel side slope equilibration for the PA.
    e. Estimate the zone of equilibration of the equilibrated channel and effects of the equilibrium side slope on adjacent infrastructure.
3) The Port utilized the zone of the equilibration estimated in Step 2c above to evaluate the geotechnical global stability of the future equilibrium side slopes and adjacent infrastructure. The results of the evaluation were compared to the existing global stability at each adjacent infrastructure location. If the evaluation indicated a significant change to the global stability, the Port re-evaluated the global stability with mitigation in place.

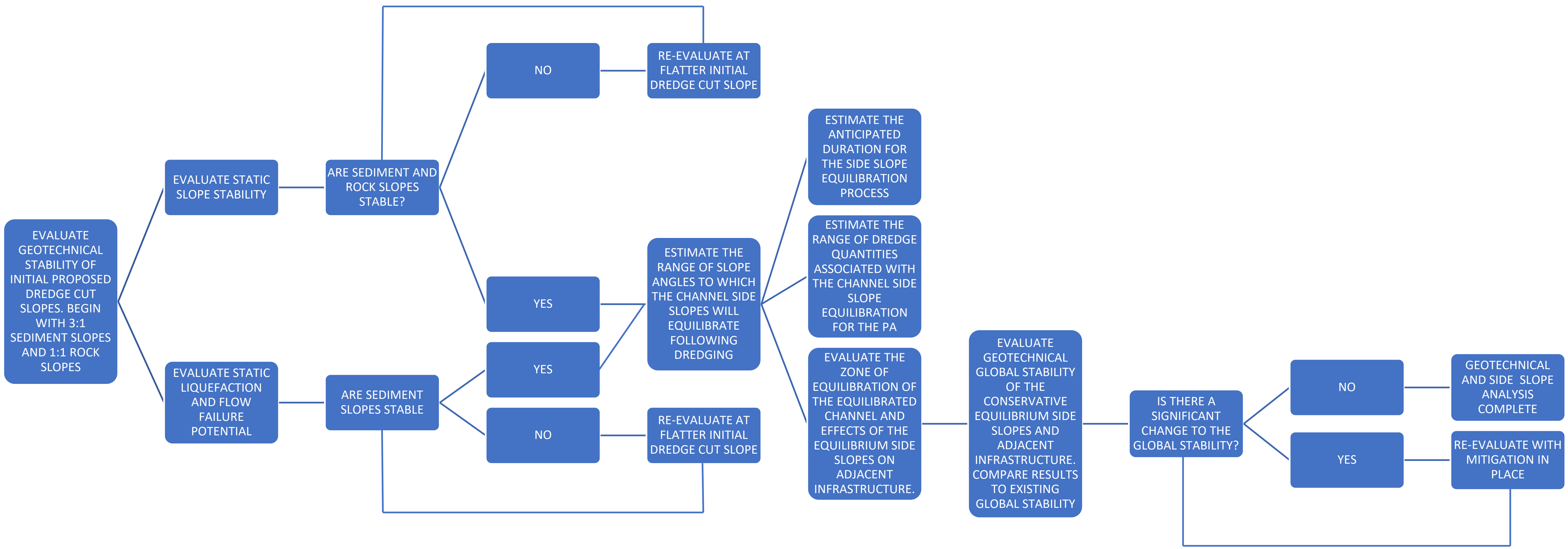


Figure 3-1
Side Slope Analysis Process Flow Chart

3.1 Methodology for Geotechnical Stability of the Initial Dredge Cut

This section summarizes the geotechnical evaluation process used to assess channel side slope stability for the proposed initial dredge cut prior to consideration of any morphological changes. Assessments of both traditional static slope stability and static liquefaction were completed to evaluate the proposed initial dredge cuts.

Based on the geotechnical data report for the project, dredged material will generally consist of:

- Very loose to very dense, fine- to medium-grained sand;
- Extremely soft to soft sandstone and mudstone of the Empire Formation;
- Extremely soft to very soft siltstone and mudstone of the Bastendorff Formation; and,
- Soft to hard sandstone and siltstone of the Coaledo Formation.

The initial dredge cut for the left bank in Reaches 6 through 9 may encounter some soft to very stiff silt (see Sub-Appendix 5 – Geotechnical Data); however, the vertical and lateral extents of this material will be substantially smaller than the other sand and rock units. Zones of peat were also encountered in Reach 9.

The assessments began with assumed initial dredge cuts of 3:1 (Horizontal to Vertical) and 1:1 for sand and rock, respectively, which is in accordance with typical assumptions used by the USACE for initial dredge cuts within the Coos Bay channel (USACE, 1996). This report recommends a flatter initial dredge cut slope and less impactful dredging methodology, including dredging completed in a stepped, top-down manner with limited cut heights, if the assessments identified significant static slope stability or static liquefaction concerns.

Lastly, the assessment includes an evaluation of the risks associated with the geotechnical stability of the initial dredge cut and dredge methods that can mitigate those risks.

3.1.1 Static Slope Stability

For the purpose of this dredging evaluation, static slope stability generally refers to 1) the initial stability of sediment above the point of dredging until equilibrium is reached from a geotechnical perspective and 2) the initial dredge slope angle that results in a stable rock cut. The Port assessed the static slope stabilities for the initial dredge cut for both the left and right banks in Reaches 1 through 9. Based on guidance from the USACE, a factor of safety of 1.5 was considered acceptable for static slope stability (USACE, 2003). The following sections summarize the potential materials that may be encountered during dredging and the factors affecting the stability of these materials within the initial dredge cut slopes. It should be noted that long-term slope equilibrium from channel morphological processes is separate from geotechnical processes. The Port's evaluation of the long-term slope equilibrium based on channel morphological processes is presented in separate sections of this report.

3.1.1.1 Sand Stability

Sand sediment is prevalent within much of the Coos Bay channel. Important factors that impact the static slope stability of the initial dredge cut within the sand sediment include the slope angle and the drained friction angle. For sand slopes in the Coos Bay channel, the USACE typically assumes initial slope angles of 3:1 in their dredging plans for sand sediment (USACE, 1996). The purpose of the analyses in this report is to review this initial slope angle assumption for the proposed cuts throughout the channel.

The slope stability of submerged sand slopes is commonly evaluated based on the infinite slope method. In this method, static slope stability for the initial dredge cut slopes is evaluated using a factor of safety against sliding (F), where F is the ratio of the forces opposing motion to the forces causing motion. For cohesionless (i.e., sandy) soils, where soil strength is expressed in terms of the effective stress and the pore water pressures are proportional to the depth of slide, the following formula expresses the factor of safety:

$$F = \frac{\tan \phi'}{\tan \beta}$$

In this formula, ϕ' = effective stress friction angle and β = slope angle measured from the horizontal. The friction angle is primarily a function of relative density and particle angularity.

Based on the above equation, a 3:1 initial slope angle and a lower-bound friction angle of 30° in sand results in an acceptable factor of safety for the cuts. Baker et al. (2005) describes that partially submerged slopes in coastal environments, such as those present in Coos Bay, may have a somewhat lower factor of safety than calculated by infinite slope methods. The factors described in Baker et al. (2005) typically contribute to somewhat flatter slopes than calculated based on infinite slope methods; however, the resulting slopes generally fall within typical dredging factors of safety for the proposed 3:1 initial dredge cut slope angle.

Sloughing, retrogressive breaching, and slope collapse are other terms sometimes used interchangeably in the literature to describe processes that result in the movement of sediment into the channel from above the point of dredging. They are related to static slope stability of sediment and occur over time where the factor of safety is less than about 1.0. Sloughing generally refers to surficial movement of sediment into the channel, and slope collapse generally refers to more severe and sudden movement of sediment into the channel. The term retrogressive breaching is more commonly used to describe conditions where the dilatancy of the denser sands results in over-steepened slopes that lead to more gradual downslope movements of materials over time (Van Rijn, 2005).

3.1.1.2 Rock Stability

Siltstone and sandstone are also prevalent within much of the Coos Bay channel (see Sub-Appendix 5 – Geotechnical Data). For rock slopes in the Coos Bay channel, the USACE typically assumes initial slope angles of 1:1 in their dredging plans for rock (USACE, 1996). Important factors that impact the static slope stability of the initial dredge cut include the compressive strength of the rock and the rock fracture orientation and spacing. The strength testing completed on rock within the channel ranges between approximately 23 psi and 11,361 psi (see Sub-Appendix 5 – Geotechnical Data). Based on strength index testing completed on samples near Guano rock (see Sub-Appendix 5 – Geotechnical Data) the analysis assumes soft (1,000 to 4,000 psi) sandstone of the Empire Formation. Based on the range of rock strengths present or assumed in the channel, a 1:1 slope results in an acceptable factor of safety for a majority of the cuts. A direct evaluation of fracture orientation was not possible with the rock coring techniques utilized in the explorations for the project (see Sub-Appendix 5 – Geotechnical Data). It should be noted that fractures can often form on rock bedding surfaces and geologic maps for the project area indicate that bedding surfaces dip towards the east-northeast down stream of approximately RM 2 and towards the west-northwest upstream of RM 2 (Madin, et al., 1995). The subsurface explorations indicate that the Empire formation has fractures that are generally closely to widely spaced (2 inches to 10 ft) and are horizontal to inclined. The Bastendorff formation has fractures that are generally very close to closely spaced (less than 2 inches to 12 inches) and vary from horizontal to nearly vertical. The Coaledo formation has fractures that are generally closely to widely spaced (2 inches to 10 ft) and inclined. Localized additional raveling could occur in zones of close fracture spacing. Adverse fracture orientation could also contribute to raveling, especially where fractures have orientations that dip down towards the channel cut. Additionally, rock with larger fracture spacing and greater compressive strength may be stable at initial slope angles steeper than 1:1. In the berth and access channel near RM 5, there are limited areas where rock is present in the dredge cut beneath a significant sand profile. In these areas, we have recommended the rock is cut at 2H:1V (where present) to reduce the risk of upslope raveling.

3.1.1.3 Silt and Peat Stability

As mentioned previously, the initial dredge cut for the left bank in Reaches 6 through 9 may encounter some interbedded layers of soft to very stiff silt, and the initial dredge cut for the left bank in Reach 9 may encounter some interbedded thin layers of peat (see Sub-Appendix 5 – Geotechnical Data). The vertical and lateral extents of the silt and peat will be substantially less than the other sand and rock units. Typical initial slope angles of 3:1 are recommended in Reaches 6 and 7 and the right bank in Reaches 8 and 9. As described in subsequent sections, initial dredge cuts of 4:1 to 5:1 are recommended for the left bank in Reaches 8 and 9, respectively, due to the risk of static liquefaction. Important factors that impact the static slope stability of the initial dredge cut within the silt sediment include 1) the undrained shear strength (Su) for the short-term condition immediately following dredging and 2) the drained friction angle for the long-term condition following dredging as excess pore water pressures dissipate over time. An undrained shear strength of 500 psf for undrained conditions (short-term) and a friction angle of 30 degrees for drained conditions (long-term) have been assumed for the controlling soft silt sediments encountered in the Coos Bay channel. The static slope stability analysis indicates a proposed initial dredge cut of 3:1 in silt, in combination with the assumed geotechnical conditions for Reaches 6 through 9, results in an acceptable factor of safety for the cuts. To evaluate the impact of the peat on the global slope stability, an undrained shear strength of 100 psf was assumed in the analysis. The static slope stability analysis including peat lenses was completed as part of the analyses described in Section 3.3 and is shown in Attachment A, Figure A-16. The static slope stability analysis indicates a proposed initial dredge cut of 5:1 with interbedded layers of peat results in an acceptable factor of safety for the cuts. The shear strengths for the soft silt sediments and peat in the static slope stability analyses were based upon guidance from correlations developed by Kulhawy and Mayne (1990) and blow count data from explorations completed as part of the geotechnical data report for the project (see Sub-Appendix 5 – Geotechnical Data).

3.1.1.4 Other Factors

Other factors that significantly impact the static slope stability include morphological processes such as seepage, scour, and wave action. Wave action commonly results in slopes angles of 6:1 to 10:1. Scour conditions tend to result in slope angles steeper than 3:1. Other sections of this report evaluate the impact of scour and wave action in depth.

3.1.2 Static Liquefaction and Flow Failure

Static liquefaction also involves the movement of sediment into the channel from above the point of dredging. Static liquefaction results in flow failure and occurs when a mass of loose, sandy soil on an underwater slope suddenly liquefies and flows downward into a flatter slope as the result of a small, but rapid, change in soil stresses. Static liquefaction typically results in substantially flatter slopes rather than static slope stability failures. Koppejan et al. (1948) and Kramer (1988) describe three conditions necessary for static liquefaction to occur: 1) the sandy soil must be liquefiable (loose, fine-grained, sandy soils are most susceptible to static liquefaction), 2) the slope geometry must be sufficiently unfavorable (as slope height and slope angle increase, the probability of static liquefaction increases), and 3) an initiation or triggering mechanism must be present. Whether a flow failure will be initiated depends on the contractive behavior of the sand sediment, which is a function of the mean effective stress and density, and the initial shear stresses determined by the slope height and slope angle (Stoutjesdijk et al., 1998). Loosely packed sand tends to contract in response to a change in shear stress, such as during the removal of material at the toe of a slope as a result of erosion or dredging. Relatively loose saturated soil loaded in an undrained manner will generate positive excess pore water pressure in the pore space between the soil grains, resulting in a reduction of effective stress and, thus, soil strength and potentially leading to static liquefaction. Review of the available literature suggests static liquefaction can result from removal of material at the toe of a slope as a result of erosion or dredging, seepage due to tidal water-level changes, vibrations, waves from ships, and steepening and/or deepening of the channel due to scour or dredging activities (Stoutjesdijk et al., 1994). The assessments primarily considered the risk of static liquefaction from dredging. While the remaining triggering mechanisms also exist within the channel, the risk of static liquefaction from these triggers are not anticipated to change significantly as a result of the proposed channel modifications.

Table 3-1 summarizes the parameters used in the assessments of static liquefaction. The static liquefaction evaluation was based on the channel and upland geometry and characteristics presented in the project plans (see Sub-Appendix 8 - Drawings) showing the PA channel, rock surface, bathymetry, and topography. Screening for static liquefaction and flow failure was completed along the entire length of the PA channel, including locations with proximal infrastructure, as discussed in Section 3.3. In general, further assessments were completed at locations with the most adverse cross-sectional geometry in each reach, which typically coincided with the greatest height of the initial dredge cuts in sand. A detailed description of the methodology used in the assessments is presented below.

Table 3-1
Static Liquefaction Assessment Parameters

Reach	Location		Assumed Sediment Parameters		Approximate Maximum Initial Dredge Cut Height, ft
	Bank	River Mile	Soil Type	Soil Relative Density	
Reach 1	Left Bank	RM 0.00	Very Loose Sand	0.3	27
	Right Bank	RM 0.11	Very Loose Sand	0.3	19
Reach 2	Left Bank	RM 0.15	Very Loose Sand	0.3	24
	Right Bank	RM 0.11	Very Loose Sand	0.3	20
Reach 3	Left Bank	RM 0.49	Very Loose Sand	0.3	28
	Right Bank	RM 0.55	Very Loose Sand	0.3	22
Reach 4	Left Bank	RM 1.25	Loose Sand	0.6	10
	Right Bank	RM 1.21	Loose Sand	0.6	17
Reach 5	Left Bank	RM 4.47	Medium-Dense Sand	0.8	17
	Right Bank	RM 4.39	Medium-Dense Sand	0.8	13
Reach 6	Left Bank	RM 4.55	Medium-Dense Sand	0.8	22
	Right Bank	RM 5.30	Medium-Dense Sand	0.8	60

Reach	Location		Assumed Sediment Parameters		Approximate Maximum Initial Dredge Cut Height, ft
	Bank	River Mile	Soil Type	Soil Relative Density	
Reach 7	Left Bank	RM 6.40	Medium-Dense Sand	0.6	15
	Right Bank	RM 5.68	Medium-Dense Sand	0.6	20
Reach 8	Left Bank	RM 6.63	Very Loose Sand	0.3	28
	Right Bank	RM 6.59	Medium-Dense Sand	0.8	10
Reach 9	Left Bank	RM 7.50	Very Loose Sand	0.2	22
	Right Bank	RM 7.99	Medium-Dense Sand	0.6	13

The risk of static liquefaction for the initial dredge cuts was assessed using two approaches. Multiple approaches were used to review that the two separate approaches calculated similar results. The first approach included a screening analysis, and the second approach included a probabilistic analysis.

The first approach was a screening analysis developed by Stoutjesdijk et al. (1994) and method developed by Raaijmakers (2005) that compares the geometric and sediment-density characteristics of the proposed initial dredge cuts to the likelihood of static liquefaction. Stoutjesdijk et al. (1994) compiled data from more than 100 flow failures, including the average and steepest slope angles, slope height, and relative density of the sediment within the observed flow failure. As mentioned previously, the work of Stoutjesdijk et al. (1994) shows that the risk of flow failure increases with slope height and angle; however, considerable scatter is present in the data. Raaijmakers (2005) developed a criterion that also assesses the likelihood of static liquefaction based on relative density, critical slope height, and slope angle without presenting specific case histories.

Relative density, D_R, was computed based on a correlation with Standard Penetration Test (SPT) blow counts (Cubrinovski and Ishihara, 2001). The SPT blow counts (N) were based on boring data from overwater exploration programs for the project (see Sub-Appendix 5 – Geotechnical Data), σ'_v = vertical effective stress, $(e_{max} - e_{min}) = 0.23 + 0.06/D_{50}$, and D_{50} = mean grain size in millimeters.

$$D_R = \left[\frac{N\left(e_{\max} - e_{\min}\right)^{1.7}}{9} \left(\frac{98}{\sigma'_v} \right)^{0.5} \right]^{0.5}$$

No geotechnical information is available in Reaches 1 through 3 to evaluate the relative density of the assumed sand sediment. Therefore, the analysis assumes very loose sand within these reaches to evaluate the static liquefaction. An assessment of the likelihood of static liquefaction for the initial dredge cuts for Reaches 1 through 9 was completed by comparing the geometric and sediment density characteristics of the proposed initial dredge cuts to the observed or predicted failures presented in the preceding analyses. In general, the assessment demonstrated agreement between Stoutjesdijk et al. (1994) and Raaijmakers (2005) methods.

The second approach used to assess the risk of static liquefaction for the initial dredge cuts was a physics-based probabilistic analysis developed by Van den Ham et al. (2014) to quantify the risk of static liquefaction. Van den Ham et al. (2014) developed a physics-based, semi-empirical method to assess the risk of static liquefaction based on statistical information from 710 documented flow failures in the Province of Zeeland, Netherlands, described by Wilderom (1979). In the method by Van den Ham et al. (2014), the probability of static liquefaction is a function of slope height, H_R; slope angle, α_R; and relative density, D_R, where $f_1(H_R) = (H_R/24)^{2.1}$, $f_2(\alpha_R) = (5/\text{cotan } \alpha_R)^{5.7}$ and $f_3(D_R) = (1/3)^{10-(D_R - 0.3)}$:

$$P(SL) = f_1(H_R) \cdot f_2(\alpha_R) \cdot f_3(D_R)$$

In general, the results from the screening analysis corroborate the probabilistic analyses. In this regard, lower static liquefaction probabilities calculated with Van de Ham et al. (2014) were associated with stable slopes identified using the Raaijmakers (2005) and Stoutjesdijk et al. (1994) approaches.

As used in this analysis, a relatively high risk of static liquefaction is defined as 1) sediment relative density, initial dredge cut slope height, and initial dredge cut slope angle that plotted in the unstable region on both the Stoutjesdijk et al. (1994) and Raaijmakers (2005) charts; and 2) sediment relative density, initial dredge cut slope height, and initial dredge cut slope angle that resulted in a probability of failure of greater than approximately 0.01 flow failures per kilometer of slope per year. An approximation was used for borderline situations. Details regarding the use of the Raaijmakers (2005) and Stoutjesdijk et al. (1994) charts, and the Van de Ham et al. probabilistic analyses within the analysis are included in Attachment B. The Raaijmakers (2005) and Stoutjesdijk et al. (1994) charts are included in Attachment B, Figures B-1 and B-2, respectively. An example calculation using the Van de Ham et al. probabilistic analysis is attached in Attachment B. A sensitivity analysis of the screening and probabilistic approaches is also included in Attachment B.

To summarize the assessment, in cases where the screening analysis and the probabilistic results indicated the proposed initial cut had a high risk of static liquefaction, the analysis was repeated using a flatter initial dredge cut slope until the results indicated a stable cut could be made without a significant risk of static liquefaction. Section 4.1 of this report summarizes the recommended initial dredge cuts for Reaches 1 through 9.

3.1.3 Flow Failure Dredging Considerations

Dredging techniques can strongly influence the risk associated with sand sediments susceptible to static liquefaction and flow failure. Specifically, it is important to specify a dredging process that can remove material from the appropriate location on the slope and create a shallower design slope if there is a risk of flow failure. For example, a deep box cut at the base of a relatively loose sand slope has the potential to cause static liquefaction and large flow failures. To mitigate the risk, the loose sand slopes may require dredging completed in a stepped, top-down manner with limited cut heights. The preceding method is also more suited to control the slope angles for the initial dredge cut.

Section 4.1 of this report includes more detailed dredging considerations for Reaches 1 through 9 based on the sediment types present within each reach.

3.1.4 Liquefaction

The more common definition of liquefaction (different than static liquefaction) is a process where loose, saturated, granular materials, such as clean sand, temporarily lose stiffness and strength during and immediately after a rapid cyclic loading event, such as a seismic event. Liquefaction occurs as rapid cyclic shear stresses propagate through a saturated soil and distort the soil structure, causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure causes the pore-water pressure to increase between the soil grains. If the pore-water pressure becomes sufficiently large, the inter-granular stresses become small and the granular layer temporarily behaves as a viscous fluid rather than a solid. The ratio of pore-water pressure to effective stress is defined as Ru, and as Ru values increase, there is an increased risk of settlement, loss of bearing capacity, lateral spreading, and/or slope instability. Liquefaction-induced settlement occurs as the elevated pore-water pressures dissipate and the soil consolidates after the rapid cyclic loading event.

The geotechnical evaluation excludes assessments of seismic hazards including seismic liquefaction throughout the channel; however, Section 3.3 includes a general description of the seismic susceptibility of the North Jetty following a Cascadia Subduction Zone earthquake.

3.2 METHODOLOGY TO ESTIMATE FUTURE EQUILIBRIUM SIDE SLOPES

Capital dredging represents a large, short-term removal of sediment from the navigation channel in previously un-dredged areas. The capital dredging may increase the depth of established navigation channels, may widen the navigation channel into adjacent areas, or both, as described in Section 1.4. Following capital dredging, newly established side slopes may require an adjustment period before they achieve morphologic equilibrium. Historic maintenance dredging records show increased maintenance dredging volumes immediately following capital dredging projects. This increased maintenance dredging is caused by greater sediment contribution by side slopes as they readjust from the initial dredge cut angles to more gentle equilibrium slopes. As a result, it is necessary to estimate a probable range of slope angles to which the side slopes will equilibrate following capital dredging. Once the equilibrated slopes have been determined, changes in dredge volumes, environmental effects, and effects on adjacent infrastructure can be assessed.

When an existing navigation channel is deepened and/or widened, the dredged side slopes may or may not erode or shift, depending on many factors. Dredging-specific effects to side slopes are time-dependent and generally include both geotechnical and morphological slope changes. During and shortly after dredging, side slope stability is primarily driven by geotechnical considerations as upslope sediments respond to the new dredging-induced geometry. Following capital dredging, the side slope equilibration in an initially stable slope can last anywhere from weeks to years. Currents erode material from the top of the slope, causing sediment transport towards the toe of the slope. This transport displaces sediment from the top of the slope towards deeper water into the channel. Over time, the currents continue to flatten the slope, until an equilibrium slope is achieved.

The measured side slopes within Coos Bay provide insight into the slope angles that may form after channel equilibration. These slopes have been subject to the processes that drive equilibration for over twenty (20) years since the last major capital dredging project in 1996. Nevertheless, measured side slopes cannot be linked exclusively to side slope equilibrium development, and instead may be the product of several other ongoing processes that occur simultaneously throughout Coos Bay. For example, in the Entrance Channel, several ongoing morphological processes cause the side slope angles to vary throughout the year (as described below). This variation is not a result of side slope equilibration due to channel modification, but rather an indication of the active morphology throughout the estuary. In light of these observations, future side slope equilibrium is assessed by identifying and isolating the various processes within the channel that do and do not drive side slope equilibration. This methodology consists of three key assessments that conform to the definition of side slope equilibration presented below (Section 3.2.1):

1. Bound the range of existing condition side slope angles based on median measured side slope angles within each reach;
2. Identify particular portions of the channel that could be subject to future equilibration; and
3. Predict the long-term channel behavior based on quantitative and qualitative assessment of morphological processes.

These assessments are presented in the sub-sections following Section 3.2.1.

3.2.1 Definition of Future Equilibrium Side Slopes

Future equilibrium side slopes are defined herein as the slope angles that are likely to develop in the channel as a result of capital dredging. The process of side slope equilibration occurs simultaneously with several other processes that move sediment within the channel. The impacts to side slopes that are specific to channel modification include geotechnical instability and morphological slope flattening due to a cross slope gradient in bed velocity. The cross-slope velocity gradient and gravity leads to erosion at the top of the slope and deposition at the toe of the slope. It should be noted that all slopes will be dredged to a geotechnically stable angle.

3.2.2 Range of Slope Angles

At each location where capital dredging occurs in sediment, the range of long-term slopes is assumed to be bound by the median measured slope angle, projected from the toe of the dredge cut, and the design dredge cut. The median measured slope angles are calculated based on various data sources for the existing, without project condition (detailed in Section 4.2.1). The median measured slope analysis represents one line of evidence in determining the range of future equilibrium side slopes. This first assessment using median measured slopes does not consider other physical processes, local characteristics, or environmental changes that can lead to a different slope than what has been measured historically.

Slopes flatter than the median measured side slopes are not expected to occur after channel modification. The effect of widening and deepening the channel has the effect of reducing current velocities throughout the channel, as shown by the detailed modeling presented in Sub-Appendices 3 and 4 (*Estuarine Dynamics* and *Offshore and Entrance Dynamics*). As a result, there is less hydrodynamic forcing on the tops of the slopes, and less potential for erosion of the slopes into the channel ("sloughing"). Work published by Fredsoe (1978) and Raaijmakers (2005) also indicates that deeper channel dredge cuts produce steeper equilibrium sides slopes. This relationship between channel depth and equilibrium side slopes can be seen in historical cross-sections charted at the Coos Bay Entrance (Figure 3-2). As the authorized channel depth increases in Figure 3-2, the slopes adjacent to the channel steepen.

For a normally distributed dataset, the median represents the central, middle-most value. Assuming that the side slopes for a given reach are normally distributed over space and time, the median measurement is appropriate for assessing long-term effects for that reach. Taking a single, more extreme static measurement from the collection of data to analyze long-term affects is not reasonable due to the dynamic nature of bathymetry and data uncertainty. Furthermore, the future equilibrium side slope model projects the side slopes out from the dredged toe. In reality, dredged slopes rotate back about the mid-point of the dredge cut and form more of an s-curve. In summary, a median measured side slope of a shallower, narrower channel, projected out from the dredge toe, is appropriate for bracketing possible future equilibrium side slopes for a deeper and wider channel.

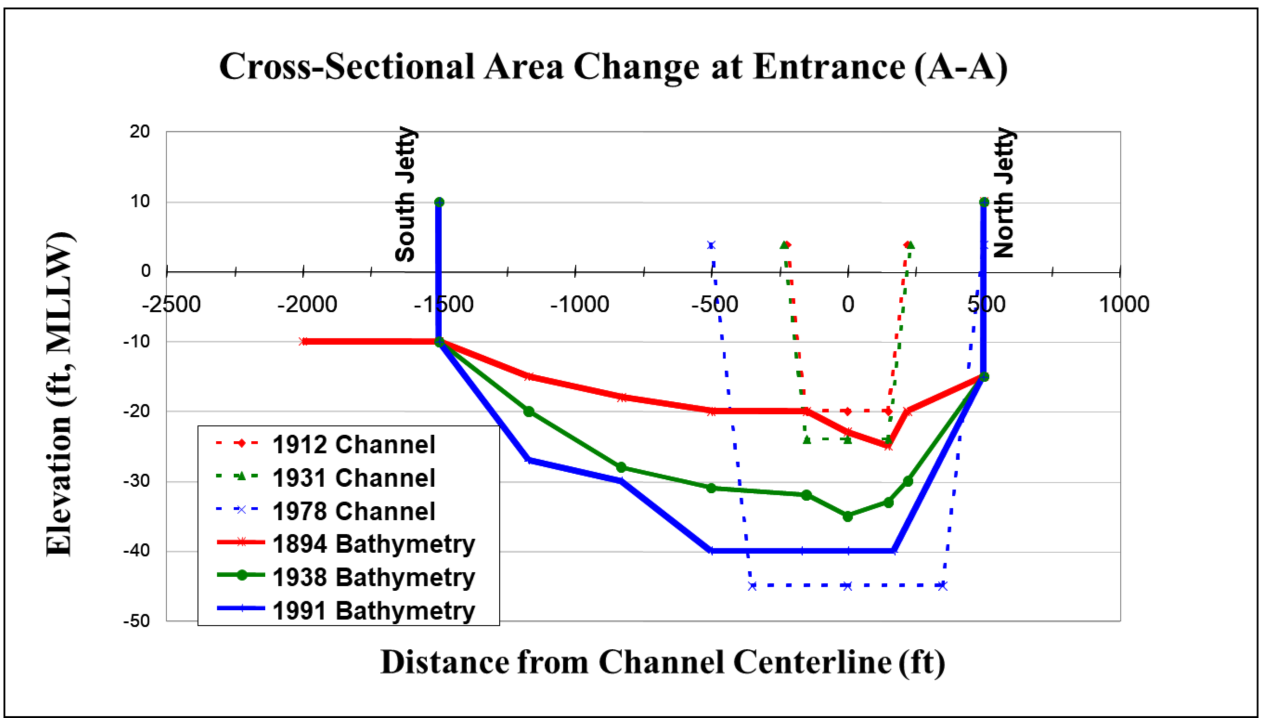


Figure 3-2
Steepening Side Slopes as the Coos Bay Entrance Channel is Deepened (USACE 2019)

3.2.3 Identify Areas where Equilibration May Occur

Side slope equilibration may occur where sediment is present on the side slopes above the proposed cut. Side slope equilibration is not expected to occur in rock; all dredging in rock is assumed to be stable at the cut angle of 2:1 at the vessel slip and 1:1 everywhere else (Horizontal:Vertical – this notation is used to describe side slope angles throughout this report). However, if sediment overlays rock, then the overlying sediment will be subject to future equilibration. This step entails identifying whether rock or sediment is present, and whether the material will result in side slope equilibration.

3.2.4 Morphological Processes

Van Rijn (2005) categorizes the morphological development of a navigation channel by three flow conditions that induce morphological changes, as seen in Figure 3-3. Morphological changes are driven by:

- unidirectional flow perpendicular to the channel;
- bidirectional flow perpendicular to the channel; and
- bidirectional flow parallel to the channel.

Different portions of the Coos Bay FNC are subject to the latter two flow conditions. For bidirectional flow perpendicular to the channel, erosion occurs at the top of both slopes, and deposition occurs in the channel by reduction of sediment transport capacity (Figure 3-3, middle). For flow parallel to the channel, gravitational effects transport sediment from the slopes into the channel (Figure 3-3, bottom).

These two morphological processes are similar in that sediment is eroded from one portion of the cross section and deposited on another. In energetic environments such as Coos Bay, sediment is constantly being eroded and deposited, regardless of channel slopes. However, the rate of net transport from the top of the slope into the channel depends on the hydrodynamic gradient and on the influence of gravity, both of which are determined by the side slope angle.

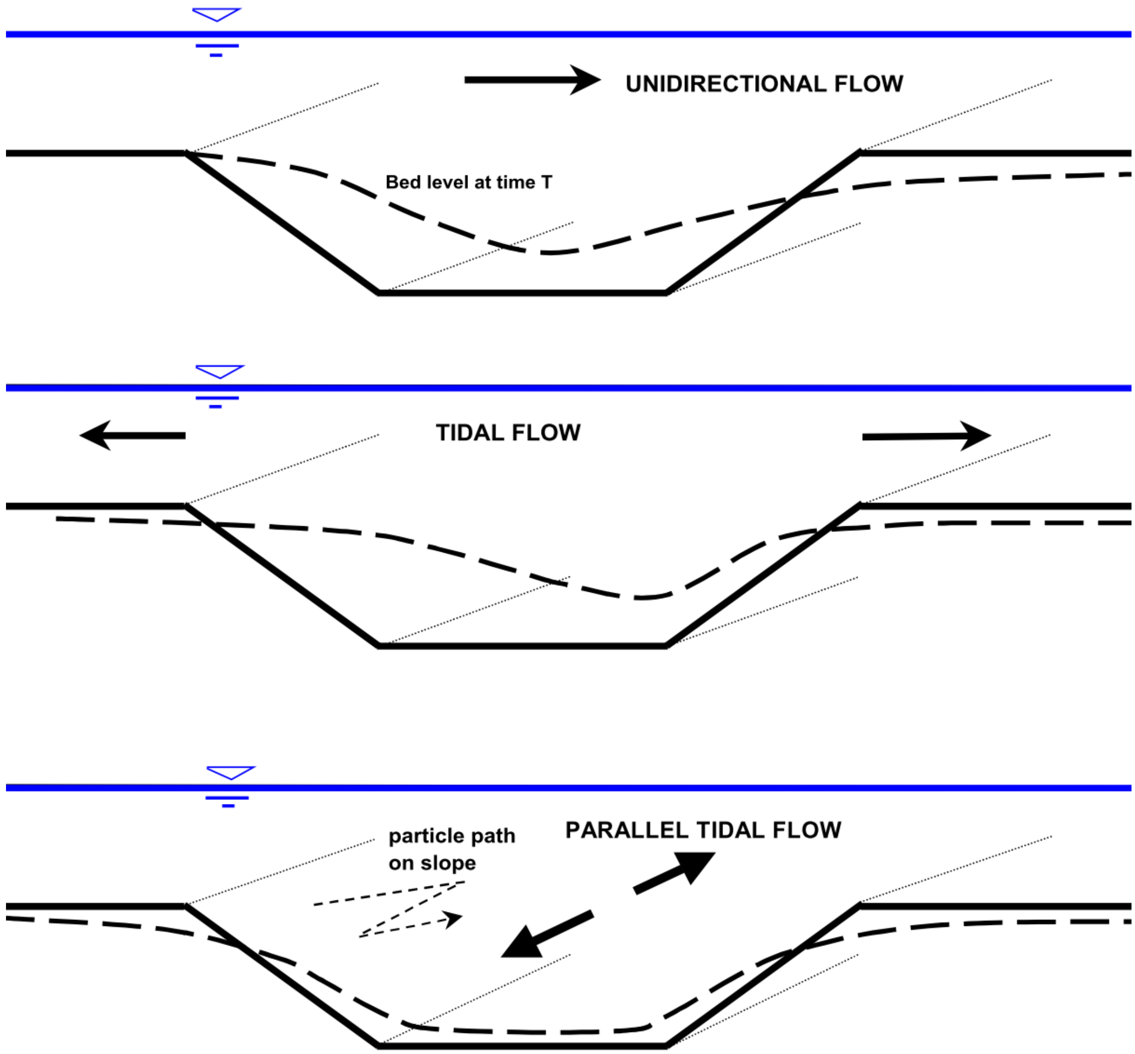


Figure 3-3
Morphological Slope Adjustment Mechanisms (Source: Van Rijn 2005)

3.2.4.1 Currents Perpendicular to the Channel Alignment

The process of sedimentation in a channel subject to perpendicular currents, such as in Reach 1 offshore of the jetties, is described in Van Rijn (2005):

"When a current crosses the channel, the current velocities decrease due to the increase of the water depths in the channel and hence the sediment transport capacity decreases. As a result, the bed load particles and a certain amount of the suspended particles will be deposited in the channel. The settling of sediment particles is the dominant process in the downsloping (decelerating) direction and in the middle section of the channel. In the case of a steep-sided channel with flow separation and associated extra turbulence energy, the settling process may be reduced considerably. In the upsloping (downstream) section of the channel the dominant process is sediment pick-up from the bed into the accelerating flow, resulting in an increase of the suspended sediment concentrations."

The particular side slope angle to which the channel will equilibrate can be estimated by a set of equations provided by Raaijmakers (2005). This method predicts the development of a "threshold profile," which is based on the assumption that bed load is constant throughout the cross section. For this assumption, the hydrodynamics are assumed to be such that sediment particles on the equilibrium side slopes are just stable (or at the threshold of motion, based on a shear stress analysis). This threshold profile is calculated by modifying the static stability and threshold of motion equations by stability reduction factors based on the geometry of the bed and the sediment properties. The equations themselves consider the depth, the slope height, the grain size, and the critical current discharge. A schematic of the initial and threshold profiles is provided in Figure 3-4.

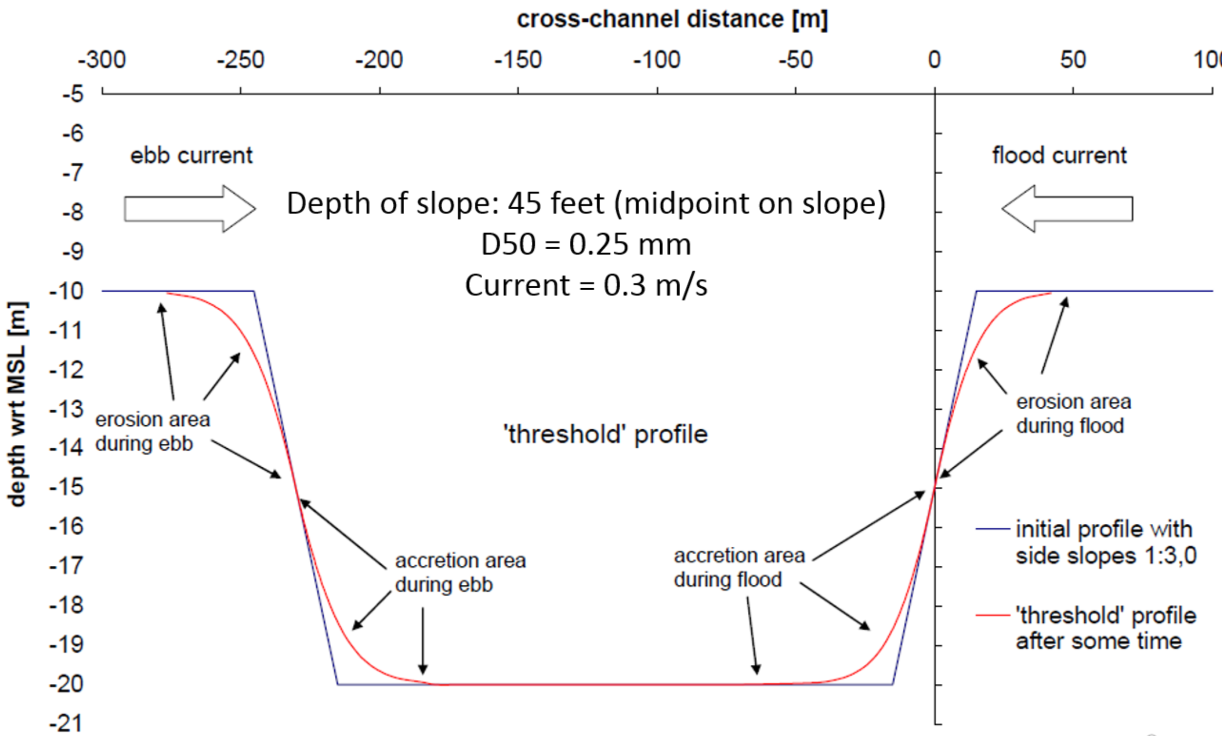


Figure 3-4
Schematic of the "Threshold Profile" (Source: Raaijmakers 2005)

The equations to implement this analysis are presented below.

$$\beta_{crit} = \phi - \arcsin\left(\frac{\sin \phi q_{crit}^2 \kappa^2}{\Psi_C d_{50} \Delta g h^2 \ln^2\left(\frac{12h}{k_r}\right)} \right)$$

where:

β_{crit} is the critical side slope angle (°)

φ is the internal angle of friction = 30 (°)

q_{crit} is the critical discharge, equal to the product of the depth and the critical velocity (m^2/s)

κ is the von Karman constant = 0.4 (-)

Ψ_c is the Shiled's stability parameter (-)

d_{50} is the median gran size = 0.00025 (m)

Δ is the specific gravity of sand = 1.65 (-)

g is the gravitational constant = 9.81 (m/s^2)

h is the water depth = 15 (m)

k_r is the bed roughness = 0.03 (-)

where the stability parameter is calculated from:

$\Psi_C = 0{,}24 d_*^{-1}$	for	$1 \le d_* \le 4$
$\Psi_C = 0{,}14 d_*^{-0{,}65}$	for	$4 < d_* \le 10$
$\Psi_C = 0{,}04 d_*^{-0{,}1}$	for	$10 < d_* \le 20$
$\Psi_C = 0{,}013 d_*^{0{,}29}$	for	$20 < d_* \le 150$
$\Psi_C = 0{,}055$	for	$150 < d_* \le 1000$

and the critical velocity can be calculated from:

$$\bar{u}_{crit} = \sqrt{\Psi_c d_*} \frac{\sqrt[3]{\Delta \nu g}}{\kappa} \ln\left(\frac{12h}{k_r}\right)$$

where ν is the kinematic viscosity = 1.4*10^{-6} (m^2/s).

In Coos Bay, the current discharge is greater than the critical current discharge assumed in the threshold profile analysis. Raaijmakers developed a "dynamic equilibrium slope," which estimates slope angles for stronger hydrodynamic forcing to account for the conditions encountered in Coos Bay. Under a "dynamic equilibrium slope," when a current enters a channel at the upstream side slope, the bed load transport remains constant if the negative influence of increasing water depth is counterbalanced by an increasingly downward sloping bottom (i.e., the decreasing flow velocity is counterbalanced by an increase in gravitational effects). Due to the consideration of active bed load transport, the side slopes become flatter for larger discharges. This model assumes a discontinuity at the channel bottom, where transport cannot be in equilibrium with the transport over the side slopes. Therefore, increased maintenance dredging at the channel bottom is required during channel equilibration. This schematic model makes several simplifying assumptions, such as neglecting suspended sediment and the interaction between the slope and the channel bottom, but it still provides a reasonable estimate for estimating how side slopes equilibrate due to bed load transport, and, therefore, it can be used to estimate the future equilibrium side slope, for purposes of estimating capital dredge quantities, in Reach 1.

3.2.4.2 Currents Parallel to the Channel Alignment

Upstream of Reach 2, the currents are generally parallel to the channel alignment. In this case, the side slopes of the channel are flattened/smoothed due to gravitational effects. When a sediment particle resting on the side slope is set into motion by currents, it will move very nearly in the direction of flow. However, due to gravity, the path of the particle will deviate slightly, in a downward direction. By this mechanism, sediment is transported to the deeper part of the channel yielding reduced depths and smoothed side slopes. This is illustrated in the bottom of Figure 3-3.

This process is quantified by the method presented by Fredsoe's (1978) equations of gravity infill. These equations are based on the paper, "Sedimentation of River Navigation Channels" and predict how a dredged slope will evolve in time. Input variables include sediment properties, hydraulic properties (e.g., shear stress), and initial channel geometry (slope angle, slope height, sediment characteristics, and bed load transport parallel to the channel). The output consists of resultant side slope angles after various time intervals. Higher flow, finer and looser sediments lead to flatter slopes. Larger cuts and more compact sediments lead to steeper slopes. The complex mass transport equation is approximated using the equation of heat conductivity. The midpoint of the slope is chosen as the origin and fixed in time (Figure 3-5). As time progress, the flattening rate slows, reaching an equilibrium.

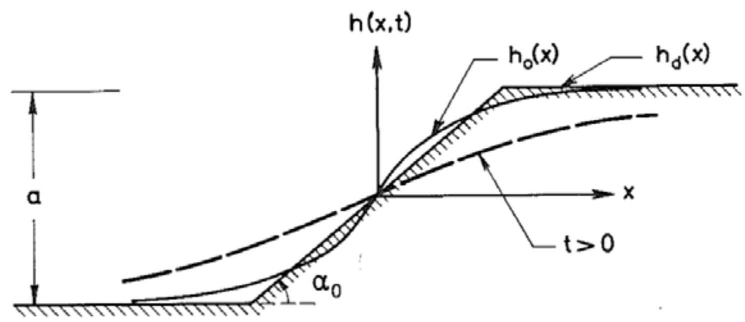


Figure 3-5
Development of Slope in Time (Source: Fredsoe 1978)

The Fredsoe approach has been adapted to a desktop model. The general model workflow is as follows:

1. Determine slope characteristics (height, mid-slope elevation, toe elevation, angle).
2. Estimate geotechnical characteristics using available geotechnical data (porosity, friction angle, d50, d90).
3. Extract site-specific flow data (depth-averaged velocity or shear stress) from hydrodynamic model. If needed, adjust flow characteristics to represent the mid-slope location.
4. Calculate threshold velocity to initiate sediment motion at the mid-slope location.
5. Calculate yearly-average bed load transport rate at the mid-slope location based on the results of (3) and (4).
6. Run the desktop model using the average bed load transport rate, porosity, friction angle, and slope characteristics.

The model inputs are based on field data, proposed dredge plans, and hydrodynamic modeling of the Coos Bay Channel. Conceptually, these equations assume that material moves from the top of the slope into the channel via gravity and calibrate the model to the measured slope angle.

While this process is likely occurring to some extent throughout the channel, it is only expected to drive side slope behavior in straight portions of the channel (Reaches 6 and 7). In curved areas, hydrodynamic complexities such as secondary flow may transport sediment more rapidly than this mechanism. Along curved portions of a channel, a horizontal gradient in velocity develops, causing erosion and deposition on the outer and inner bend, respectively; these morphological processes dominate the local morphology, and, therefore, the infill method described in this subsection does not drive side slope equilibration in curved areas.

3.2.4.3 Time Scale of Side Slope Equilibration

The time scale of side slope equilibration is estimated based on studies of side slope equilibration within Coos Bay and the Columbia River. Previous channel modification projects in Coos Bay (1977-9 and 1996) and research papers describing side slope equilibration in the Columbia River (Babcock 1989) were both considered.

The Engineering Appendix to the 1994 EIS for the 1996 channel modification (USACE 1994a) presents dredge records following the 1977-9 dredging project to estimate the timescale for equilibration. Analyzing these data show a significant increase in dredge volumes 4.5 years after dredging was completed. After 7.5 years, the dredging rate returns to the long-term annual volumes (data is presented as a 3-year rolling average). From these data, it appears that equilibration lasted about 5 years for a 5-ft deepening project; the volume associated with equilibration decreased exponentially with each year after dredging. This document also states that the side slope equilibrium volume is proportional to the cut depth (Table B-5 of USACE 1994a). Assuming that the volume is proportional to the cut depth, and that the excess shoaling decreases exponentially, the equilibration length decreases nearly proportionally to the depth of cut.

Coordination with USACE (in-person meetings and review of dredge records) indicates that side slope equilibration lasted 2-3 years after the 2-ft channel deepening in 1996. Babcock (1989) provides a review of side slope equilibration within the Columbia River. The work states that dredge cuts between 2 to 5 ft take about 3 to 5 years to equilibrate. Over this time, the excess sedimentation volumes decrease exponentially.

Review of data on previous deepening projects indicates that dredge cuts of 2 ft equilibrate over a course of 3 years, and that dredge cuts of 5 ft equilibrate over 5 years. Consistent with this trend, a channel deepening of 8 ft (such as the PA) is expected to equilibrate over 8 years.

The desktop model based on the Fredsoe approach (Section 3.2.4.2) confirmed that this timescale is reasonable. The model calibration process found that an equilibration time period of 8 years was generally used for model calibration, which yielded reasonable results compared to the measured slope angles. Therefore, a time period of 8 years was used for subsequent Fredsoe analysis.

In the offshore area, equilibration is expected to happen more quickly. Raaijmakers (2005) analyzed slope stability based on information from various dredged trenches for pipelines in Great Britain and the Netherlands, ranging from 30 to 100 ft deep. This research found that, for unprotected channels, equilibration lasted from 15 days in storm weather to 60 days in calm weather (the depth of the cut ranged from 13 to 20 ft). Therefore, in the offshore and entrance reaches (Reaches 1 through 3), it has been assumed that equilibration will be complete within 2 months. This implies that a portion of this equilibration will occur while construction is ongoing. Post-dredge surveys show side slopes in the range of 4:1-7:1, indicating that approximately 20% of the material will slough into the Entrance Channel during construction; this volume is factored into the volume during the construction year. The remainder will slough in over the following year (combined with infill), and be removed during the subsequent year (i.e., year 3).

3.2.4.4 Other Morphological Processes

The two processes described above include transport of material from the top of the slope into the channel, effectively flattening the channel. The processes described in this section differ from those presented in the previous sections in two ways. Conceptually, they differ from the processes described above because they describe transport of sediment from one location in the channel to another (i.e., between two channel stations) instead of transport within the cross section; they are channel-wide processes as opposed to localized side slope processes. The processes described in this section also differ because they are a result of the present channel configuration and hydrodynamics, and not necessarily the result of channel modification; they can be observed in several locations under the existing condition.

As noted above, the processes described within this section are highly dependent on the hydrodynamic regime. Appendix 3 (*Estuarine Dynamics Report*) presents the results of hydrodynamic modeling within the estuary, which ultimately showed that implementation of the channel modification is expected to result in a change in current velocities of less than 16%, relative to the Existing Condition. Therefore, the processes described below are expected to continue as presently observed. These processes are described in detail in subsequent sub-sections.

- Shoaling. The long-term process of side slope evolution is expected to take 8 years (see Section 3.2.4.3). In certain reaches, however, shoaling occurs annually over the entire cross section – from the toe of slope through the top of slope (Figure 3-6 and Figure 3-7). As noted above, equilibrium side slopes are defined as the slope angles that are expected in the channel after the effects of capital dredging no longer affect channel morphology. In areas of the channel where the timescale of shoaling is less than 1 year, maintenance dredging records show that shoaling occurs annually and the morphology is dominated by this annual pattern of shoaling and dredging, not by a response to deepening.
- Erosion. In portions of the channel, a review of historic bathymetry indicates that erosion is still occurring, e.g. the outer bend of Jarvis Turn. In these areas, sediment is continuously removed from the bottom of the slopes; removal of sediment from the toe of the slope maintains steep side slopes or even continues to steepen the slopes. Erosion occurs independent of dredging and appears to steepen the channel beyond the dredged slope. Therefore, the channel is expected to be stable at the initial slope or steeper. Erosion may decrease as a result of channel modification, which is expected to decrease current velocities in the channel.

3.2.4.4.1 Areas of Shoaling

In many locations throughout the channel, the morphology is dominated by shoaling of sediment from offshore or from other portions of the channel. In particular, this is evident throughout Reaches 1 and 3, and on the inner bank of Reaches 8 and 9.

This is best observed in Reach 3, where the measured side slopes reflect sediment shoaling from the toe of slope towards the top of slope, and not the actual stable side slopes within the channel. Pre-and post-dredge surveys provided by the USACE for several years can be used as evidence of this infill trend; less than 1 year after maintenance dredging is complete, shoaling from offshore sediment has flattened onside slopes. The side slopes in Reach 3 are measured from survey data collected in April 2008; this dataset was selected because it was the only surveyed surface in this reach that provided complete, bank-to-bank coverage. While each survey varies due to the dynamic nature of the inlet, surveys conducted in October-December tend to show steeper slopes, because they were taken soon after dredging (despite the limited coverage). The April survey was collected approximately six months after dredging was completed. The channel was exposed to severe winter storms during this period, which pushed sediment into the channel. These nearly flat slopes represent sediment infill as opposed to post-dredging side slope adjustments.

Figure 3-6 and Figure 3-7 show the shoaling at Station 22+50 that occurred from 2007 to 2008 and from 2008 to 2009, respectively, by comparing pre-and post-dredge surveys provided by USACE. (It should be noted that all cross sections presented in this report are oriented to face downstream.) While these surveys do not offer full coverage of the channel and have coarse spatial resolution, they clearly show the magnitude of sediment infill that occurs annually. Shoaling occurs over the entire cross-section; material is not lost from the top of the cross section into the channel but accretes throughout. After channel modification, this observed pattern of shoaling across the entire cross-section is expected to continue; in fact, shoaling is expected to increase (Sub-appendices 3 and 4) both within the channel and along the side slopes as a result of increased channel trapping efficiency or mobilization of additional sediment. Therefore, the net behavior is material accumulating on the tops of the slopes, not sloughing into the channel bottom. A more detailed discussion of future shoaling in Reach 3 is presented in Section 4.2.4.3.

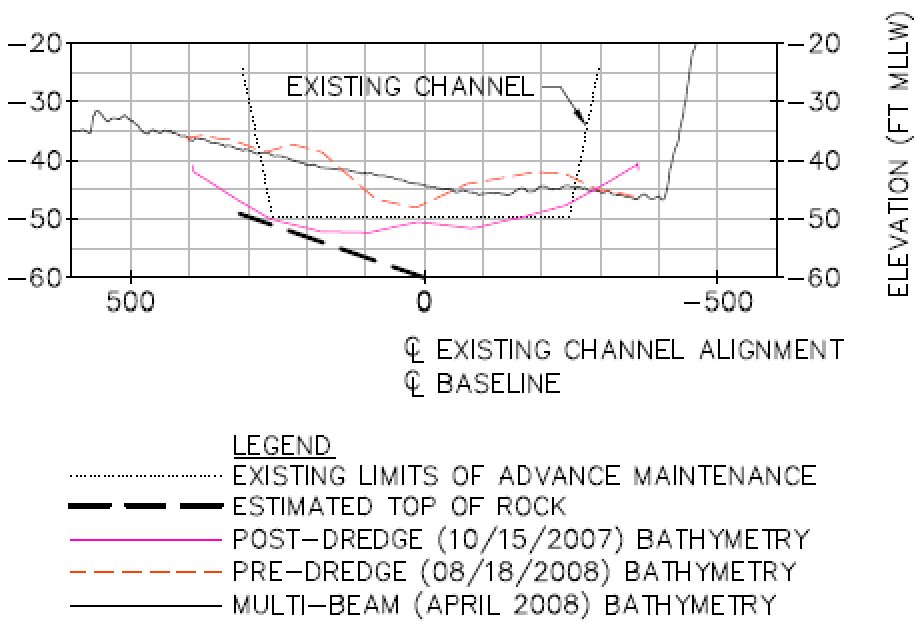


Figure 3-6
Shoaling at Station 22+50 from October 2007 to August 2008

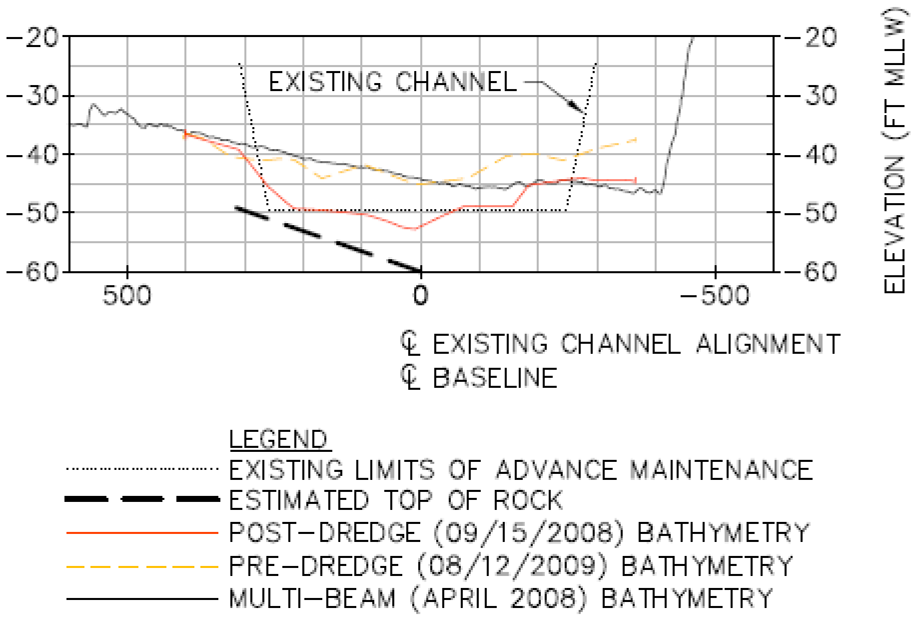


Figure 3-7
Shoaling at Station 22+50 from September 2008 to August 2009

The same occurrence can be observed in the Reaches 8 and 9, along the inner bank of Jarvis Turn. Examining the cross section at Station 375+44 (Figure 3-8) it can be seen that on the inner bank (left side) sediment has been steadily accumulating from 2009 through 2015 (see the area in the red box). Despite the apparent shoal at the bottom of the channel, no loss of sediment is occurring on the upper portion of the side slope. This may indicate that the sediment accumulation at the bottom of the channel may be sourced from elsewhere in the channel. It could be from another part of the bay, or even from erosion on the outer bend of Jarvis Turn. After channel modification, this area is still expected to be depositional, with material depositing on the bottom of the channel, appearing to flatten the slope.

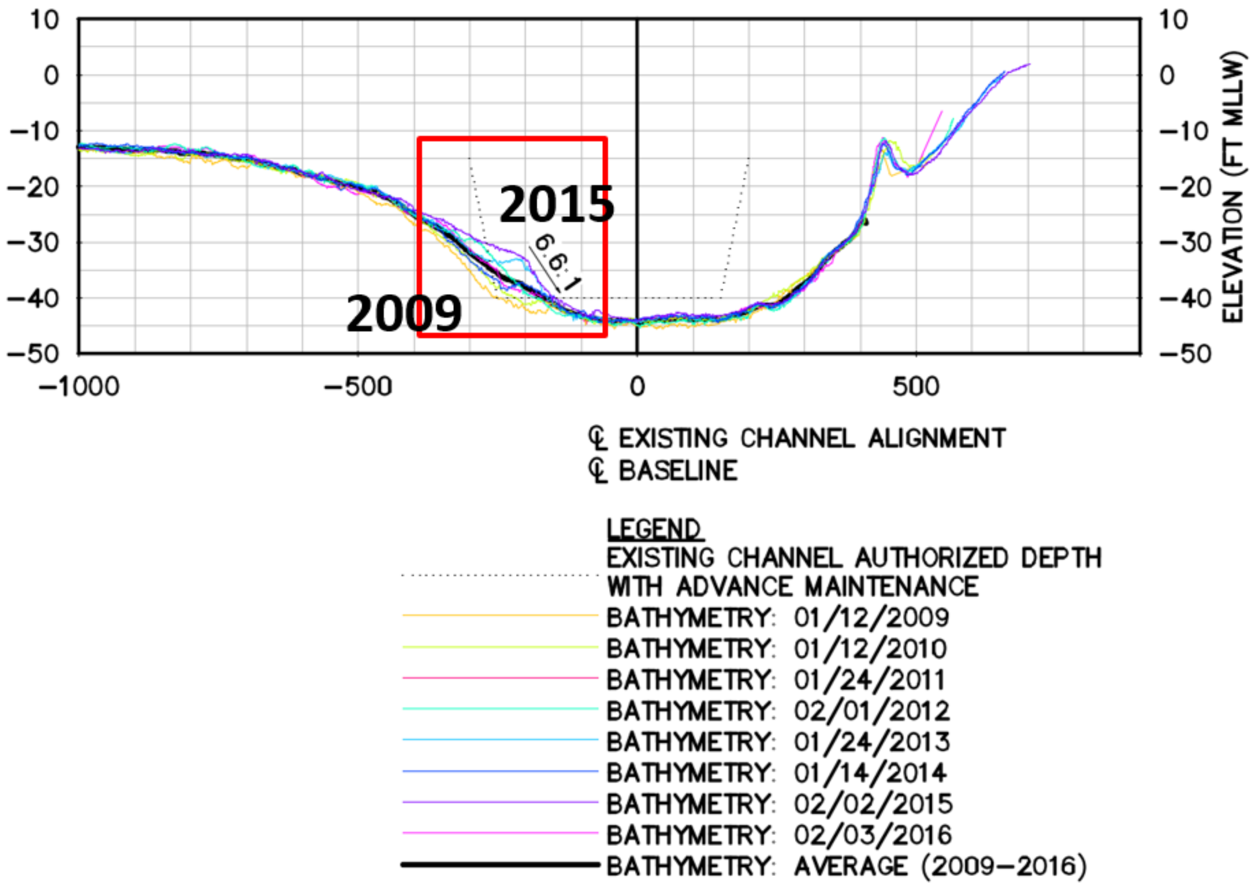


Figure 3-8
Deposition on the Inner Slope of Station 375+44 (Approx. RM 7.1)

The shoaling process causes an accumulation of sediment within the authorized channel, without a loss of sediment from the upper portion of the side slope. In this way, the shoaling process differs from the methods described in this document. Significant shoaling has been observed to occur on time scales shorter than 1 year, such that the authorized depth and width can only be maintained by an active maintenance dredging program. These locations are in a dynamic equilibrium state, in which the annual shoaling is maintained by the dredging program. Therefore, the side slope equilibration will not outpace deposition on the slopes.

The presence of shoaling in the measured side slope angle is important to note because of how side slope equilibration is defined in Section 3.2.1 as, “slope angles that are likely to develop in the channel as a result of capital dredging.” However, where the measured angles result from shoaling, they no longer represent the result of capital dredging. The ultimate goal of this analysis is to estimate a side slope angle that emerges from the channel toe and daylights along the adjacent bathymetry – changes to shoaling are considered in Sub-Appendices 3 and 4 (*Estuarine Dynamics* and *Offshore and Entrance Dynamics*). Moreover, the measured side slopes in shoaling reaches originate 5-10 ft above the channel boundary, not the dredged channel toe. To evaluate the effects of the project, these angles are being used as the future equilibrium side slopes angles and form the basis for analysis of effects.

3.2.4.4.2 Areas of Erosion or Scour

The outer slopes of Reach 8 have shown to be erosional in nature. In these areas, the ongoing erosion removes sediment from the bottom of the channel, effectively steepening the side slopes after they are dredged. In this area, erosion occurs independent of dredging, and appears to remove material from the bottom of the slope, steepening the channel. Therefore, the channel is expected to be stable at the initial slope or steeper.

This process is particularly evident at Station 345+54 (Figure 3-9), where the slope evolved from 10:1 to 3:1 over 10 years because of erosion. After channel modification, erosion is expected to continue; therefore, it is reasonable to expect that the side slopes may become even steeper after dredging. The median side slopes are used as the basis for future equilibrium side slopes for purposes of estimating capital dredge volumes as well as O&M dredge volumes and analysis of project effects.

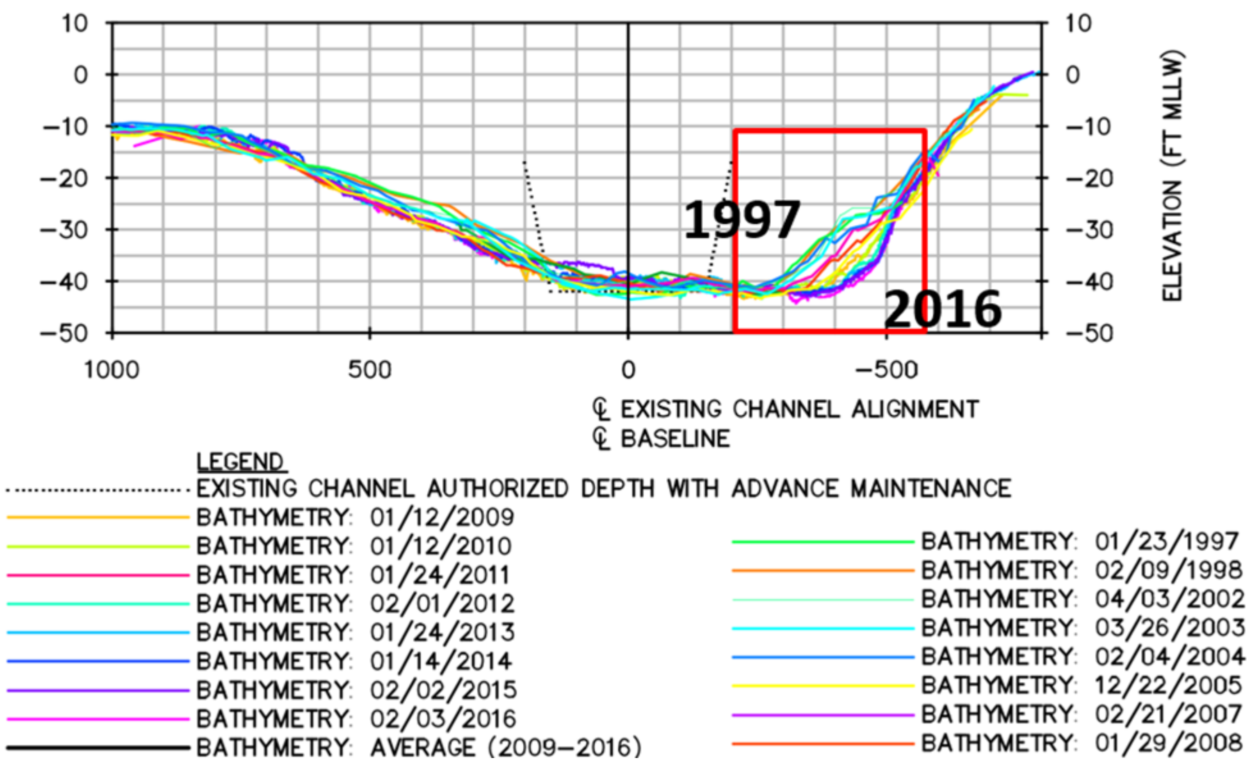


Figure 3-9
Erosion of the Outer Slope at Station 345+54 (Approx. RM 6.6)

The outer slopes of Reach 4 may also be erosional based on the distribution of currents around the bend. The lack of sand in this area may be due to continued erosion.

3.3 Methodology for Geotechnical Global Stability of the Equilibrium Side Slopes Adjacent to Infrastructure

The Port estimated the future equilibrium side slopes controlled by factors other than typical geotechnical slope stability (e.g., channel morphology) as presented in previous sections of this report. Therefore, it is prudent to evaluate the long-term effects of the proposed channel modifications on the global stability of select infrastructure along the channel. Select infrastructure includes the North Jetty, the South Jetty, the relic trestle located downstream of approximately RM

2, the berth and access channel at approximately RM 5.0, the Arago dock in Empire at approximately RM 5.3, the North Bay Marine Industrial Site T-dock located at approximately RM 5.6, the pile dike at RM 6.8, and the Southwest Oregon Regional Airport. The locations of the select infrastructure with respect to the slope stability sections used in the analyses are identified in Attachment A, Figures A-1 through A-7 as "Slope Stability Section Location.". The evaluations performed as part of the analyses are shown in Attachment A, Figures A-8 through A-16 and were conducted using the future equilibrium side slopes.

The following infrastructure locations along the channel were evaluated for inclusion into the global stability evaluation; however, due to distance of the channel infrastructure from the extent of the future equilibrium side slopes, further analysis was not completed: The North Jetty root located between approximately RM 1 and 2, the Coos Bay waste water treatment plant located at approximately RM 4.7, the Cape Arago Dock/Sause Bros., the Southport Forest Products Sawmill and Barge Facility docks located at approximately RM 6.2, and the pile dikes located at RM 6.4, 6.6, 7.0 and 7.3.

The assessment analyzed the relative change in global static slope stability of the infrastructure before and after the proposed channel modifications and was based on calculating the relative change in the global factor of safety between the two scenarios. Specifically, the assessment evaluated the critical failure surface at or near the existing infrastructure location before the proposed channel modifications and compared them to the critical failure surface evaluated after channel modifications. Wave loading and other hydrodynamic processes that can impact slopes are addressed in other sections of this report.

The global static slope stability evaluation was based on the channel and upland geometry and characteristics presented in various reports and drawings, including 1) the project plan drawings showing the PA channel, rock surface, bathymetry, topography, and existing or proposed rock aprons (see Sub-Appendix 8 - Drawings); 2) the future equilibrium side slopes presented in other sections of this report; and 3) historical drawings showing the bottom elevations of the jetty rock for both the North Jetty and South Jetty (USACE, 1924a, b). Sediment/rock properties within each reach were based on geotechnical data presented in the geotechnical data report for the project (See Sub-Appendix 5 – Geotechnical Data). It should be noted that borings were not completed in Reach 1 through Reach 3 primarily due to ocean conditions; therefore, the sediment properties were assumed in these locations. A peat layer and a silt layer were included in the global static slope stability assessment at the Southwest Oregon Regional Airport. The assessment of the peat and silt layers showed that the peat layer controlled the static slope stability. The assessment analyzed a total of eleven channel cross sections: three at the North Jetty, two at the South Jetty, one at the relic trestle, one at the berth and access channel, one at the Arago dock, one at the North Bay Marine Industrial Site T-dock, one at the pile dike at RM 6.8, and one at the Southwest Oregon Regional Airport. The cross section(s) for each infrastructure location were selected based on the most proximal location of the channel to the infrastructure and/or the greatest initial dredge cut height in sand. The analyses assumed the channel water level was coincident with the MLLW elevation. Table 3-1 summarizes the cross-section geometry and assumed sediment properties used in the assessments.

Table 3-1
Global Static Slope Stability Parameters

Infrastructure	Location (River Mile)	Long-Term Equilibrium Slope, H:V	Material Type	Unit Weight, pcf	Cohesion, psf	Friction Angle, degrees
North Jetty	RM 0.27, 0.38, 0.53	15:1	Jetty Rock and Rock Apron	130	0	50
			Very Loose Sand	115	0	30
			Rock	120	Infinite Strength[1]	Infinite Strength[1]
South Jetty	RM 0.38 and RM 0.49	22:1	Jetty Rock and Rock Apron	130	0	50
			Very Loose Sand	115	0	30
			Rock	120	Infinite Strength[1]	Infinite Strength[1]
Relic Trestle	RM 2.05[2]	5:1	Loose Sand	115	0	32
			Rock	120	Infinite Strength[1]	Infinite Strength[1]
Berth and Access Channel	RM 4.95	5:1	Medium Dense Sand	115	0	34
			Very Soft Rock[2]	120	3000	0
Arago Dock	RM 5.36	5:1	Dense Sand	115	0	36
			Very Soft Rock[2]	120	3000	0
T-Dock	RM 5.55	5:1	Medium Dense Sand	115	0	34
			Very Soft Rock[2]	120	3000	0
Pile Dike	RM 6.80	6:1	Rock Apron	130	0	50

	Dense Sand	115	0	36
	Rock	120	Infinite Strength[1]	Infinite Strength[1]

Infrastructure	Location (River Mile)	Long-Term Equilibrium Slope, H:V	Material Type	Unit Weight, pcf	Cohesion, psf	Friction Angle, degrees
Regional Airport	RM 8.02	5:1[3]	Very Loose Sand or Silt	115	0	30
			Loose Sand	115	0	32
			Medium-Dense Sand	115	0	34
			Dense Sand	115	0	36
			Peat	105	100	0
			Rock	120	Infinite Strength[1]	Infinite Strength[1]

Notes:

1. *Infinite strength is used in the software program Slide to represent a slip surface exclusion zone through which slip surfaces are not allowed to pass. Evaluation of rock stability was completed separately.*
2. *The Bastendorff formation was modeled as a soil to more conservatively evaluate global stability.*
3. *Location is at support platform for the relic trestle. At locations immediately upstream and downstream, the rock is at the surface without sand overburden. See Sub-Appendix 8 - Drawings and other sections of this report.*
4. *Long-term equilibrium slope will be maintained at 5:1. See Sub-Appendix 8 - Drawings and other sections of this report*.

The software program Slide (developed by Rocscience, Inc., of Toronto, Canada) was used to support the slope stability analyses. The analyses evaluated global stability at each infrastructure location using Spencer’s and Morgenstern-Price’s methods of slices, which satisfy both force and moment equilibrium. The program computes a limit equilibrium factor of safety, defined as the ratio of the forces and moments resisting movement to the forces and moments driving movement of the sediment mass. In a limit equilibrium approach, a slope with a factor of safety greater than 1.0 is considered stable, whereas a slope with a factor of safety equal to 1.0 is at incipient failure. Although this approach does not allow for a direct assessment of the margin of safety against failure, the risk of slope failure is considered to increase as the factor of safety approaches 1.0. A factor of safety less than 1.0 implies the sediment mass is not in equilibrium. A factor of safety of 1.5 is acceptable for jetties for normal long-term loading conditions (USACE, 2003).

The global slope stabilities of the infrastructure locations were analyzed using circular and bilinear failure surfaces. The bilinear failure surfaces represented a failure plane applicable to the Infinite Slope method and evaluated a failure surface through an assumed laterally extensive peat layer within the offshore zone located north of the Southwest Oregon Regional Airport. The bilinear failure surfaces had a comparable calculated factor of safety to the circular surfaces for the North Jetty, the South Jetty, the relic trestle, the berth and access channel, the Arago Dock, the North

Bay Marine Industrial Site T-dock, and the pile dike at RM 6.8. Section 4.1 and Attachment A present the results of the slope stability analyses. For brevity, only circular failure surfaces are presented, with exception to the analysis at the Southwest Oregon Regional Airport, where a bilinear surface is presented.

The geotechnical evaluation excludes assessments of seismic slope stability and the potential effects of seismic hazards on the proposed channel and adjacent infrastructure. However, the USACE has indicated they have completed screening evaluations of the jetties that indicated a risk of seismically induced liquefaction and slope movement during a magnitude 9 Cascadia Subduction Zone seismic event (personal communication, USACE 2018). The proposed channel modifications are not anticipated to significantly change the seismic susceptibility of the North Jetty.

4. RESULTS OF THE SIDE SLOPE ANALYSIS

This section presents the results for the individual evaluations completed as part of the channel side slope analysis. As described previously, the results presented below are based upon the following objectives:

- Evaluate the geotechnical stability of the initial proposed dredge cut slopes during and at the completion of capital dredging;
- Estimate the range of slope angles to which the side slopes will equilibrate following the conclusion of capital dredging;
- Estimate the anticipated duration for the side slope equilibration process;
- Estimate the range of dredge quantities associated with the channel side slope equilibration for the PA for the purpose of calculating project costs and cost sharing;
- Estimate the physical zone of equilibration of the channel modifications following the equilibration process; and
- Evaluate the effect of the equilibrium side slopes on adjacent resources.

4.1 Geotechnical Stability of the Initial Dredge Cut

The approach used in the evaluations for geotechnical stability of the initial dredge cuts is presented in Section 3.1 of this report. As described in Section 3.1, static slope stability and the risk of static liquefaction were considered to evaluate the geotechnical stability of the initial dredge cuts. The results of this analysis are used to create the *constructed condition* of the dredged channel, which represents the immediate, post-construction condition.

The majority of the static slope stability analyses assumed a proposed initial dredge cut of 3:1 in sediment and 1:1 in rock, in combination with the assumed geotechnical conditions for each reach, result in acceptable factors of safety. Therefore, the below subsections do not address the static slope stability of the initial dredge cut on a reach-by-reach basis, and the conclusions are focused on the risk of static liquefaction. It should be noted that the proposed 2H:1V cuts in rock and sand in the proposed berth and access channel are the primary exception and this configuration is described separately.

The static liquefaction analyses indicate a proposed initial dredge cut of 3:1 for sediment, in combination with the assumed geotechnical conditions, results in a relatively high risk of dredging-induced static liquefaction and flow failure within Reach 1 through Reach 3 and the left bank of Reach 8 and Reach 9. Reduced initial dredge cuts and stepped, top-down dredging techniques are recommended to lower the risk of static liquefaction and flow failure during dredging within these reaches. It should be emphasized that the recommended initial cut slopes are controlled by the risk of static liquefaction rather than static slope stability. The following subsections present the results of the static liquefaction analyses for the initial dredge cuts within Reach 1 through Reach 9 and include recommendations for initial dredge cut slopes and construction of the cut slopes within each reach.

Table 4-1 summarizes the recommended initial dredge cut slopes for the constructed condition based on the static liquefaction analyses.

Table 4-1
Recommended Initial Dredge Cut Slopes in Soil

Reach	**Cross Section Location**		**Initial Cut Slope for the Constructed Condition (H:V)**
	Bank	**River Mile**	
Reach 1	Left Bank	RM 0.00	4:1
	Right Bank	RM 0.11	4:1
Reach 2	Left Bank	RM 0.15	4:1
	Right Bank	RM 0.11	4:1
Reach 3	Left Bank	RM 0.49	4:1
	Right Bank	RM 0.55	4:1
Reach 4	Left Bank	RM 1.25	3:1
	Right Bank	RM 1.21	3:1
Reach 5	Left Bank	RM 4.47	3:1
	Right Bank	RM 4.39	3:1
Reach 6	Left Bank	RM 4.55	3:1
	Right Bank	RM 5.23	2:1*
Reach 7	Left Bank	RM 6.40	3:1
	Right Bank	RM 5.68	3:1

Reach	Cross Section Location		Initial Cut Slope for the Constructed Condition (H:V)
	Bank	River Mile	
Reach 8	Left Bank	RM 6.63	4:1
	Right Bank	RM 6.59	3:1
Reach 9	Left Bank	RM 7.50	5:1
	Right Bank	RM 7.99	3:1

*See discussion in Reach 6

4.1.1 Reach 1

Reach 1 extends from the proposed limit of advanced maintenance at RM - 0.85 to just offshore of the relic North Jetty head at RM 0.1. No geotechnical information is available in Reach 1 to evaluate the relative density of the assumed sand sediment. Therefore, the analysis assumes very loose sand within this reach to evaluate the static slope stability and the risk of static liquefaction and flow failure. The analysis indicates a proposed initial dredge cut of 3:1, in combination with the assumed geotechnical conditions, results in a relatively high risk of static liquefaction and flow failure. A reduced initial dredge cut of 4:1 is therefore recommended, which will result in a significantly lower risk of flow failure during dredging.

Due to the significant risk of static liquefaction and flow failure, the initial cut slope should be dredged using a top-down dredging approach. In general, slope angles for the initial dredge cut are typically better controlled when the dredging is completed in a stepped, top-down manner and the dimensions of the cuts and benches are limited.

4.1.2 Reach 2

Reach 2 is located between the jetty tips and extends from the relic jetty head at RM 0.1 to the location of the present jetty head at RM 0.3. No geotechnical information is available in Reach 2 to evaluate the relative density of the assumed sand sediment. Therefore, the analysis assumes very loose sand within this reach to evaluate the static slope stability and the risk of static liquefaction and flow failure. The analysis indicates a proposed initial dredge cut of 3:1, in combination with the assumed geotechnical conditions, results in a relatively high risk of static liquefaction and flow failure. A reduced initial dredge cut of 4:1 is therefore recommended, which will result in a significantly lower risk of flow failure during dredging.

Due to the significant risk of static liquefaction and flow failure, the initial cut slope should be dredged using a top-down dredging approach. In general, slope angles for the initial dredge cut are typically better controlled when the dredging is completed in a stepped, top-down manner and the dimensions of the cuts and benches are limited.

4.1.3 Reach 3

Reach 3 is located between the jetties. It extends from the present jetty head at RM 0.3 to the start of the Entrance Turn at RM 0.9. No geotechnical information is available in Reach 3 to evaluate the relative density of the assumed sand sediment. Therefore, the analysis assumes very loose sand within this reach to evaluate the static slope stability and the risk of static liquefaction and flow failure. The analysis indicates a proposed initial dredge cut of 3:1, in combination with the assumed geotechnical conditions, results in a relatively high risk of static liquefaction and flow failure. A reduced initial dredge cut of 4:1 is therefore recommended, which will result in a significantly lower risk of flow failure during dredging.

Due to the significant risk of static liquefaction and flow failure, the initial cut slope should be dredged using a top-down dredging approach. In general, slope angles for the initial dredge cut are typically better controlled when the dredging is completed in a stepped, top-down manner and the dimensions of the cuts and benches are limited.

Rock is likely present within this reach near Guano Rock and is known to be present at or near the mudline in this vicinity. Rock removal techniques may also increase the risk of liquefaction occurring where loose sand is present over rock. Therefore, the sand slopes above the rock should be dredged prior to rock dredging. Rock dredging techniques could likely induce localized sloughing of the sand side slopes above the rock, which would result in somewhat shallower slopes than the recommended static slope angle.

4.1.4 Reach 4

Reach 4 encompasses the Entrance Turn from RM 0.9 to RM 2.0. Based on the available geotechnical information, the channel side slopes in Reach 4 primarily consist of loose to dense sand. Therefore, the analysis assumes loose sand within this reach to evaluate the static slope stability and the risk of static liquefaction and flow failure. The analysis indicates that a proposed initial dredge cut of 3:1, in combination with the assumed geotechnical conditions, results in a relatively low risk of static liquefaction and flow failure. Therefore, the initial dredge cut of 3:1 is reasonable for typical dredging techniques.

Rock is present within portions of this reach. Rock removal techniques may also increase the risk of liquefaction occurring where loose sand is present over rock. Therefore, the sand slopes above the rock should be dredged prior to rock dredging. Rock dredging techniques could likely induce localized sloughing of the sand side slopes above the rock, which would result in somewhat shallower slopes than the recommended static slope angle.

4.1.5 Reach 5

Reach 5 encompasses the portion of the channel directly upstream of the Entrance Turn that is underlain by rock. It extends from RM 2.0 to RM 4.5. Based on the available geotechnical information, the channel side slopes in Reach 5 are mantled with medium-dense to dense sand. Therefore, the analysis assumes medium-dense sand within this reach to evaluate the static slope stability and the risk of static liquefaction and flow failure. The analysis indicates the limited overburden, in combination with the assumed geotechnical conditions, results in a relatively low risk of static liquefaction and flow failure. Therefore, an initial dredge cut assumptions of 3:1 in sand is reasonable for typical dredging techniques.

Rock is likely present within much of this reach. Where sand is present over rock, rock dredging techniques could induce localized sloughing of the sand side slopes above the rock, which would result in somewhat shallower slopes than the recommended static slope angle.

4.1.6 Reach 6

Reach 6 is located from RM 4.5 to RM 5.6 and includes the new berth and access channel on the right bank. Based on the available geotechnical information, the channel side slopes in Reach 6 primarily consist of medium-dense to dense sand overlying Bastendorff formation. Therefore, the analysis assumes medium-dense sand within this reach to evaluate the static slope stability and the risk of static liquefaction and flow failure. The analyses in this reach considered both 3:1 and 2:1 initial cuts in for the primary channel and steeper berth and access channel, respectively. The analysis indicates a proposed initial dredge cut of 3:1, in combination with the assumed geotechnical conditions, results in a relatively low risk of static liquefaction and flow failure. The 2:1 cuts are proposed in areas with a large concentration of borings through the sand layers. With the exception of one near-ground-surface sample, all samples within the sand unit exceed a relative density of 60%, with most exceeding 80%. The analysis results in a relatively low risk of static liquefaction and flow failure, and the research presented in Raaijmakers 2005 asserts that static liquefaction and flow failure is typically not considered for soils with a relative density greater than 60%. Therefore, an initial dredge cut of 2:1 is reasonable provided top-down dredging techniques are used.

The rock in this section consisted of the Bastendorff formation and was encountered relatively shallow on the left bank compared to the right bank. Where sand is present over rock, rock dredging techniques could induce localized sloughing of the sand side slopes above the rock, which would result in somewhat shallower slopes than the recommended static slope angle.

4.1.7 Reach 7

Reach 7 is located directly upstream of Reach 6, from RM 5.6 to RM 6.4. Based on the available geotechnical information, the channel side slopes in Reach 7 primarily consist of medium-dense to dense sand. Therefore, the analysis assumes medium-dense sand within this reach to evaluate the static slope stability and the risk of static liquefaction and flow failure. The analysis indicates a proposed initial dredge cut of 3:1, in combination with the assumed geotechnical conditions, results in a relatively low risk of static liquefaction and flow failure. Therefore, an initial dredge cut of 3:1 is reasonable for typical dredging techniques.

Rock is likely present over most of the reach, but drops-off before reaching Reach 8. Where sand is present over rock, rock dredging techniques could induce localized sloughing of the sand side slopes above the rock, which would result in somewhat shallower slopes than the recommended static slope angle.

4.1.8 Reach 8

Reach 8 extends from RM 6.4 to RM 7.2, encompassing Jarvis Turn. Reach 8 is separated into right and left banks in the following subsections to distinguish between the differing conditions in these areas.

4.1.8.1 Right Bank

Based on the available geotechnical information, the right bank side slopes in Reach 8 primarily consist of medium-dense to dense sand. Therefore, the analysis assumes medium-dense sand within this reach to evaluate the static slope stability and the risk of static liquefaction and flow failure. The analysis indicates a proposed initial dredge cut of 3:1, in combination with the assumed geotechnical conditions, results in a relatively low risk of static liquefaction and flow failure. Therefore, an initial dredge cut of 3:1 is reasonable for typical dredging techniques.

4.1.8.2 Left Bank

Based on the available geotechnical information, the left bank channel side slopes in Reach 8 consist of loose to dense sand. The left banks of Reaches 8 and 9 exist within a similar depositional environment (shoaling area), and very loose sand was encountered within the left bank of Reach 9. Therefore, very loose sand and silt could exist within the left bank of Reach 8 as well. Therefore, the analysis assumes very loose sand within this reach to evaluate the static slope stability and the risk of static liquefaction and flow failure. The analysis indicates a proposed initial dredge cut of 3:1, in combination with the assumed geotechnical conditions, results in a relatively high risk of static liquefaction and flow failure. A reduced initial dredge cut of 4:1 is therefore recommended, which will result in a significantly lower risk of flow failure during dredging.

Due to the significant risk of static liquefaction and flow failure, the initial cut slope should be dredged using a top-down dredging approach. In general, slope angles for the initial dredge cut are typically better controlled when the dredging is completed in a stepped, top-down manner and the dimensions of the cuts and benches are limited.

4.1.9 Reach 9

Reach 9 is the farthest upstream reach, located from the upstream extent of Reach 8 at RM 7.2 to the end of the proposed channel modifications at RM 8.2. The proposed alterations in this reach include a turning basin (left bank) across from Roseburg Forest Products (right bank). Reach 9 is separated into right and left banks in the following subsections to distinguish between the differing conditions in these areas.

4.1.9.1 Right Bank

The right banks of Reach 8 and Reach 9 exist within the same depositional environment (erosion area), and medium-dense to dense sand was encountered within the right bank of Reach 8. It is therefore highly likely that medium-dense to dense sand exists within the right bank of Reach 9 as well. Therefore, the analysis assumes medium-dense sand within the right bank of Reach 9 to evaluate the static slope stability and the risk of static liquefaction and flow failure. The analysis indicates a proposed initial dredge cut of 3:1, in combination with the assumed geotechnical conditions, results in a relatively low risk of static liquefaction and flow failure. Therefore, an initial dredge cut of 3:1 is reasonable for typical dredging techniques.

4.1.9.2 Left Bank

Based on the available geotechnical information, the left bank side slopes in Reach 9 primarily consist of sand of variable density with interlayered peat. Because of the variability of the relatively density and consistency of the soils encountered in the borings B-10, B-37, and B-1-23, our analyses conservatively assumed the generally looser profile observed in B-10. The analysis indicates a proposed initial dredge cut of 3:1, in combination with the assumed geotechnical conditions, results in a relatively high risk of static liquefaction and flow failure. A reduced initial dredge cut of 5:1 is therefore recommended, which will result in a significantly lower risk of flow failure during dredging.

Due to the significant risk of static liquefaction and flow failure, the initial cut slope should be dredged using a top-down dredging approach. In general, slope angles for the initial dredge cut are typically better controlled when the dredging is completed in a stepped, top-down manner and the dimensions of the cuts and benches are limited.

4.2 Future Equilibrium Side Slope Analysis

This section describes the future equilibrium side slope estimates developed within Coos Bay. This includes an overview of the data used, and a reach-by-reach breakdown of the methodologies introduced above to estimate the range of side slopes and the expected future equilibrium side slopes. Section 3.2 presented the methodologies to predict future equilibrium side slopes for purposes of estimating capital dredge quantities. The median measured side slopes are used to formulate the more conservative future equilibrium side slopes used to estimate future O&M dredge volumes and assess potential impacts on adjacent infrastructure. For this more conservative case, the measured median side slope was artificially assumed to originate at the toe of the dredged channel slope. The median measured and future equilibrium side slopes are summarized for each reach in the following sections and summarized in Section 4.3 and Table 4-10.

4.2.1 Data Review

Several sources of bathymetric data are available to measure the side slopes in Coos Bay. These sources include:

- Annual USACE surveys (2000-2016)
- Underwater LiDAR data collected by the Northwest Division, Portland District (NWP) in 2014
- Multi-beam bathymetry data collected by David Evans Associates (DEA) from 2007-2016
- As-built bathymetry by USACE following 1996 channel deepening

The measured median value within each reach is used to bound the equilibrium side slopes. The median was selected because it represents the middle value, and is not influenced by extreme (i.e., flat or steeper than 2:1) side slopes that occur in areas of rapid contractions or expansions in the channel.

4.2.1.1 Annual USACE Surveys

The USACE regularly conducts bathymetric surveys within the navigation channel. The USACE has designated survey areas throughout the channel; the proposed project falls into the USACE survey areas CB1 through CB4 (Figure 4-1). It should be noted that the survey areas defined by USACE are not the same as the reach designations used in this analysis and presented in Section 2.4. The USACE conducts two types of surveys – channel line surveys and cross line surveys. The channel line surveys are conducted along track lines parallel to the channel (as seen in CB1 in Figure 4-1), and cross line surveys are collected along transects perpendicular to the orientation of the channel (as seen in CB2-CB4 in Figure 4-1). The transects used in channel line surveys are located directly over the channel, extending approximately 100 ft in width beyond the authorized channel limits in each direction. The cross-line survey transects extend well beyond the authorized channel limits, almost surveying the entire area between the estuary banks.

The spacing of the survey data (i.e., the distance between adjacent measurements) varies. The track lines for the channel line surveys are approximately 75 ft apart; therefore, the depth measurements across each cross section are essentially spaced at 75 ft. For the cross-line surveys, the soundings collected prior to 2008 (prior to 2009 in Reaches 7 through 9) offer depth measurements spaced at 50 ft on center. However, starting in 2008 (or 2009 in Reaches 7 through 9), USACE began to store "all" surveys, in which the soundings were spaced at 3 ft. Cross line survey transects were initially 300 ft apart, but this spacing was reduced to 200 ft in 2004.

In terms of resolution, temporal variability and availability, the “all” surveys represent the best source of bathymetric data. The high resolution allows precise measurements of the side slope angles, and the temporal variability presents any morphological trends (where applicable). Therefore, these surveys are selected for all bathymetric analysis of Reaches 4 through 9.

In areas that are not undergoing shoaling or erosion, the cross sections based on the “all” surveys do not change significantly from year to year. This may indicate that these cross sections are stable. At each station, a mean cross section was computed, and side slope measurements are based on this mean cross section.

The channel line surveys are limited by their coarse resolution, but do provide temporal variability. They are particularly useful for estimating shoaling from one year to the next (see Figure 3-6 and Figure 3-7). These surveys were also selected for the measurement of side slopes in Reach 1, due to the lack of other data. The annual surveys in Reach 1 vary widely. Therefore, the side slope is computed for each year.

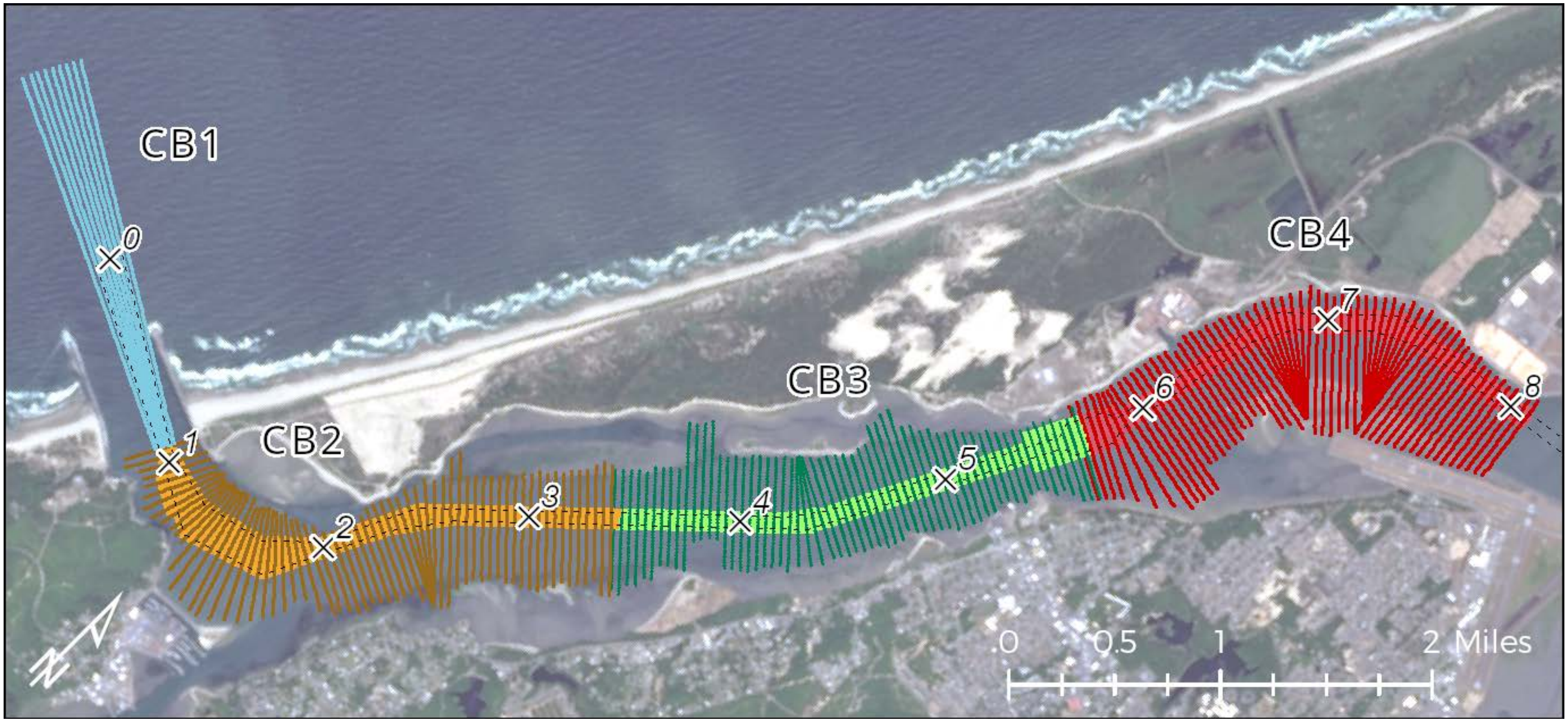


Figure 4-1

USACE Survey Areas in the Vicinity of the Channel Modification Project

4.2.1.2 2014 NWP LiDAR

The 2014 NWP LiDAR data offers dense bathymetry data upstream of RM 0. This survey was conducted October 2-5, 2014. Therefore, it reflects side slopes that have been smoothed after maintenance dredging, but have not experienced significant shoaling. However, there are several gaps in coverage, particularly in the areas adjacent to the North Jetty in Reach 2 and in the deepest portions of Jarvis Turn. Moreover, this survey does not extend far enough offshore to cover Reach 1.

This data set is used to measure the existing side slopes on the southern portion of Reach 2 and in Reach 3. It was selected for these areas because it shows complete coverage and high spatial resolution.

4.2.1.3 DEA Bathymetry

The DEA bathymetry used herein is the same bathymetry used to define the Existing Condition and is described in detail in the Main Report. The dataset is composed of multi-beam data collected during several surveys from 2007-2016; surveys were collected by DEA and by the National Oceanographic and Atmospheric Administration (NOAA). Similar to the 2014 NWP Lidar, this dataset offers dense coverage throughout the channel. The majority of the data that makes up the Entrance Channel (Reaches 2 and 3) was surveyed in the late summer, immediately before O&M dredging. Therefore, the side slopes measured in the Entrance Channel by this survey reflect annual sedimentation, as opposed to the equilibrium side slopes.

This data set is used to measure the side slopes on the northern portion of Reach 2.

4.2.1.4 1996 Channel Deepening As-Built

As-built bathymetry following the 1996 channel deepening were collected by USACE in August 1996 (northern portion of CB2) and March 1997 (the northern portion of CB2 and the southern portion of CB3). The soundings were collected at 5-ft intervals (both parallel and perpendicular to the channel) and provide a precise record of the post-dredge channel in these reaches. Unfortunately, this data is limited to areas where the channel is directly underlain by rock, without sand overburden. Therefore, the side slopes reflected in this bathymetry are indicative of side slopes dredged into rock, and do not provide information on how the dredged sediment will equilibrate after the proposed dredging.

4.2.1.5 Cross Section Analysis

The data described above was provided as point files and digitized to create three-dimensional (3D) surfaces. The digital surfaces essentially interpolate elevations between points, using a linear interpolation scheme. Cross sections were then cut from these digital surfaces; for cross line surveys, cross section transects were taken at the locations where the data were measured; therefore, these cross sections replicate the measured data. For other data types, where the data is sampled evenly throughout the channel, the cross section transect locations do not impact accuracy.

For each reach, the following analysis is presented:

- Survey data used
- Locations of cross sections analyzed

- Representative cross sections, depicting typical features of the reach
- Measured side slopes angles
- Range of expected equilibrium side slopes

The median value of the measured slopes for each bank was taken as the representative slope for the respective banks. This value represents the slope at the middle of the slope distribution, and therefore is not influenced by extreme slopes (i.e., flat or steeper than 2:1) to the extent the mean value would be impacted.

A total of 73 cross sections were considered in this analysis. The locations of these cross sections, along with the reach designations, can be seen in the plan view of the reach in the sections that follow. All cross-section stations are in ft, relative to RM 0.

4.2.2 Reach 1

Reach 1 extends from the proposed limit of advanced maintenance at RM -1 to just offshore of the relic North Jetty head at RM 0.1 (Station 7+00). In this reach, the morphology is dominated by longshore currents that run perpendicular to the channel alignment, which mobilizes sediment from the top of the slopes into the channel.

Analysis of the seabed bathymetry shows that the area on the left of the channel is shallower than the area to the right of the channel. This may be due to a net northward sediment transport, which is interrupted by annual O&M dredging; essentially, this annual dredging removes a sand source that would otherwise be transported north.

4.2.2.1 Measured Side Slopes

The limit of maintenance dredging extends to RM -0.55 for the existing authorized FNC. Therefore, this analysis is confined to this limit. Bathymetry data in Reach 1 is limited; The 2014 NWP data set extended only to station 10+00, and the only high-density data available was the DEA multibeam. As explained in Section 4.2.1.3, the side slopes measured in the Entrance Channel by this survey reflect the presence of shoaling, and are not reflective of side slope equilibration, and do not even show the channel. The USACE channel line surveys from 2000 through 2009 were used. These years were selected because the USACE indicated these surveys correspond to post-maintenance dredge conditions. This period of data provides a reasonable amount of temporal variability upon which a statistical analysis of 2008-2016 slopes may be performed.

The primary advantage of using post-dredge surveys is that channel infill is not present. While this infill contributes to the annual O&M dredge quantities, it does not represent the equilibration side slope condition. Therefore, the post-dredge surveys more accurately depict the equilibrium side slopes. The disadvantage of using these surveys is the coarse resolution.

Three transects are selected for analysis. These transects, located at stations –12+50, -5+00, and 2+50 (Figure 4-2), are selected because the depth of the adjacent seabed differs; therefore, they depict how they represent a variety of cut depths. 30 slope angles, 10 years each at 3 transects, are considered in the statistical analysis. Slopes offshore of -12+50 were not considered because, offshore of this station, the slope cuts are so shallow that slopes are not present – the offshore bathymetry appears flat (i.e., channel is not detectable from surveys). Moreover, using the data from stations –12+50, -5+00, and 2+50 is acceptable for measuring slopes in this area. Further offshore, the bathymetry is deeper and sediment is less affected by waves – as a result, the offshore slopes would be expected to be more stable.

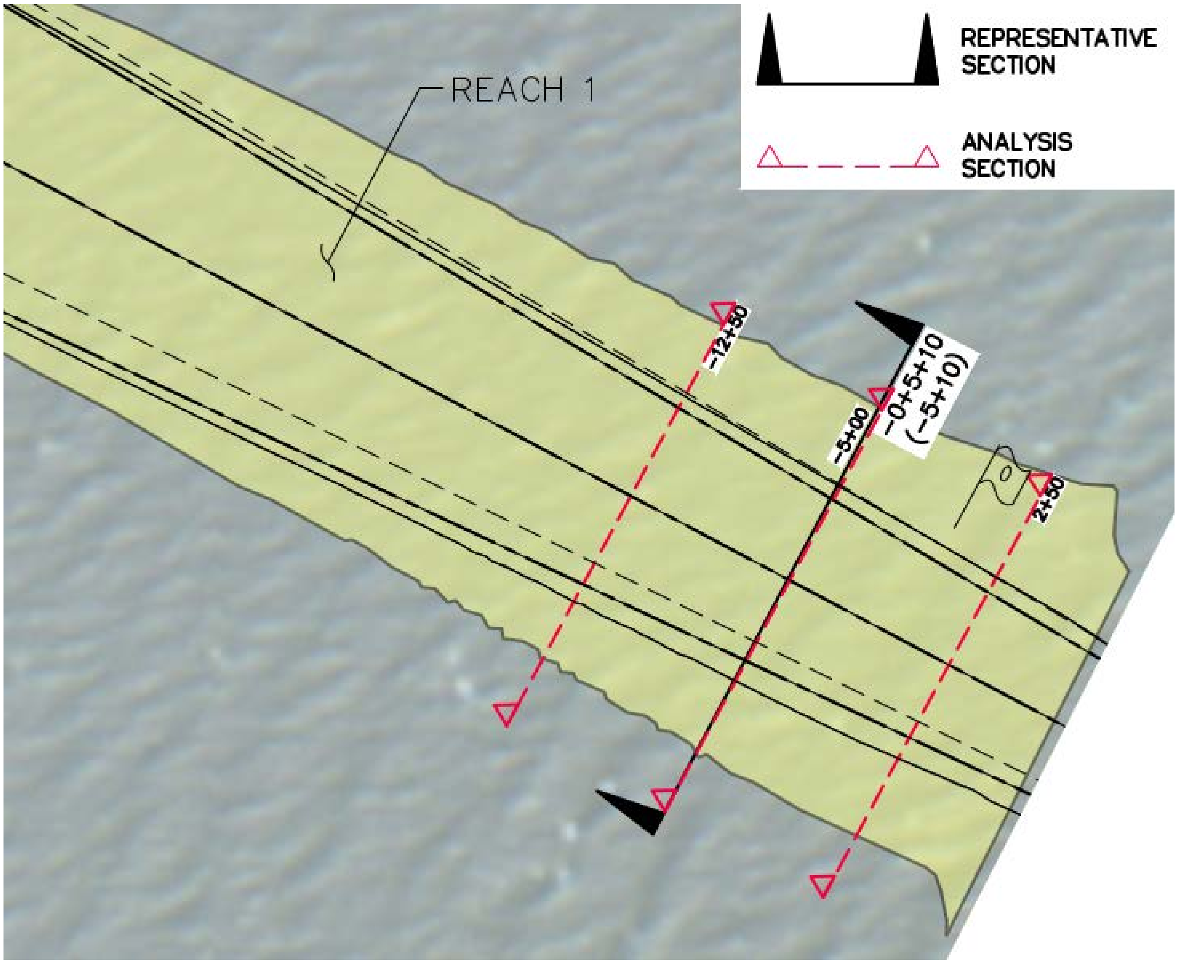


Figure 4-2
Cross Section Locations in Reach 1

shows the measured side slopes on the left and right sides of the channel, tabulated by station and by survey date. To bound the range of side slopes, the entire data set was considered. The measured side slopes range from 5.7:1 to flat, with a median value of 16:1 on the left side and 29:1 on the right side. The measured slopes are much steeper along the left side of the channel and at the landward sections; this may imply that steeper slopes form where the cut is deeper. The prior sentence refers to morphological stability and should not be confused with the conclusions for geotechnical stability. All of the slopes measured are geotechnically stable (i.e., milder than 4:1). It is more likely that the left side appears steeper because of the relationship between cut depth and sounding interval. In this area, soundings are spaced at approximately 75 ft (perpendicular to the center line). So, wherever the slope height is 3 ft or less (which is common), the steepest angle that can be measured is 25:1, even if the actual angle is steeper. The logic is that the two adjacent soundings, which are 75 ft apart, would show a maximum depth differential of 3 ft, so the slope would be 75/3 or 25:1. On the right side, where slopes are shallow, this may cause measurements of mild slopes. Regardless, this represents the best data for which to estimate slopes and was used without any adjustments.

Although **Error! Reference source not found.**Table 4-2 appears to show steeper slopes closer to the shore, this is likely a result of the survey spacing and cut depths (as described above). Therefore, one value is assumed on each side of the channel for the entire length of the channel. There is no infrastructure present in this reach, so in this case the side slope estimates are used for volume calculations. Using one value (in this case, the median) is equivalent to using the midpoint method of performing volume calculations.

The one resource present in this Reach is ODMDS E. However, dredging into ODMDS E is not anticipated to mobilize a significant volume of sediment, as there is no mound of sediment there. Cross sections at Stations -0+57+16 and -0+52+80 in Sub-appendix 13 (*Cross Sections*) show the bathymetry south of the channel, where ODMDS E is located to be flat. A complete description of the effects of dredging in the vicinity of ODMDS E is presented in Sub-appendix 10 (*Dredged Material Disposal Sites*). A representative cross section, measured at Station -5+10, can be seen in Figure 4-3. This cross section is shown at a 10x vertical exaggeration.

Table 4-2
Side Slope Measurements in Reach 1

	Side Slopes (Horizontal:Vertical)		
	Sta. 2+50	**Sta. -5+00**	**Sta. -12+50**
Survey	**Left | Right**	**Left | Right**	**Left | Right**
9/5/2000	6.8|18.5	7.6|30.3	13.2|44.8
8/30/2001	17.9|41.7	21.7|Flat	28.6|97.0
9/9/2002	11.5|12.3	9.7|20.0	25.6|91.9
9/11/2003	9.7|18.5	6.7|11.5	5.7|58.2
8/3/2004	15.6|40.0	18.5|50.0	41.7|Flat
9/27/2005	22.2|37.0	9.4|25.0	16.4|Flat
9/18/2006	18.9|47.6	41.7|76.9	19.6|Flat
9/24/2007	20.8|32.3	16.1|33.3	25.0|Flat
9/8/2008	10.3|22.7	8.8|31.3	10.5|Flat
9/8/2009	15.9|15.2	12.1|18.5	13.9|38.8
Median	**15.7|26.7**	**10.8|30.8**	**17.9|54.1**

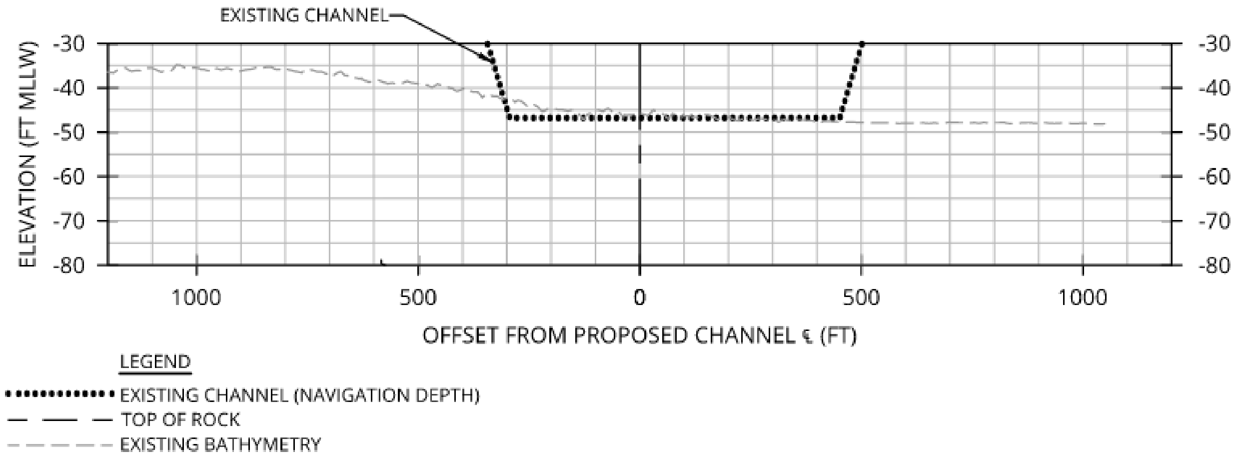


Figure 4-3
Reach 1 Representative Cross Section, Existing Condition (Station -5+10)

4.2.2.2 Substrate Conditions

Rock is not present in this reach; therefore, this reach is likely to equilibrate after dredging. Medium-grained sand is present throughout.

4.2.2.3 Long-Term Morphological Behavior

In Reach 1, longshore currents generate the transport of sediment perpendicular to the alignment of the channel. As the current crosses the channel, the magnitude decreases due to the increase in depth. As a result, some of the sediment is eroded from the top of the slope and deposited into the channel. The calculations proposed by Raaijmakers (2005), shown in Section 3.2.4.1, were used to estimate the equilibrium side slopes.

Input to these calculations include grain size, depth of slope, depth of adjacent seabed, and longshore current velocity. A median grain size of .25 mm was used, based on offshore sediment data from the 1994 EIS (USACE 1994a). The adjacent seabed was assumed to be 43 ft deep (based on survey data), and the depth of the slope was assumed to be 49 ft (the midpoint of the slope). The current was assumed to be 3 fps (DEA 2010). The current speed used in this analysis was measured at RM -1 (since this is the offshore reach) and is not expected to change a result of the project (Sub-appendix 4).

These calculations predict an equilibrium side slope of 9:1 for the with-project conditions and for the specific conditions at Reach 1.

4.2.2.4 Reach 1 – Conclusions

The side slopes in Reach 1, as measured from the toe of the dredge cut, may range from the proposed dredge cut of 4:1 (the constructed condition), to 16:1 on the left side and 29:1 on the right side (the more conservative future equilibrium condition assumed for O&M dredge volumes and assessment of potential impacts to adjacent infrastructure). A long-term morphological analysis within this reach, based on the predicted transport of sediment from the top of the slope into the channel, predicts equilibrium side slope angles of 9:1 (Section 3.2). While the existing slopes are much flatter

A representative cross section of the equilibrium side slopes under the PA shown at a 10x vertical exaggeration can be seen in Figure 4-4; true scale representations of the left and right slopes are depicted in Figure 4-5 and Figure 4-6, respectively. Cross sections in this reach at both exaggerated and true scale for the construction and median side slope angles can be seen in Stations -0+57+16 through 0+5+00 in Sub-Appendix 13 (*Cross Sections)*.

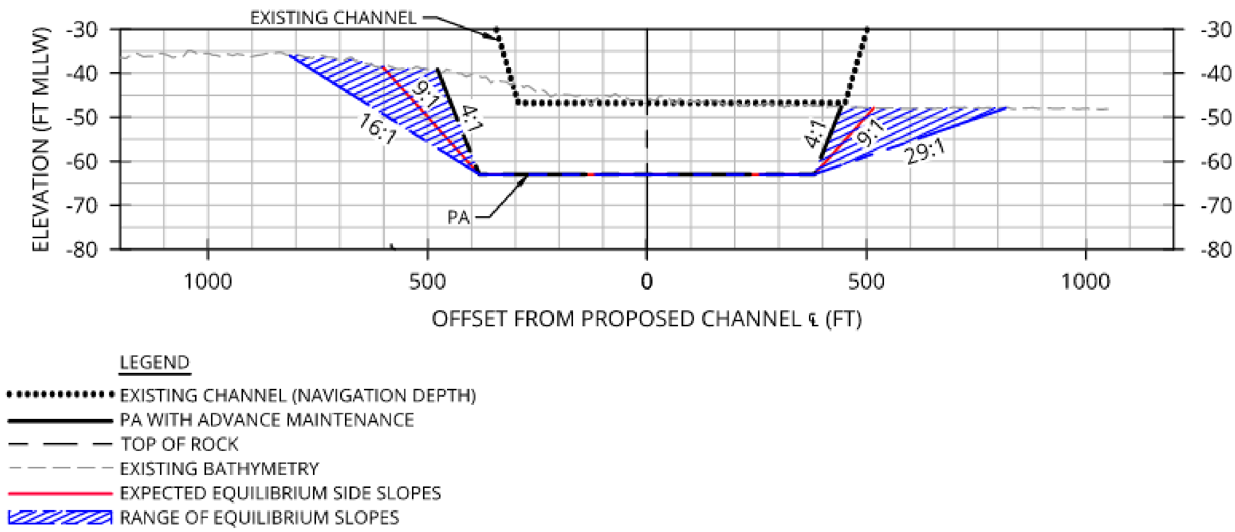


Figure 4-4
Reach 1 Representative Cross Section, PA Condition (Station -5+10)

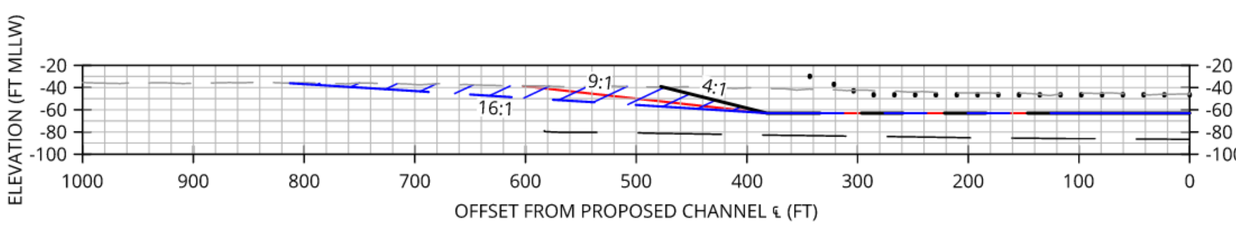


Figure 4-5
Future Equilibrium Side Slope in Reach 1, Left Slope

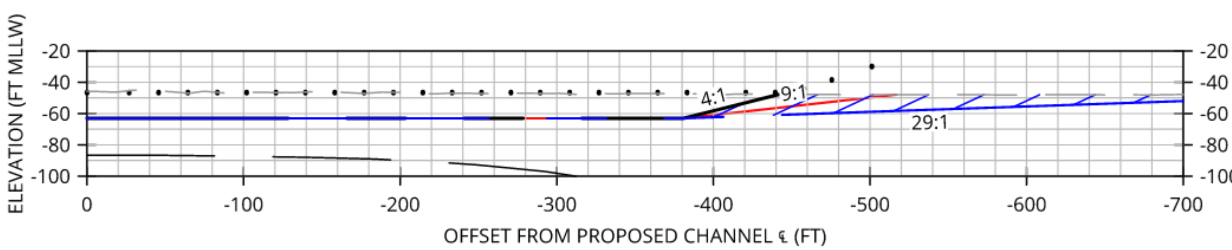


Figure 4-6
Future Equilibrium Side Slope in Reach 1, Right Slope

4.2.3 Reach 2

Reach 2 is located between the jetty tips. The reach extends from the relic jetty head at RM 0.1 (Station 7+00) to the location of the present jetty head at RM 0.3 (Station 17+00). This reach is influenced by current that run both parallel and perpendicular to the alignment of the navigation channel (from north to south) and a counter-clockwise gyre located just south of the North Jetty head. Flood currents are understood to overtop the relic North Jetty head, plunging down the channel side of the relic and causing a scour hole. Dredge records do not indicate that shoaling is significant in this reach (Figure 4-7), although significant shoaling occurs both upstream and downstream of this reach.

The relic jetty head of the North Jetty is immediately adjacent to the right side of the channel. The armor stone forms very steep slopes (on the order of 1.7:1 to 2.5:1). Originally, stone was placed to a depth of -37 ft MLLW. Today, the stone appears to have settled further down. Immediately adjacent to the North Jetty is a sand shoal; high-density multibeam data can be used to distinguish where the relic jetty head (rock) ends and where the shoal (sand) begins. The elevation of this shoal fluctuates through various survey dates, using lower-density survey data. This investigation confirmed that the top of the shoal coincides with the bottom of the steep slope (assumed to be rock). Rock appears to extend to a depth of -45 ft, MLLW.

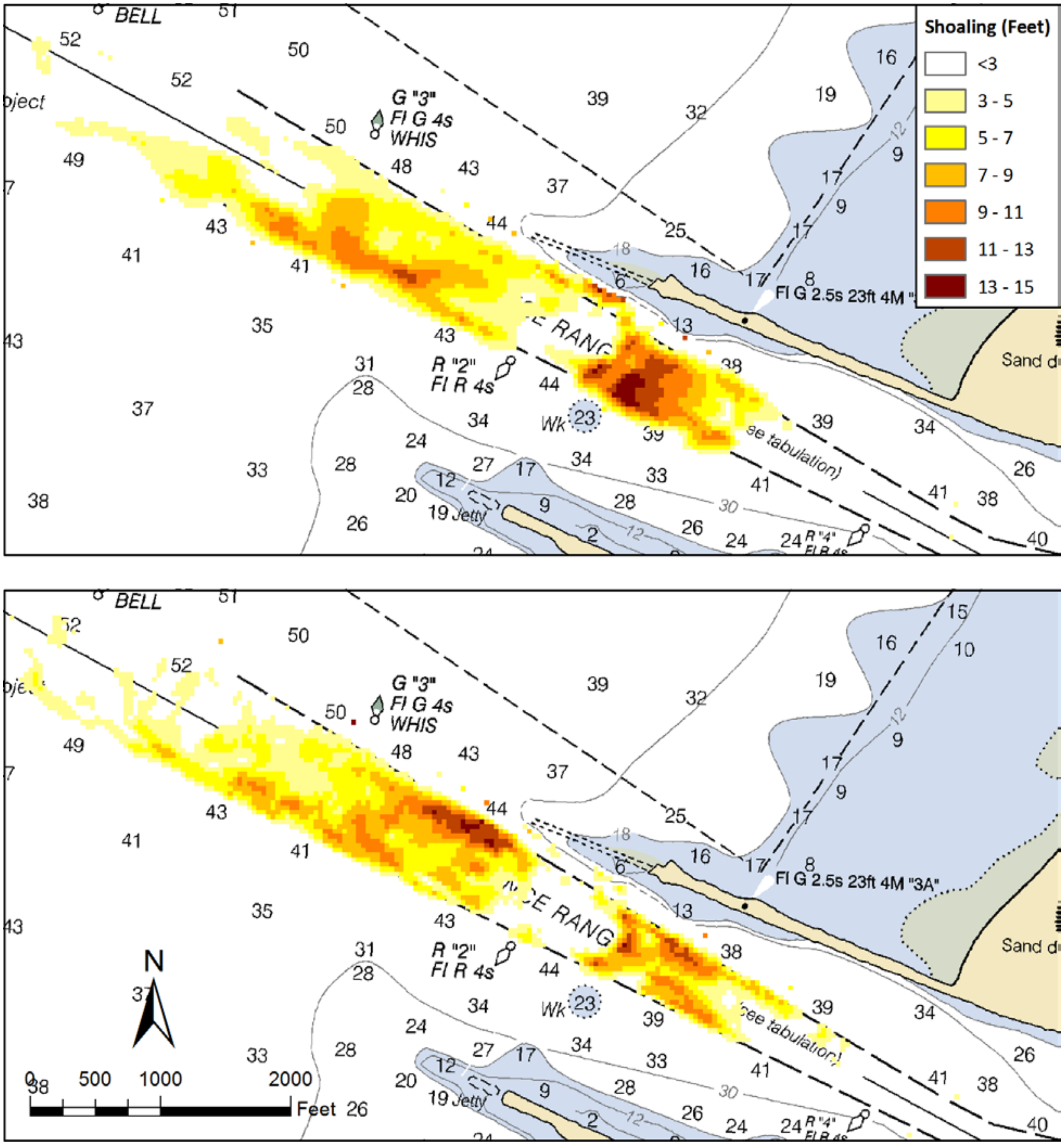


Figure 4-7
Observed Entrance Channel Shoals, 2007 (Top) and 2009 (Bottom)

4.2.3.1 <u>Measured Side Slopes</u>

Analysis of Reach 2 is limited by the amount of bathymetry data available. The bathymetry provided by DEA is available throughout the entire reach, and the 2014 NWP LiDAR bathymetry is available on the left slope and on portions of the right slope. The latter data set was used on the left side of the channel because it represents the stable, non-shoaled slopes. The DEA data set was selected on the right side of the channel for its high resolution. Four cross sections were extracted at the locations shown in Figure 4-8. The measured cross sections at these locations are shown in Table 4-3. The measured side slopes range from 18.4:1 to 33.6:1 and from 10.9:1 to 30.5:1 on the left and right sides of the channel, respectively. The median slopes are 22:1 and 15:1.

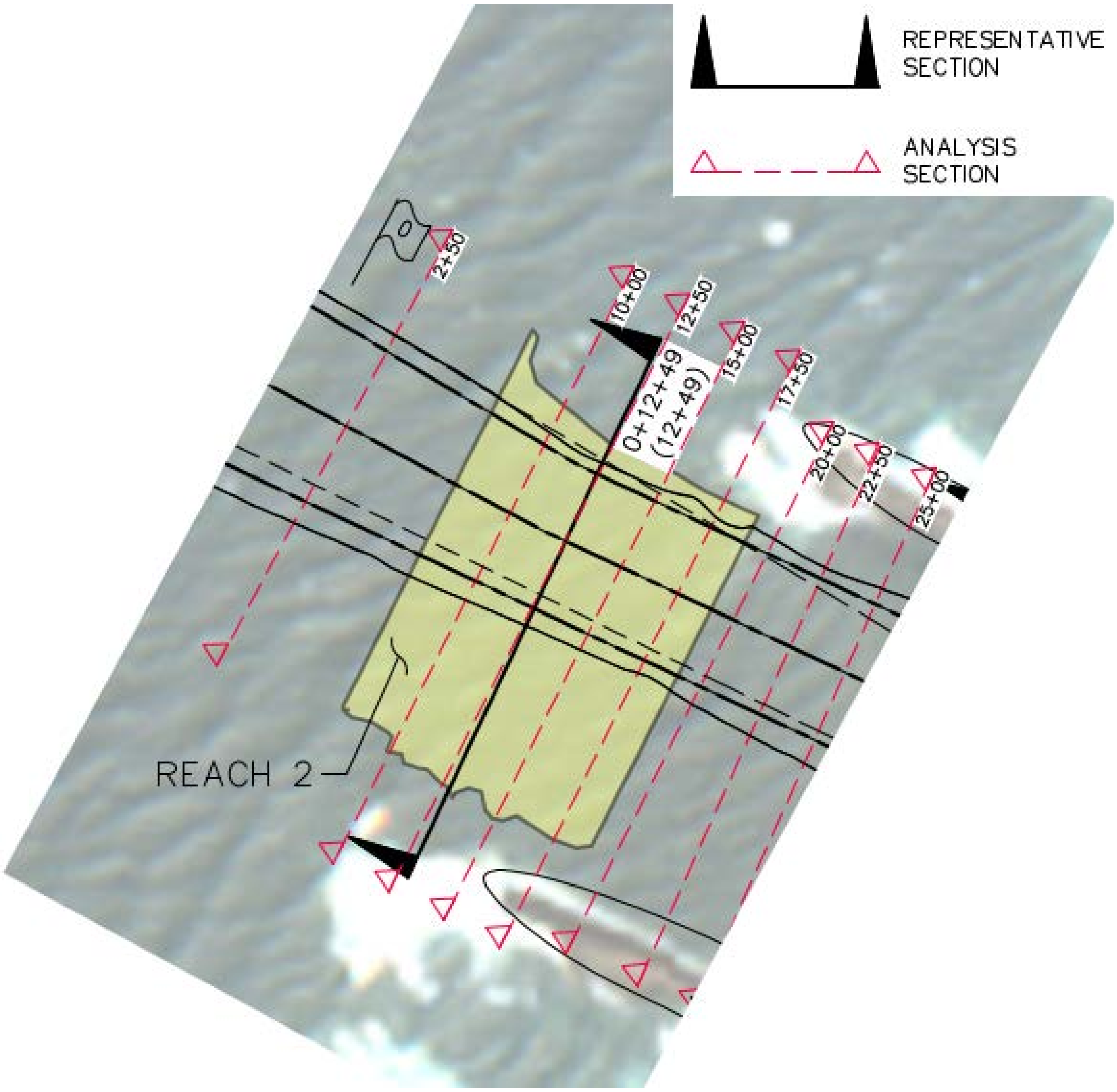


Figure 4-8
Cross Section Locations in Reach 2

Table 4-3
Side Slope Measurements in Reach 2

Station	Left Slope	Right Slope
7+50	18.4	15.8
10+00	18.7	14.2
12+50	33.6	30.5
15+00	26.1	10.9
Median	**22.4**	**15.0**

The representative cross section illustrated in Figure 4-9 shows the steep jetty slopes, and the milder slopes in sediment adjacent to the structures; this cross section is shown with a 10x vertical exaggeration. To the left of the channel, sediment slopes from the South Jetty at -20 ft MLLW to the channel at a slope of 22:1. On the right side of the channel, the sediment slopes from the North Jetty at -45 ft MLLW to the channel at a slope of 15:1.

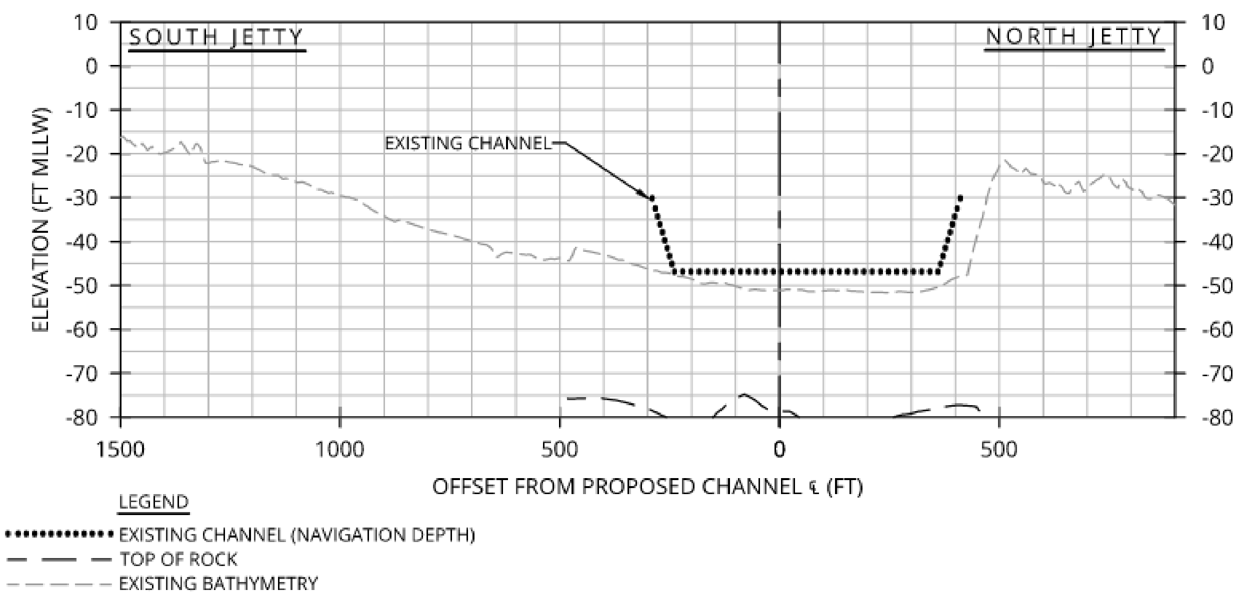


Figure 4-9
Reach 2 Representative Cross Section, Existing Condition (Station 12+49)

4.2.3.2 Substrate Conditions

Rock is not present in this reach; therefore, this reach is likely to equilibrate after dredging. Medium-grained sediment is present throughout this reach.

4.2.3.3 Long-Term Morphological Behavior

Several simultaneous processes occur in Reach 2. Hydrodynamically, the presence of channel-parallel tidal currents, local gyres, and a cross current have been observed; in addition, large waves focus and break at the jetty tips. Morphologically, this area may be erosional for loose sediments as there is a large scour hole on the right side of the channel. However, some sedimentation has been observed. The median measured side slopes are expected to persist into the future. Therefore, the future equilibrium side slope used for estimating the capital dredge volumes will be equal to the median measured side slope assumed for estimates of O&M dredge volumes and potential impacts to adjacent infrastructure. The median measured side slopes are 22:1 on the left side and 15:1 on the right side of the channel.

4.2.3.4 Reach 2 – Conclusion

Side slopes in Reach 2, as measured from the toe of the dredged channel, may range from the constructed condition of 4:1, to the future equilibrium condition of 22:1/15:1 on the left/right sides of the channel. A representative cross section of the equilibrium side slopes under the PA shown at a 10x vertical exaggeration can be seen in Figure 4-10; true scale representations of the left and right slopes are depicted in Figure 4-11 and Figure 4-12, respectively. Cross sections in this reach at both exaggerated and true scale for the construction and median side slope angles can be seen in Stations 0+6+00 through 0+15+00 in Sub-appendix 13 – *Cross Sections* (sheets 14 through 23).

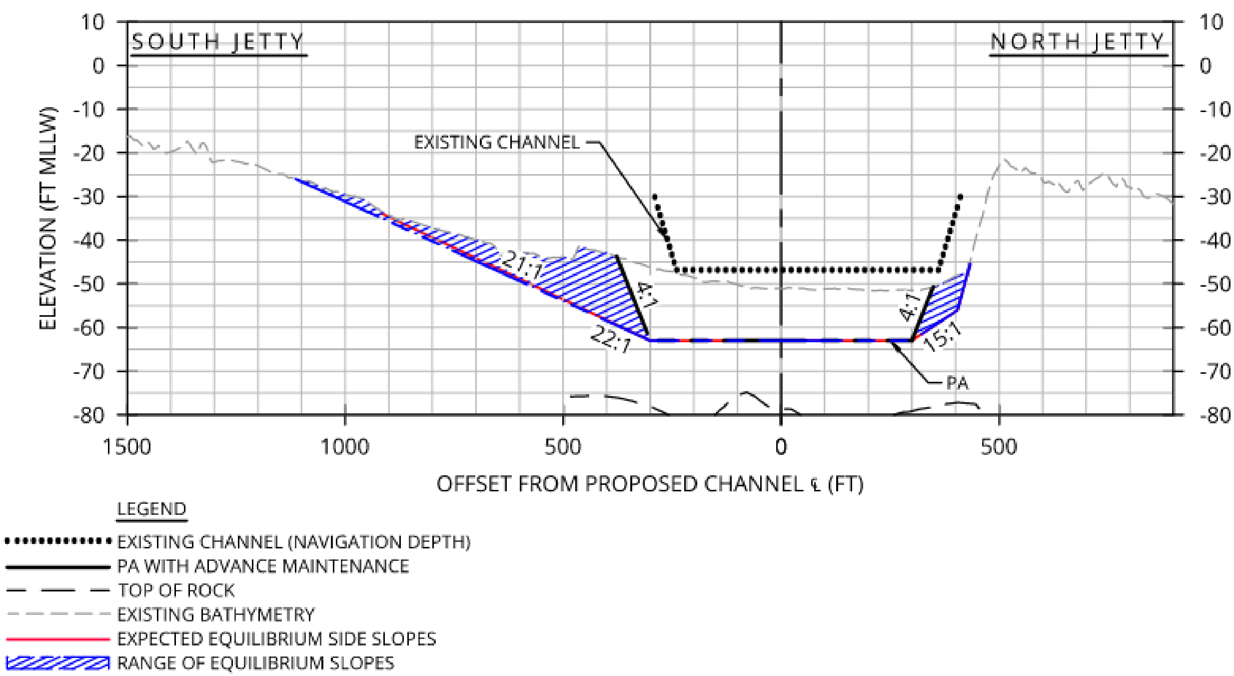


Figure 4-10
Reach 2 Representative Cross Section, PA Condition (Station 12+49)

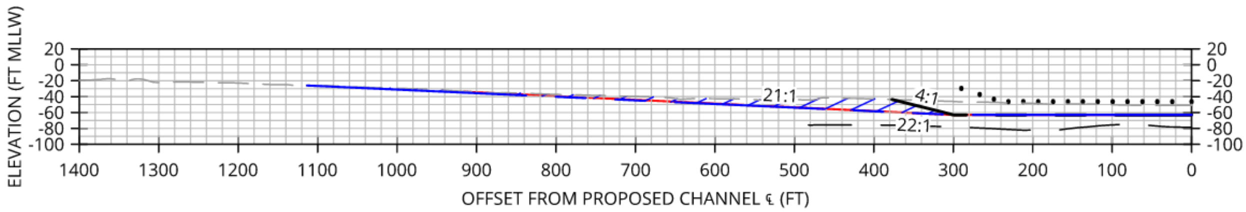


Figure 4-11
Future Equilibrium Side Slope in Reach 2, Left Slope

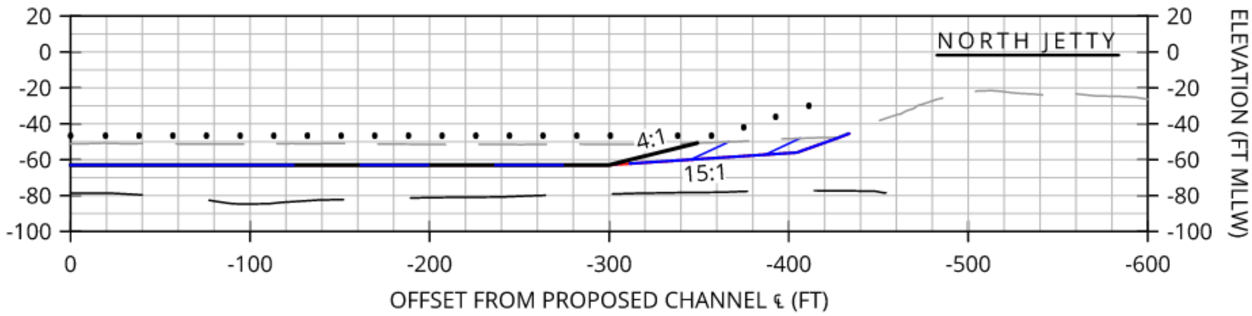


Figure 4-12
Future Equilibrium Side Slope in Reach 2, Right Slope

In this reach, side slope equilibration occurs near the relic jetty head. At present, the USACE has not indicated any intention of building as far out as this point. Essentially, this relic head serves as a rock apron for the current jetty head. If any side slope equilibration results in undercutting of the relic jetty head, it would cause the outer 20 feet to settle. Figure 4-13 shows the potential settling of the relic jetty head. If undercutting were to occur, settling would occur in the area between the existing sand-rock interface (solid red line) and the rock crest (dashed red line); none of the rock above the dashed line would settle.

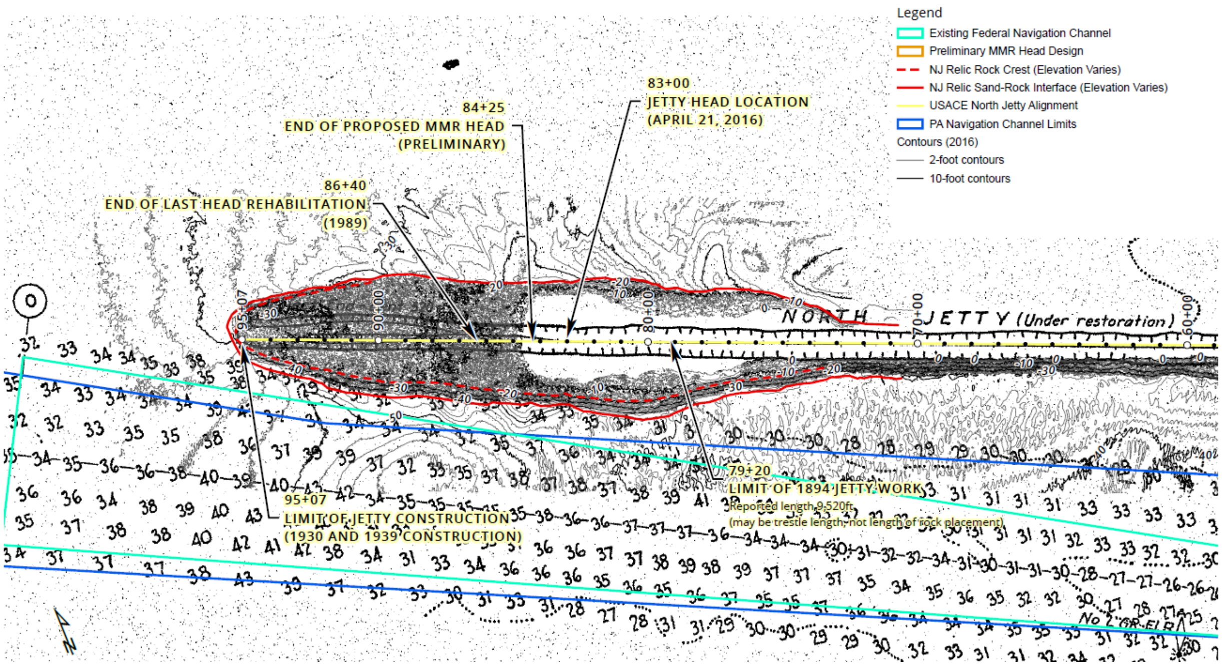


Figure 4-13
Potential Settlement of Relic Head (Background image from USACE, 1939)

In the event that this relic head is undercut, it is anticipated to behave as a rock apron, as described in Van der Hoeven (2002). That is, the outer 20 ft are expected to settle at a 2:1 slope until intersecting the equilibrated side slope, a height of about 10 ft. As a result, the width of the relic may decrease by 20 ft. 3D modeling performed for this project have modeled this relic as a rock apron.

On the other side of the channel, the South Jetty is sufficiently far from the proposed channel that side slope equilibration is not expected to undercut the structure. As shown in Figure 4-10, the side slopes are expected to daylight to the seabed at -28 ft MLLW. By comparison, the base of the structure is located at approximately -25 ft MLLW, based on historic bathymetry surveys (USACE 1924). The channel is likely to daylight with the seabed more than 50 ft away from the structure. Therefore, effects to this structure are not anticipated.

4.2.4 Reach 3

Reach 3 is located between the Coos Bay entrance channel jetties. It extends from the present jetty head at RM 0.3 (Station 17+00) to the start of the entrance Turn at RM 0.9 (Station 45+50). This reach is influenced by tidal currents parallel to the channel and by waves. In addition, this reach experiences significant shoaling, particularly from RM 0.3 to RM 0.6. This shoaling has the effect of filling the bottom of the channel, effectively flattening the measured side slopes.

Similar to Reach 2, the jetty slopes are steep, on the order of 2:1. Adjacent to each jetty is a shoal that fluctuates across different surveys. At the North Jetty, rock is exposed at a depth of about -45 ft MLLW. The shipwreck *Rossell* is located within this reach from Station 18+00 to 19+00 and is discussed in this section.

4.2.4.1 Measured Side Slopes

Analysis of Reach 3 is limited by the amount of bathymetric data. Two high resolution data sets, the DEA surface and the 2014 NWP LiDAR bathymetry are available throughout the reach. The latter data set was selected because it represents the stable, non-shoaled slopes. Four cross sections were extracted at the locations shown in Figure 4-14. The measured cross sections at these locations are shown in

Table 4-4. The side slopes range from 13.6:1 to 35:1, with a median of 22:1, on the left side of the channel. The slopes range from 11.4:1 to 35:1, with a median of 15:1, on the right side of the channel.

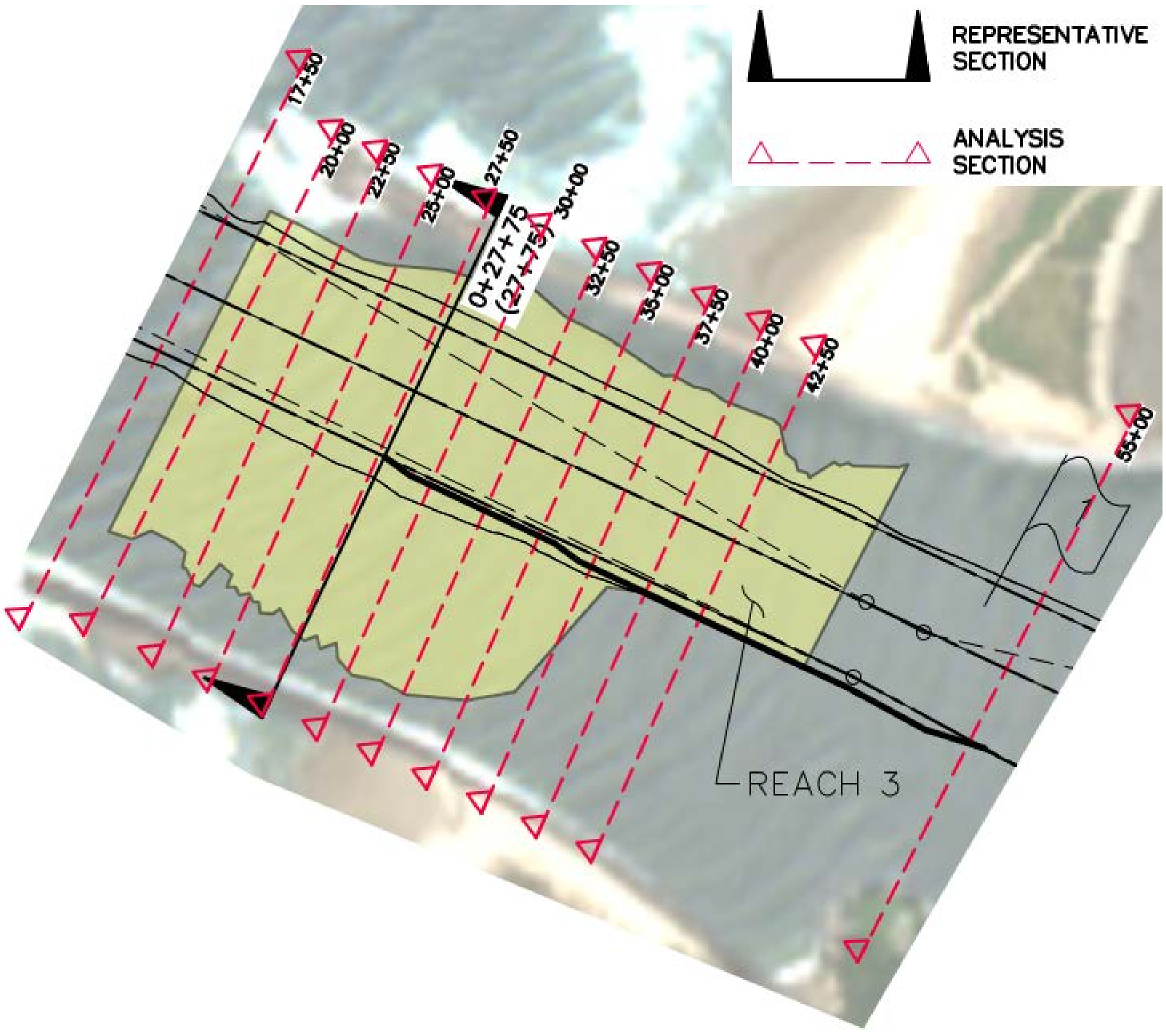


Figure 4-14
Cross Section Locations in Reach 3

Table 4-4
Side Slope Measurements in Reach 3

Station	Left Slope	Right Slope
17+50	26.1	13.1
20+00	22.0	15.6
22+50	27.8	15.7
25+00	31.1	11.4
27+50	35.0	15.1
30+00	30.1	14.1
32+50	22.1	35.0
35+00	13.6	24.9
37+50	15.7	29.6
40+00	13.8	N/A - Rock
42+50	N/A - Rock	N/A - Rock
Median	**22.1**	**15.4**

The representative cross section illustrated in Figure 4-15 shows the tall, steep jetties, and the milder slopes in sediment adjacent to the structures; this cross section is shown at a 10x vertical exaggeration. To the left of the channel, sediment slopes from the South Jetty at -20 ft MLLW to the channel at a slope of 22:1. On the right side of the channel, the sediment slopes from the North Jetty at -45 ft MLLW to the channel at a slope of 15:1. In Figure 4-15 infill is present within the channel and therefore the 15:1 side slope on the right side of the channel is not apparent.

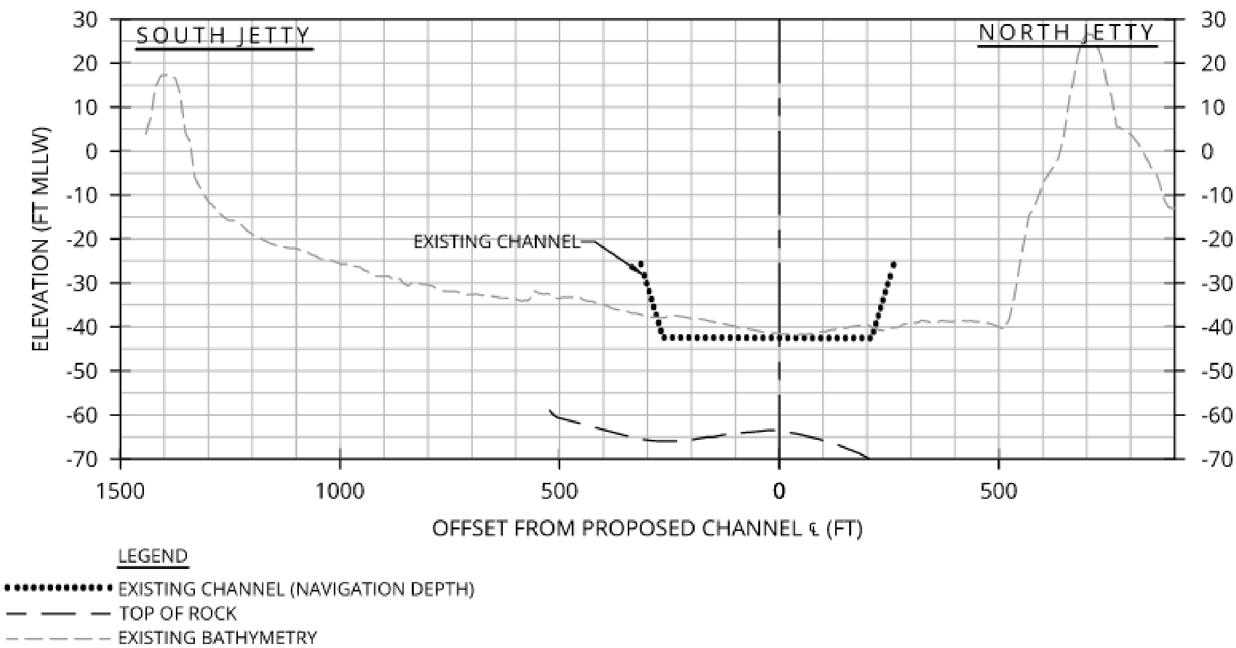


Figure 4-15
Reach 3 Representative Cross Section, Existing Condition (Station 27+75)

4.2.4.2 Substrate Conditions

Some rock is present within this reach. The bedrock intersects the left slope of the navigation channel at Station 27+50, where it is present at an elevation between a depth of -40 to -45 ft MLLW; equilibration is expected above the rockline. Upstream of Station 40+00, the rock is present at the sea floor, and no equilibration is expected. Bedrock intersects the right slope upstream of Station 35+00 and is located at the sea floor at Station 42+50. The rock can be seen in detail in Sub-appendix 13 (*Cross Sections)* which includes cross-sections showing the top of rock and the existing and PA channels at every 100 ft from Station 0+16+00 through Station 0+48+00.

The presence of rock will limit side slope equilibration and will provide some resistance to flow failure on the left side of the channel.

4.2.4.3 Long-Term Morphological Behavior

This reach of the entrance channel is subject to rapid shoaling by littoral sediments. Maintenance dredging occurs annually in this reach; after this dredging, the channel shoals again, with the shoaled material providing extra protection to the base of the North Jetty. This area is then dredged again during the subsequent cycle of annual maintenance. Figure 4-16 below is reproduced from USACE dredging surveys (Figure 3-7) and represents typical shoaling over the course of 1 year. As it shows, shoaling occurs over the entire width of the channel, and the AMD has been designed appropriately to maintain a navigable channel throughout the year. There is no noticeable trend of material sloughing from the top of the slope to the bottom of the slope. Under the PA, sediment transport modeling indicates that shoaling is expected to continue over the entire cross-section. Figure 4-17 shows sediment transport modeling results under the PA, starting from the construction side slopes. As this figure shows, shoaling continues over the entire channel, and over the side slopes (with the exception of RM 0.5 to 0.7). Shoaling occurs along the North Jetty, with a small erosional area at the jetty head where the rock apron is proposed. Therefore, net loss of material from the top of the slopes is not expected.

This reach expects no side slope equilibration for the reasons above. However, project effects and O&M volumes are based future equilibrium side slopes of 22:1 on the left side of the channel and 15:1 on the right side of the channel.

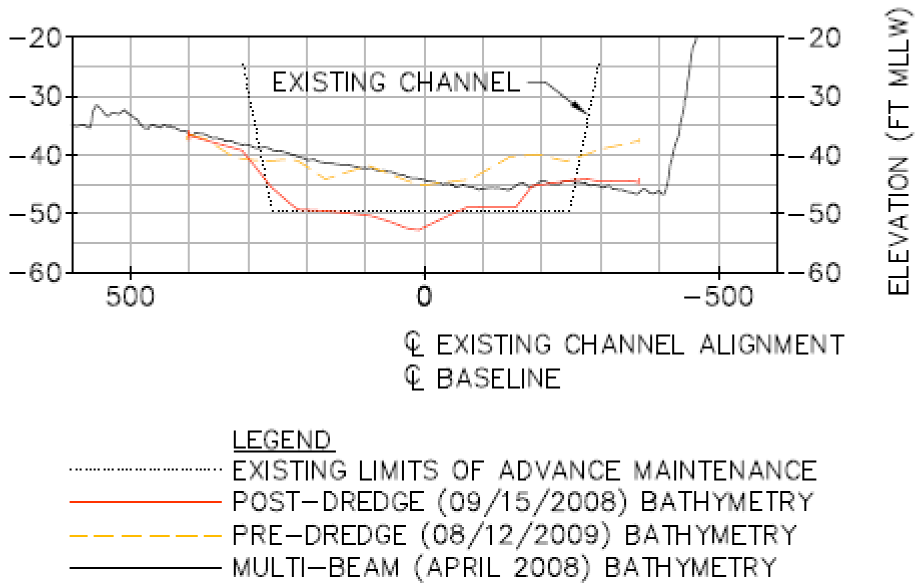


Figure 4-16
Shoaling at Station 22+50 from September 2008 to August 2009

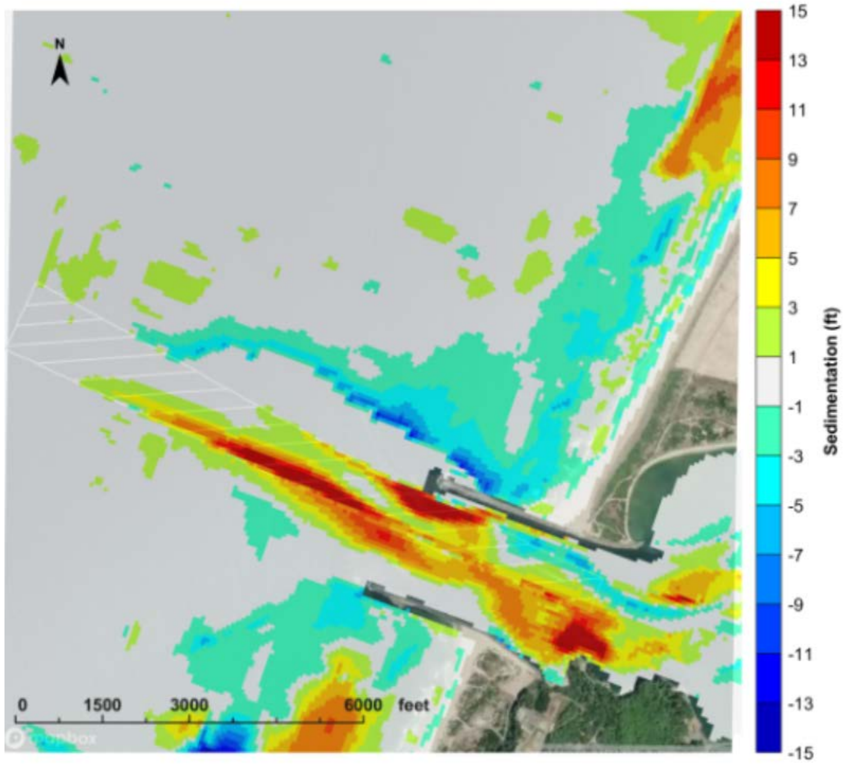


Figure 4-17
Modeled shoaling in the Entrance Channel under the PA

The presence of shoaling in the measured side slope angle is important to note because of how side slope equilibration is defined in Section 3.2.1 as, "slope angles that are likely to develop in the channel as a result of capital dredging." In cases like this, where the measured angles are based on shoaling, the analysis is no longer considering the result of capital dredging. The ultimate goal is to estimate a side slope angle that emerges from the channel toe and daylights along the adjacent bathymetry. However, the measured side slopes of this heavily shoaled reach originate 5-10 ft above the channel boundary and may not actually emerge from the dredged channel toe. The median side slopes form the basis for analysis of effects to the North Jetty.

4.2.4.4 Reach 3 – Conclusion

The future equilibrium side slopes measured from the toe of the dredged channel are 22:1 and 15:1 on the left and right sides of the channel, respectively. Under implementation of the channel modification, side slope equilibration after initial dredging is not expected – material accumulates on the top of the slopes, as opposed to slough from the top of the slopes (Figure 4-17). However, the effects of channel deepening – including side slope equilibration volumes and effects on the jetties, assume a 22:1 and 15:1 slope on the left and right sides of the channel, respectively. A representative cross section of the equilibrium side slopes under the PA shown at a 10x vertical exaggeration can be seen in Figure 4-18; true scale representations of the left and right slopes are depicted in Figure 4-19 and Figure 4-20, respectively. Sub-appendix 13 (*Cross Sections)* includes cross-sections showing the PA and median equilibrium side slopes at every 100 ft from Station 0+16+00 through Station 0+48+00.

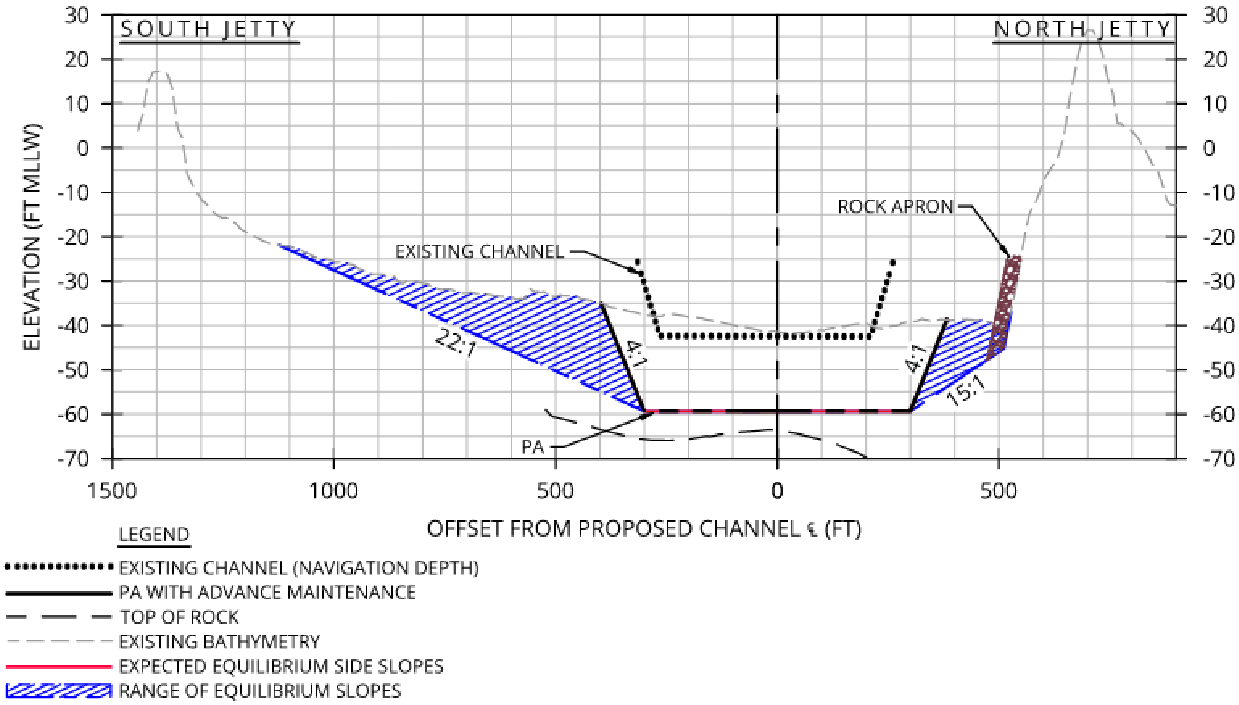


Figure 4-18
Reach 3 Representative Cross Section, PA Condition (Station 27+75)

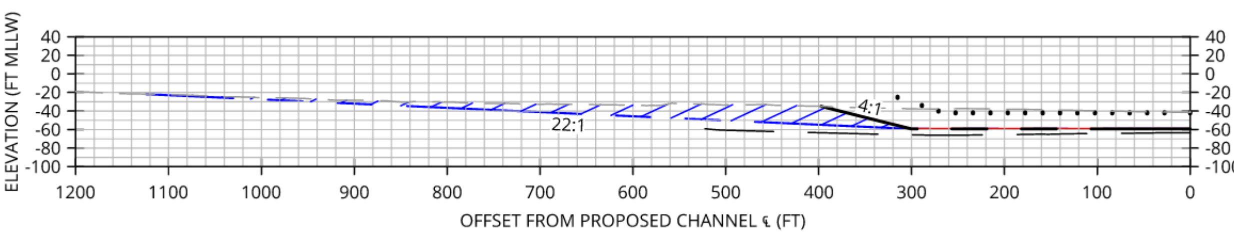


Figure 4-19
Future Equilibrium Side Slope in Reach 3, Left Slope

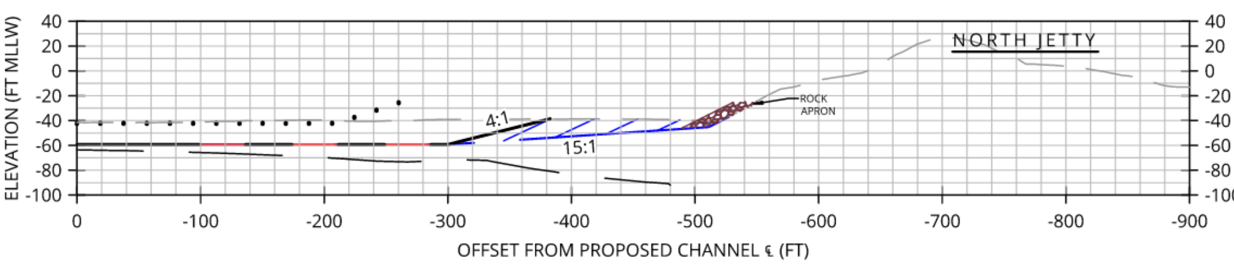


Figure 4-20
Future Equilibrium Side Slope in Reach 3, Right Slope

In this reach, the slope analysis of the future equilibrium shows the equilibrated side slope may undercut portions of the North Jetty. This effect can be mitigated by the construction of a rock apron at the toe of the structure. Physical effects of side slope equilibration on the North Jetty are discussed in detail in Section 7.3.

Cross-sections showing the *Rossell* are presented in Sub-appendix 13 (*Cross Sections),* Stations -0+18+00 and -0+19+00. As these sections show, the wreck is within the potential range of future equilibrium side slopes. Physical effects of future side slope equilibration on the *Rossell* are discussed in detail in Section 7.6.

4.2.5 Reach 4

Reach 4 encompasses the Entrance Turn, from RM 0.9 (Station 47+50) to RM 2.0 (Station 105+00). The curved nature of this reach leads to a horizontal distribution of currents with higher velocities along the outer bend and lower velocities along the inner bend. The channel bottom along the left side of the channel (the outer bend) is naturally very deep, generally deeper than the existing navigation channel.

The North Jetty root lies along the inner bank of the Entrance Turn. The channel is generally 900 ft or further from the North Jetty root, except at the rail spur (Jetty Extension 1924) at the upstream portion of the reach. At the rail spur, the channel is naturally deep; the channel footprint within 500’ of the rail spur is naturally deeper than -50 ft MLLW.

4.2.5.1 Measured Side Slopes

High-quality cross-channel surveys conducted by USACE from 2008-2016 were used to measure the side slopes in this reach. The measurement was taken at six cross sections; these six were selected because they were surveyed every year and because they are all perpendicular to the channel alignment. The measured side slopes range from 8:1 to 18:1, with a median of 13:1, on the left side of the channel. The measured side slope on the right side of the channel (adjacent to the North Jetty root) range from 11:1 to 22:1, with a median slope of 18:1. These six cross sections were extracted at the locations shown in Figure 4-21. The measured cross sections at these locations are shown in Table 4-5.

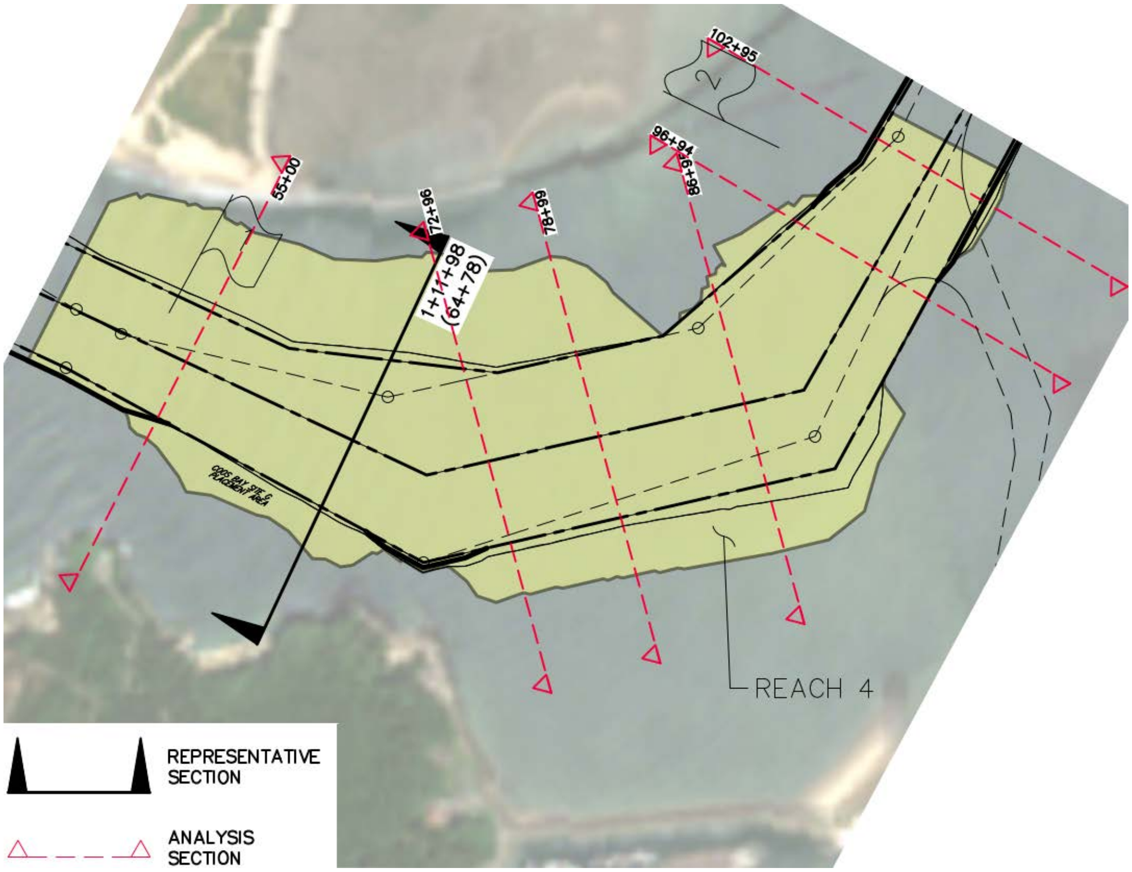


Figure 4-21
Cross Section Locations in Reach 4

Table 4-5
Side Slope Measurements in Reach 4

Station	Left Slope	Right Slope
55+00	18.4	15.5
72+96	N/A - Rock	18.2
78+99	N/A - Rock	19.01
86+95	N/A - Rock	21.9
96+94	N/A - Rock	10.7
102+95	8.1	N/A - Rock
Median	**13.3**	**18.2**

A representative cross section from this reach, taken at Station 64+78, is illustrated in Figure 4-22; this cross section is shown at a 10x vertical exaggeration. Reach 4 has a mild slope on the right side of the channel (i.e., the inner bank), and a rocky slope on the left side of the cross section, where sediment appears to have eroded away. The far right of the cross section shows the shoal adjacent to the North Jetty root. In this cross section, the North Jetty root would be located at x = 1,400 ft; the bathymetry survey did not extend this far.

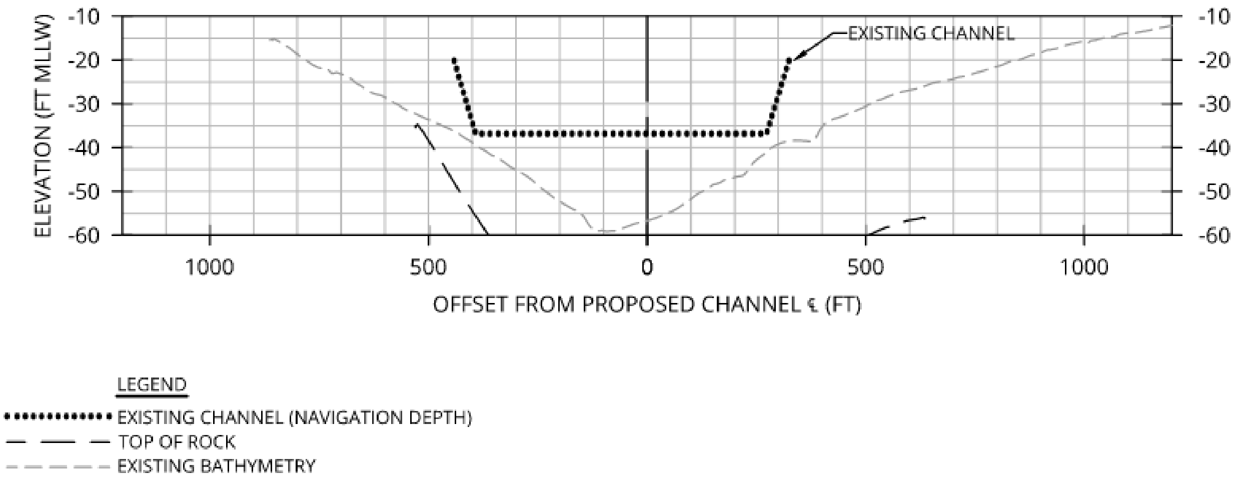


Figure 4-22
Reach 4 Representative Cross Section, Existing Condition (Station 64+78/RM 1.2)

4.2.5.2 Substrate Conditions

Some rock is present within this reach. The bedrock lines the left slope, likely due to erosion of the sediment once present on this slope. This bedrock stabilizes the left slope. Bedrock emerges on the right side of the channel at Station 100+00, which protects the North Jetty rail spur.

4.2.5.3 Long-Term Morphological Behavior

Studying a time series of high-quality cross sections in this reach shows that this reach has been stable over the past 10 years. Moreover, annual dredging patterns indicate that shoaling is limited within this reach. Over a longer time period, the channel has shifted significantly.

Long-term channel morphology has been assessed through digitization of the -18 ft MLLW from Navigation Charts dating back to 1916. The evolution of this line can be seen in Figure 4-23; as the figure shows, the inner bank of the channel has gradually been moving towards the North Jetty root. This movement coincides with deepening of the channel and widening towards the North Jetty. Channel deepenings at Section A-A’ are shown in Figure 4-24, which show the channel migrating towards the North Jetty with cross-sections from 1912, 1924, 1931 (after a channel modification authorized in 1930), 1952 (after a channel modification authorized in 1946), 1978 (after a modification authorized in 1969), and 1996 (with a channel modification the same year). On the outer bend of the turn, the -18 ft MLLW contour appears to shift between 1953 and 1967, likely in response to the 1952 dredging. On the inner bend, the -18 ft MLLW contour appears to move between 1937 and 1948, and again between 1953 and 1967, and gradually from 1974 to 1995. The second shift is likely a response to the 1952 project. Since the 1995 survey, the -18 ft MLLW contour has moved towards the channel, which is likely a result of the shoaling along this inner bend.

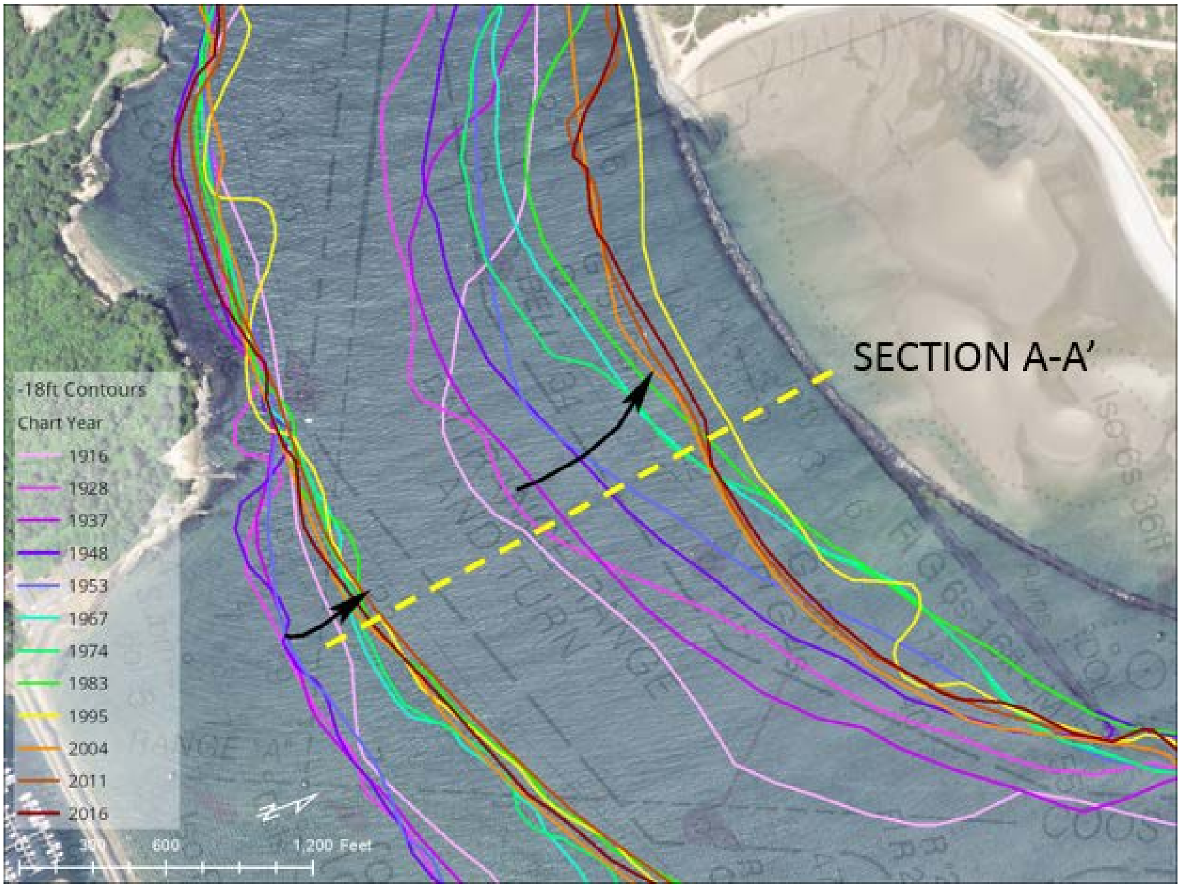


Figure 4-23
Migration of -18 ft MLLW Contour in the Entrance Turn

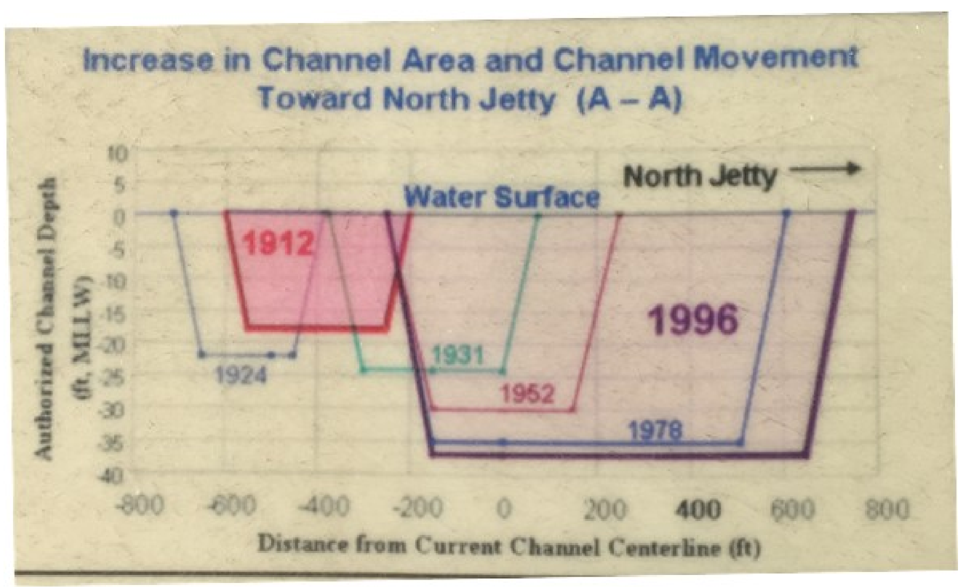


Figure 4-24
Channel Cross-section history at Station 1+27+00 (RM 1.5) (Section A-A')

During the 1996 channel deepening, it was proposed to shift the portion of the channel footprint adjacent to the North Spit channel towards the outer bend, away from the North Spit. The design (USACE 1994a) ultimately decided against this modification, based on the concern that, "the additional shoaling would cause adverse conditions in the turn" and that, "the shoal would continue to build inward over time and there would be no long-term maintenance savings." The -18 ft MLLW contour reached was closest to the North Jetty root in 1995; since that date, it has migrated inward, away from the North Spit. This confirms the assertion of the 1994 EIS, and also indicates that this area will continue to be stable after implementation of the PA.

Ultimately, the bathymetry in this reach has been controlled by channel modifications. The channel will react to these projects; after which it is stable morphologically. Therefore, the median measured side slopes of 13:1/18:1 are expected to result from equilibration on the left/right side of this reach.

One particular morphologic feature of interest in this reach is the scour hole adjacent to the rail spur. Figure 4-25 shows channel line survey data in the vicinity of the rail spur; the tip of the structure is marked with "Coos Bay Channel Light 7." In the vicinity of this feature, the channel bed has a depth of 52-63 ft below MLLW. In this area, dredging will not be required as the present bathymetry is sufficiently deep. Because dredging will not be necessary, side slope equilibration is not expected.

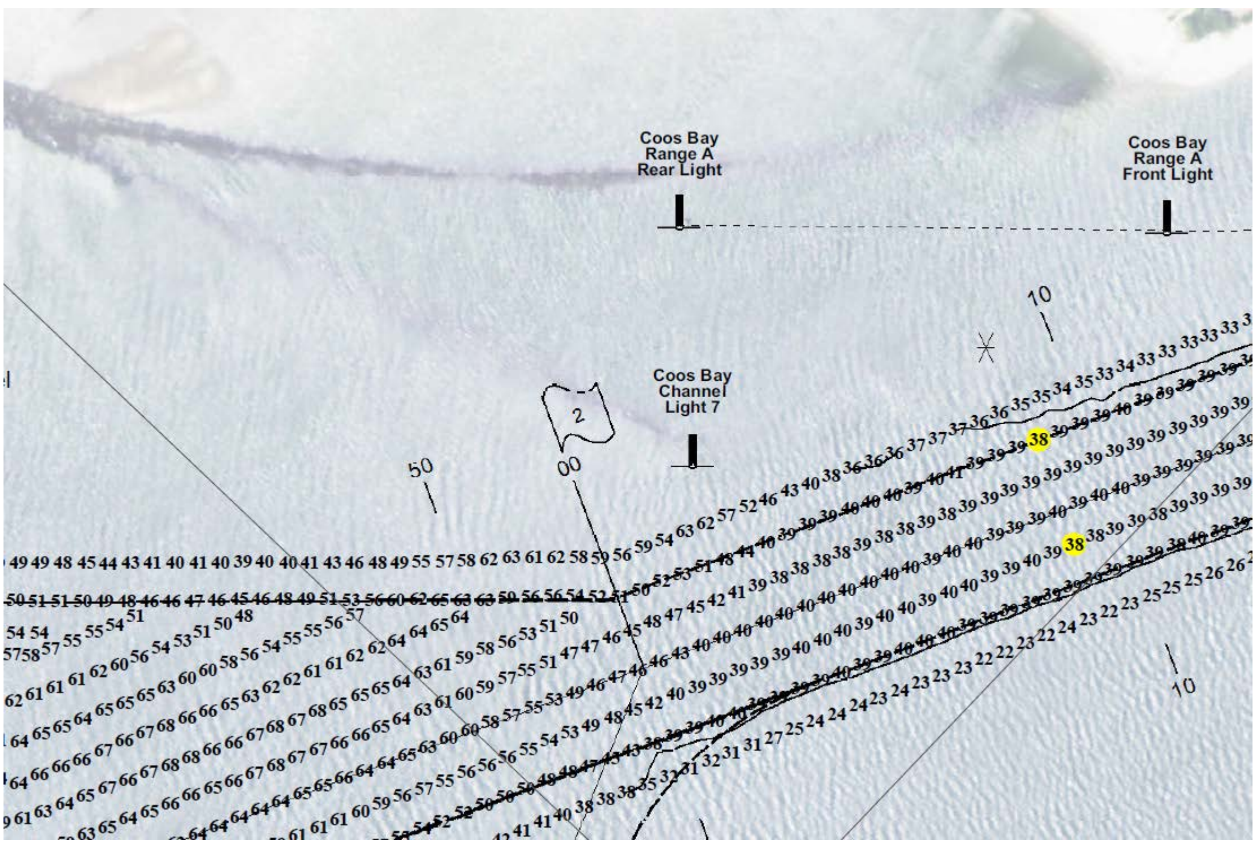


Figure 4-25
USACE Survey Data in the Vicinity of the Rail Spur (Coos Bay Channel Light 7)

4.2.5.4 Reach 4 – Conclusion

The future equilibrium side slopes measured from the toe of the dredged channel in Reach 4 are 13:1 and 18:1 on the left and right sides of the channel, respectively. Under implementation of the channel modification, the 18:1 slope on the right side of the channel is not expected to change. At some locations, steep slopes are found on the upper portion of the left side of the channel, where a compound slope develops. In these locations, the compound slope is expected to persist upon PA implementation, with the lower portion of the slope at 18:1 and the upper portion at its measured slope.

On the left side of the channel, the majority of the channel bottom is comprised of rock. Therefore, future side slope equilibration will be limited on much of the left side of the channel. Similarly, the -18 ft MLLW contour is not expected to shift. In the areas where equilibration may occur, a 13:1 slope is expected to occur over a relatively short distance. A representative cross section of the equilibrium side slopes under the PA shown at a 10x vertical exaggeration can be seen in Figure 4-26; true scale representations of the left and right slopes are depicted in Figure 4-27 and Figure 4-28, respectively. Cross sections in this reach at both exaggerated and true scale for the construction and median side slope angles can be seen in Stations 0+48+00 through 2+00+00 in Sub-appendix 13 (*Cross Sections)*.

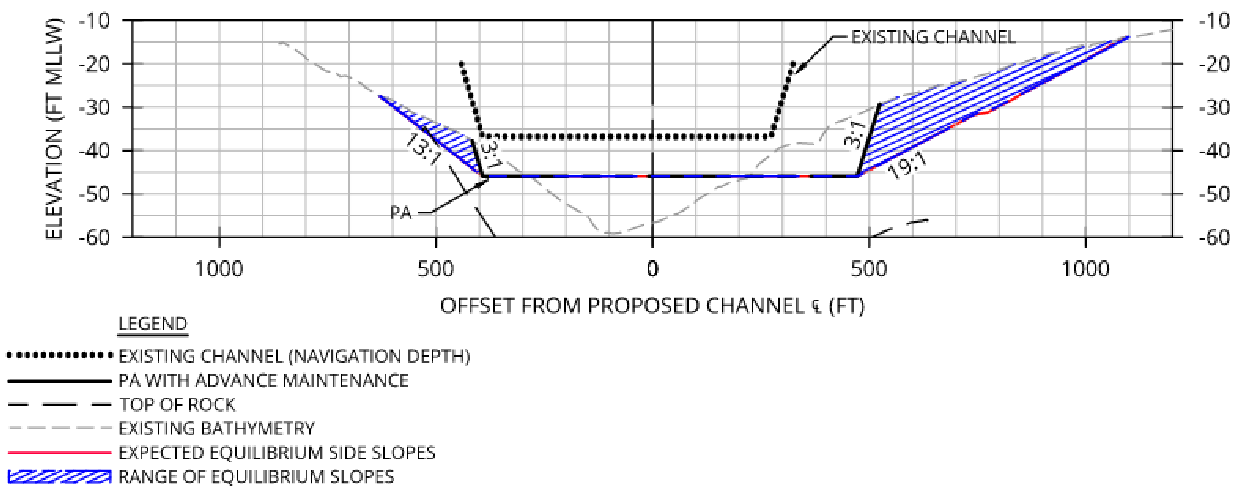


Figure 4-26
Reach 4 Representative Cross Section, PA Condition (Station 64+78)

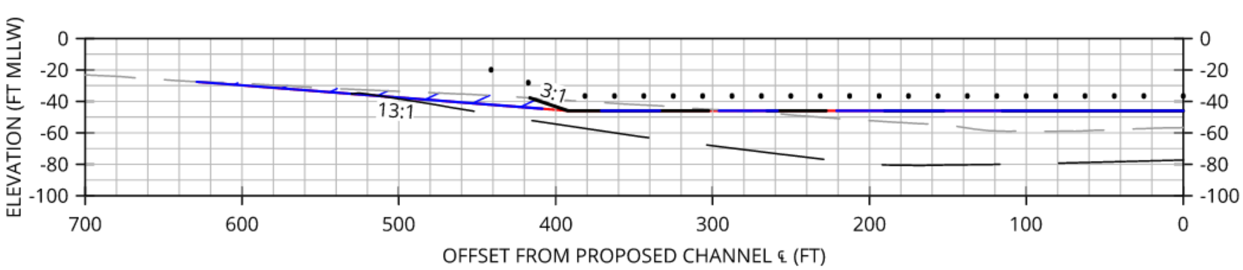


Figure 4-27
Future Equilibrium Side Slope in Reach 4, Left Slope

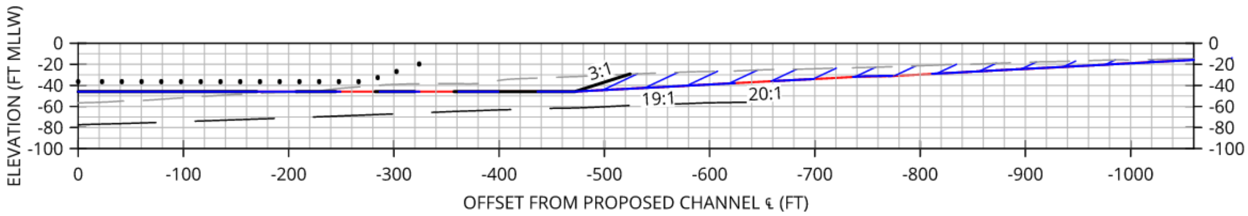


Figure 4-28
Future Equilibrium Side Slope in Reach 4, Right Slope

The North Jetty root is present in this reach. At the beginning of this reach, side slope equilibration has the potential to undercut a portion of the structure; a rock apron has been proposed to mitigate this potential undercutting, and is described in Section 7.2. Upstream of Station 56+00, the structure is sufficiently far from the channel that side slope equilibration is not expected to undercut the structure. One portion of the structure, the rail spur, does come close to the proposed channel. However, in the vicinity of this structure, the present bathymetry is sufficiently deep that dredging will not be necessary (Figure 4-25); in addition, the channel is underlain by shallow bedrock, which would limit any side slope equilibration that were to occur. Therefore, no effect to this structure is expected. This determination of no effects can be seen visually in Sub-appendix 13 (*Cross Sections)*, Stations 1+45+00 through 2+00+00.

After implementation of the PA, it is reasonable to assume that, “the shoal [will] continue to build inward over time” (USACE 1994a), which will effectively protect the base of the North Jetty.

4.2.6 Reach 5

Reach 5 encompasses the portion of the channel directly upstream of the Entrance Turn that is underlain by rock. It extends from RM 2.0 (Station 105+00) to RM 4.5 (Station 237+50). This portion of the channel will be dredged directly into rock, and therefore equilibration is not expected.

The EIS (USACE 1994a) and contract drawings (USACE 1996) for the 1996 channel deepening project indicate that the downstream portion of this reach has been dredged directly into rock. These conclusions are further supported by a comparison of the present bathymetry and the top of bedrock, which shows little to no sand overburden. The Empire Outfall is located at the upstream extent of this reach.

4.2.6.1 Measured Side Slopes

Cross sections were investigated throughout the reach; the locations of the cross sections can be seen in Figure 4-29. A typical cross section, measured at Station 168+89, can be seen in Figure 4-30; true scale representations of the left and right slopes are depicted in Figure 4-31 and Figure 4-32, respectively. At this cross-section, and each of the other cross sections, the top of the bedrock coincides with the channel bottom. Outside of the existing channel limits, the top of the bedrock coincides with the riverbed.

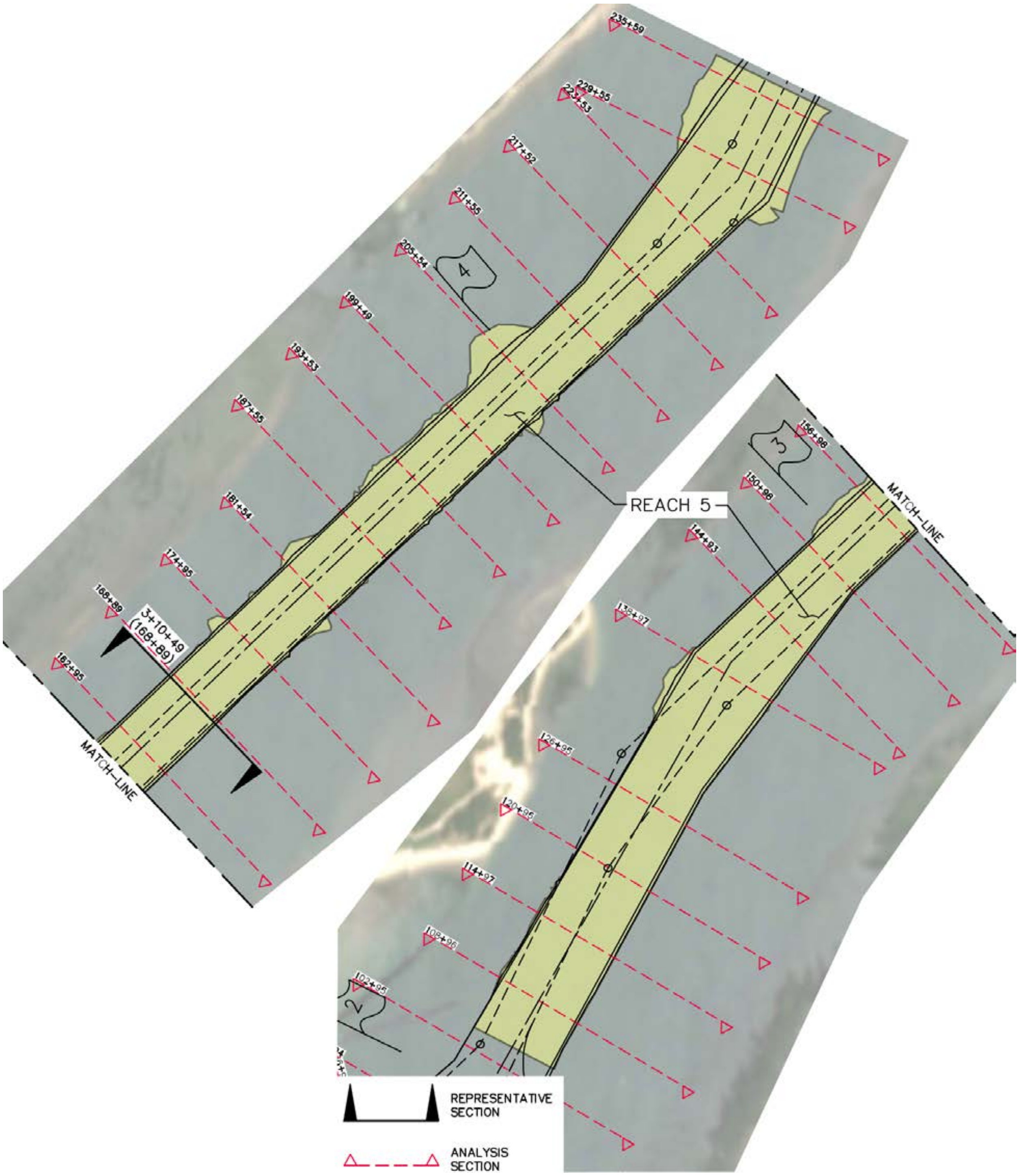


Figure 4-29
Cross Section Locations in Reach 5

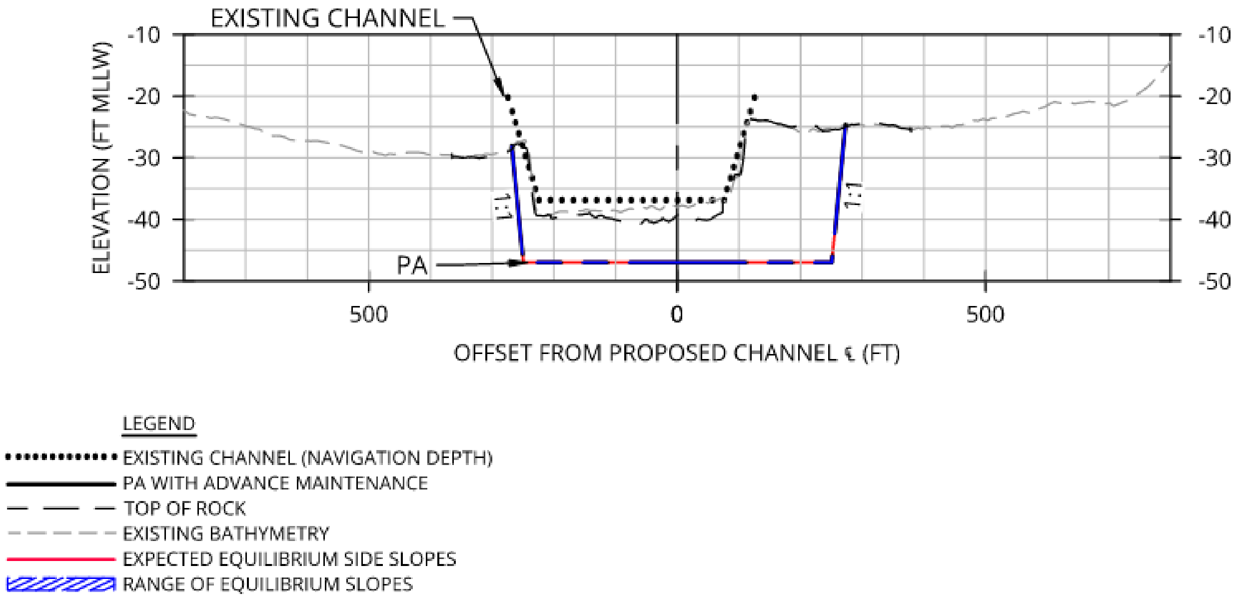


Figure 4-30
Reach 5 Representative Cross Section, PA Condition (Station 168+89/RM 3.2)

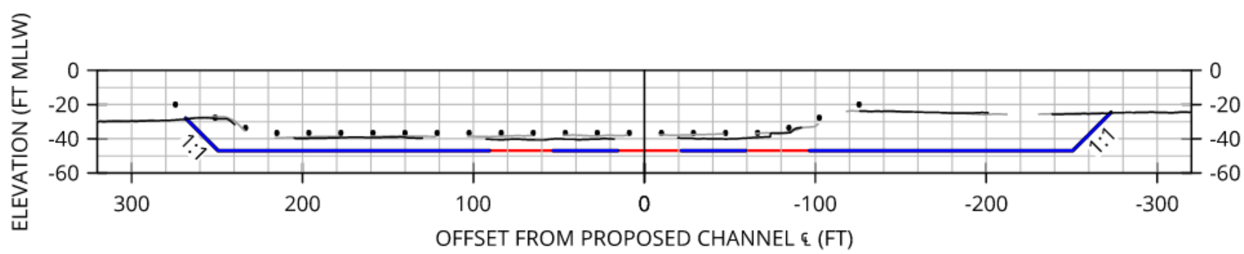


Figure 4-31
Future Equilibrium Side Slope in Reach 5, Left Slope

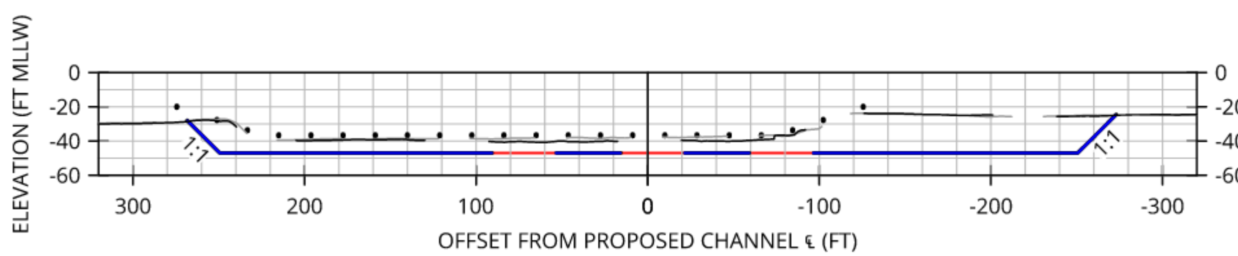


Figure 4-32
Future Equilibrium Side Slope in Reach 5, Right Slope

The maximum sand overburden thickness is 15 ft and occurs at the upstream extent of this reach. The majority of this reach has zero overburden.

4.2.6.2 Substrate Conditions

As shown in Figure 4-30, the existing channel, and the footprint of the proposed PA, is generally underlain by rock. Therefore, side slope equilibration is not expected in this reach.

4.2.6.3 Long-Term Morphological Behavior

The presence of bedrock limits the data analysis for channel morphology within this reach. In areas where limited sand overburden is present, the analysis is expected to follow the methodology and results for Reach 6, as explained in Section 4.2.7. Reach 6 provides a good representation of Reach 5 because both reaches are straight and contain bedrock (bedrock is present throughout Reach 5, and outcrops are present in Reach 6). In Reach 5, more bedrock is present and, therefore the sediment is even less susceptible to equilibration. Thus, using the side slope analysis from Reach 6 is likely conservative.

4.2.6.4 Reach 5 – Conclusion

This reach is generally underlain by rock. Therefore, future side slope equilibration will be limited. Where sand overburden is present, side slopes of 9:1 are expected. However, the equilibrium side slopes may be as flat as 11:1. Slopes will emerge from the top of the bedrock. Cross sections in this reach at both exaggerated and true scale for the construction and median side slope angles can be seen in Stations 2+00+16 through 4+33+75 in Sub-appendix 13 (*Cross Sections)*.

The Empire Outfall can be seen in Sections 4+33+25 through 4+33+75 of Sub-appendix 13 (*Cross Sections)*. The median equilibrium side slopes daylight 250 ft from the outfall. Therefore, no effects from equilibration are expected. Physical effects of side slope equilibration on outfalls are summarized in Section 7.10.

4.2.7 Reach 6

Reach 6 is located in the straight portion of the channel, where the bedrock deepens. This reach is located from RM 4.5 (Station 237+50) to RM 5.6 (Station 297+50). Tidal currents flow parallel to the channel orientation; these currents mobilize bedload sediment, which has a downward component due to gravity. This downward component of bed load may transport sediment from the top of a dredge cut to the bottom of a cut.

The present bathymetry generally coincides with the navigation channel prism; side slope equilibration is expected along the deeper slopes of the PA. Rock outcrops are present throughout this reach, which will limit the extent to which side slope equilibration may occur. The T-dock, an abandoned fish ladder, an abandoned outfall, the Cape Arago Dock (Sause Bros.), various docks at Hollering Place, and the Empire outfall are present in this reach. The abandoned infrastructure that was a part of the abandoned aquaculture facility will be demolished as a part of the new container terminal. The Port-owned T-Dock will also be encroached upon by the proposed turning basin and may be demolished. A container terminal and wharf is proposed on the right bank (northwest). The slopes under the wharf will be dredged at 2:1 in rock and loose sediments and immediately armored. Therefore, no side slope equilibration is shown along the wharf.

4.2.7.1 Measured Side Slopes

High-quality cross-channel surveys conducted by USACE from 2008-2016 were used to measure the side slopes at 10 cross sections within this reach. Because the hydrodynamic and sediment conditions are similar on both sides of the reach, all of the side slope measurements were considered to be one dataset, instead of differentiating the two sides. The measured side slopes range from 6.2:1 to 25:1, with the median side slope 10.2:1 (approximately 11:1, with the same slope angle on each side of the channel). The cross-section locations are shown in Figure 4-33.

The measured cross sections at these locations are shown in Table 4-6. As this table shows, the extent of rock varies throughout the reach, without any clear patterns or gradients.

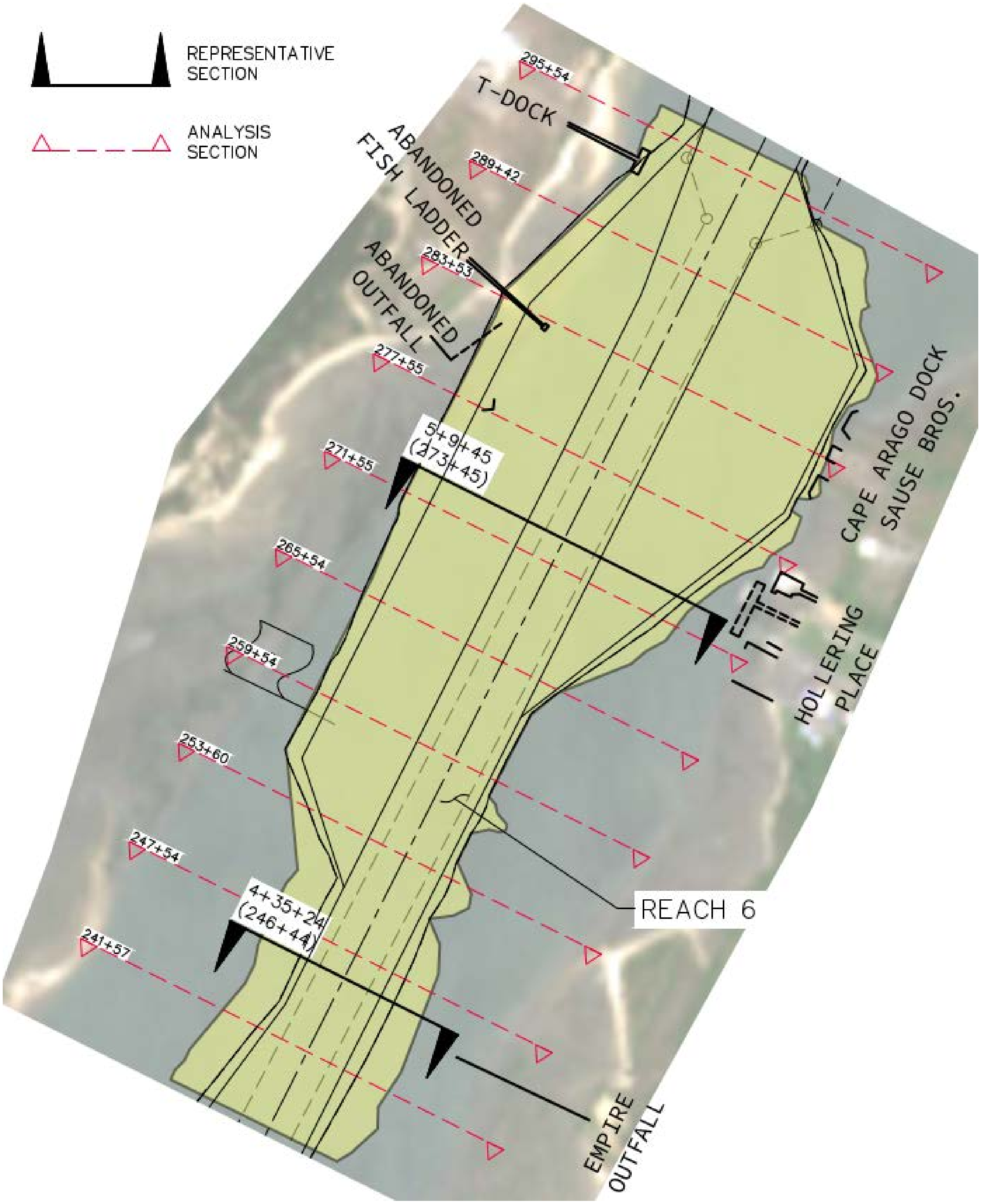


Figure 4-33
Cross Section Locations in Reach 6

Table 4-6
Side Slope Measurements in Reach 6

Station	Left Slope	Right Slope
241+57	9.5	16
247+54	8.1	N/A - Rock
253+60	N/A - Rock	N/A - Rock
259+54	10.2	10.1
265+54	N/A - Rock	9.6
271+55	N/A - Rock	12
277+55	15.1	15.1
283+53	25.5	19.2
289+42	N/A - Rock	6.2
295+54	N/A - Rock	8.6
Median	**10.2**	**11.1**

A representative cross section from this reach, taken at Station 273+45, is illustrated in Figure 4-34; this cross section is shown at a 10x vertical exaggeration. This cross section shows similar side slopes on each side of the existing channel, measuring approximately 11:1. This cross section also shows the relatively shallow bedrock, a portion of which will be dredged.

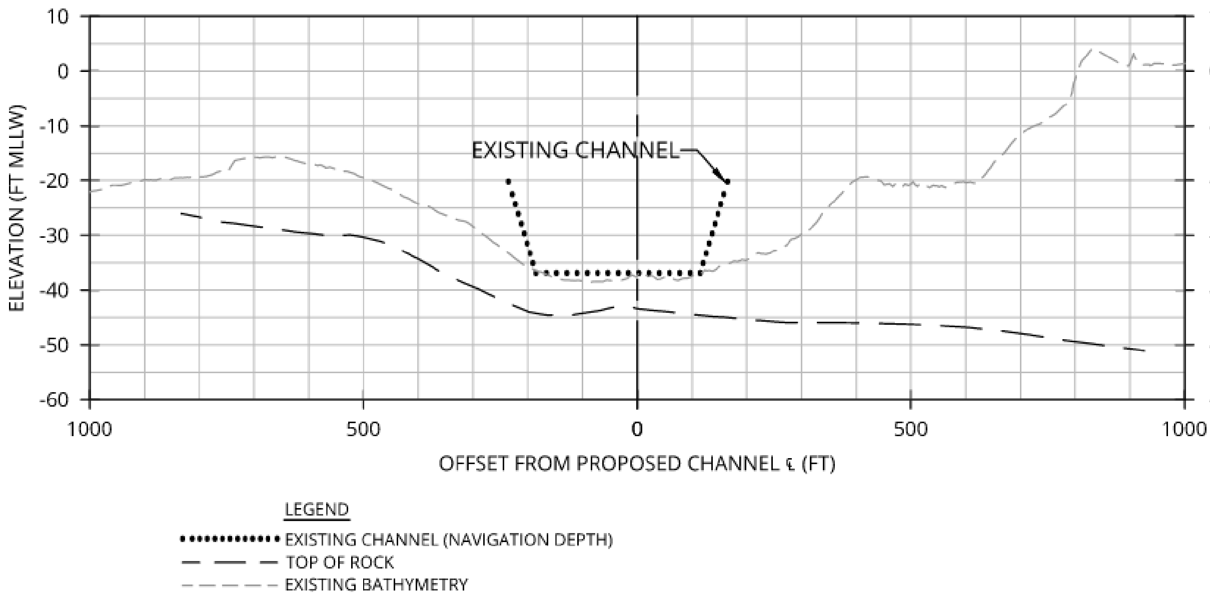


Figure 4-34
Reach 6 Representative Cross Section, Existing Condition (Station 273+45/RM 5.2)

4.2.7.2 Substrate Conditions

The elevation of the bedrock varies throughout this reach, although it is generally within 10 ft of the existing channel bottom. As seen in Table 4-6, there is not a clear pattern of rock; it raises and lowers throughout this reach. A rock contour map has been added to Sub-appendix 2 (*Geophysical Report*) that provides a visual description of the top of rock throughout the channel. The presence of rock only affects equilibration to the extent that it defines the point from which equilibration begins.

4.2.7.3 Long-Term Morphological Behavior

Currents flow parallel to the channel alignment, generating bed load transport. When a sediment particle resting on the side slope is set into motion by currents, it will move very nearly in the direction of flow. However, due to gravity, the path of the particle will deviate slightly in a downward direction (see the bottom of Figure 3-3). By this mechanism, sediment is transported from the top of the slope to the deeper part of the channel, yielding flatter side slopes. This process is quantified by the method presented by the Fredsoe (1978) equations of gravity infill, as described in Section 3.2.4.2.

The Fredsoe model is configured for the existing channel and measured side slopes in this reach. Model calibration consisted of adjusting soil parameters such that the model is able to reproduce measured side slopes. The resultant sediment used for the analysis was selected to be equivalent to one of the borings within this reach and the adjacent reaches; for calibration, the sediment properties at the top of the boring were considered. The calibrated model yields side slopes are flatter than measured but appear reasonable when overlaid upon present bathymetry; calibration results for Reach 6 can be seen in Figure 4-35. For model calibration, an equilibration timescale of 8 years was considered, as described in Section 3.2.4.3, and results are displayed at two-year intervals. This is consistent with the timescale of equilibration stated in Section 3.2.4.3.

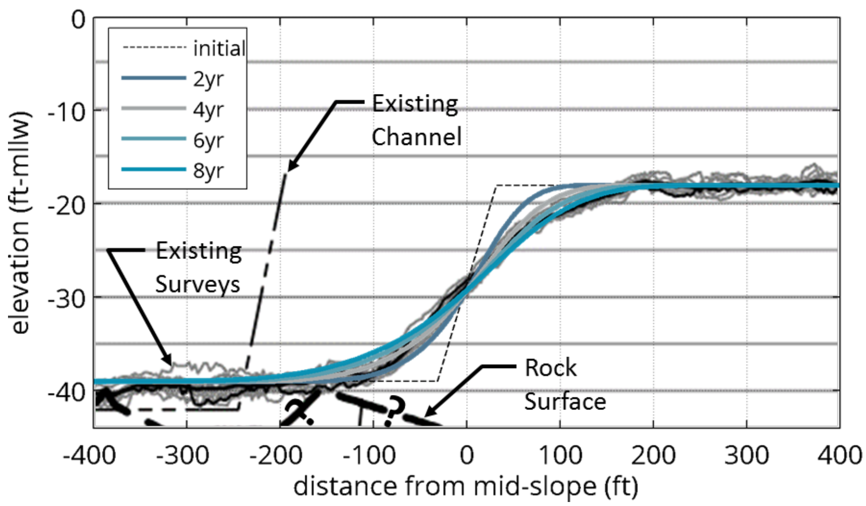


Figure 4-35
Calibration to Existing Conditions at Station 265+54 (RM 5.0)

The model is conservative in this reach, with the 8-year slope angle being flatter than the median slope, although still within the range of measured slopes. It is possible that high points in the rock surface help support steeper slopes than modeled.

The calibrated model is then applied to the dredge prism of the modified channel. For the deeper channel, the sediment parameters are adjusted to reflect the deeper sediment (i.e., the characteristics describing sediment at the bottom of the boring was used) (Sub-appendix 5). The deeper sediment within the borings were denser (Boring B-19 for example shows sand transitioning from “medium dense” to “dense” at -45 ft MLLW); therefore, changing the sediment properties for the with-project runs provides a closer reflection of the future conditions. The modeled slope height spans from the channel bottom to the top of the adjacent banks in this scenario. The measured side slope was then applied to bottom of the slope (i.e., the bottom of the channel or the location where bedrock intersects the channel).

When taking into consideration site and geotechnical variability, the modeled side slopes varied from 8:1 to 12.5:1. Given the conservative nature of the calibrated model, the future equilibrium side slope used to calculate the capital dredge volume for Reach 6 is estimated at 9:1 (Figure 4-36).

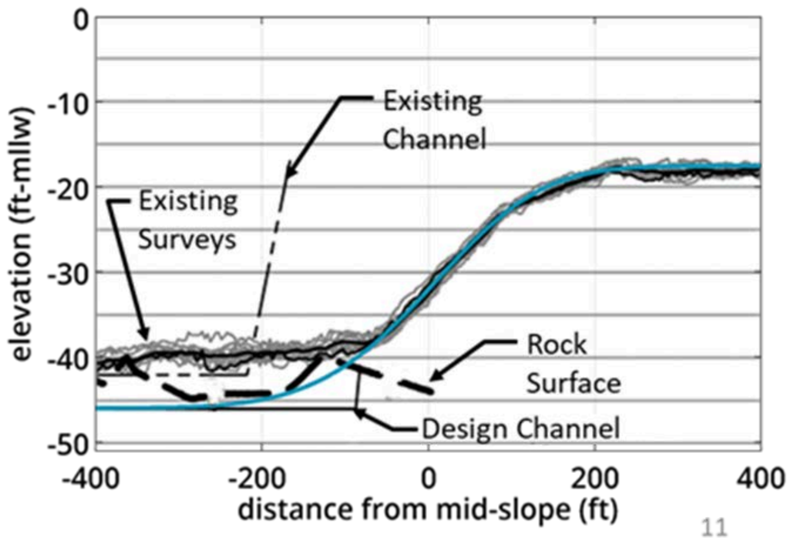


Figure 4-36
Estimated Side Slopes for the PA at Station 265+54 (output after 8 years shown)

4.2.7.4 Reach 6 – Conclusion

The future equilibrium side slope based on median measured side slopes in Reach 6 are 11:1 on both sides of the channel. Under implementation of the channel modification, 9:1 side slopes were calculated based on long-term morphological methods described in Section 3.2. On the left side of the channel, many portions of the channel are underlain by rock outcrops. Therefore, side slope equilibration will be limited on much of the left side of the channel; on this side of the channel, it will only occur above the rock. A representative cross section of the equilibrium side slopes under the PA shown at a 10x vertical exaggeration can be seen in Figure 4-37. True-scale depictions of the left and right slopes at 246+44 can be seen in Figure 4-38 and Figure 4-39, respectively. Figure 4-40 shows a representative cross-section through the berth and turning basin. Cross sections in this reach at both exaggerated and true scale for the construction and median side slope angles can be seen in Stations 4+42+24 through 5+31+69 in Sub-appendix 13 (*Cross Sections)*.

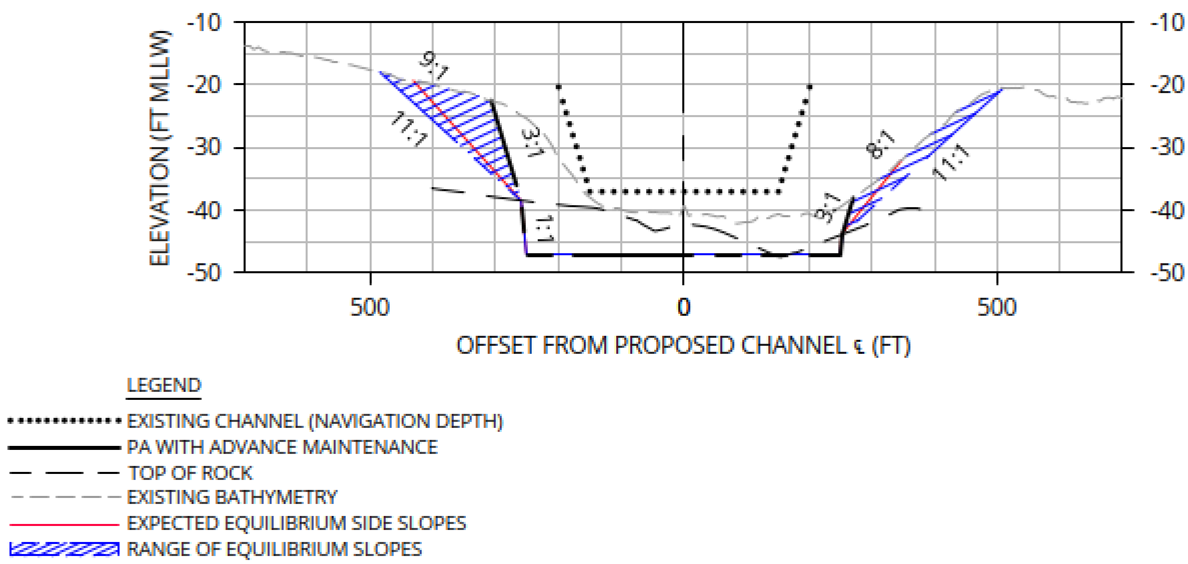


Figure 4-37
Reach 6 Representative Cross Section, PA Condition (Station 246+44)

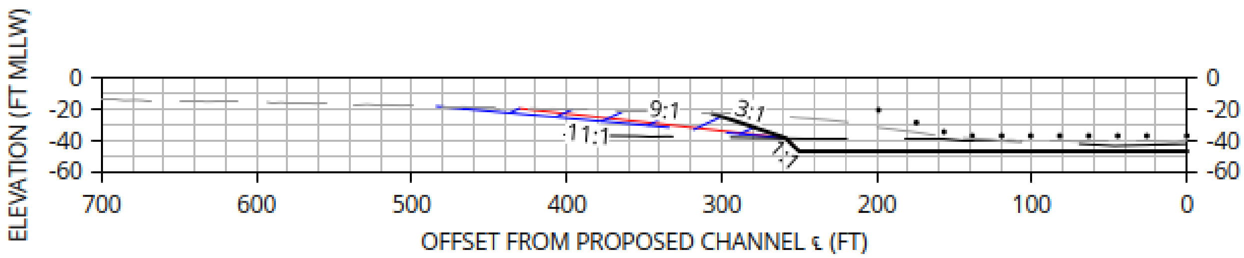


Figure 4-38
Future Equilibrium Side Slope in Reach 6, Left Slope

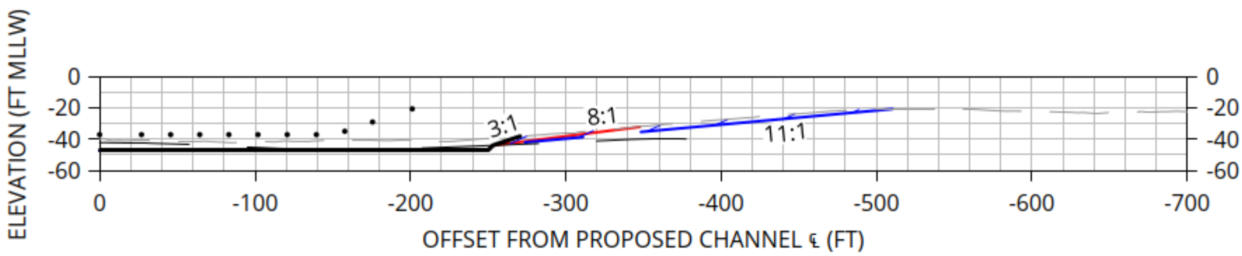


Figure 4-39
Future Equilibrium Side Slope in Reach 6, Right Slope

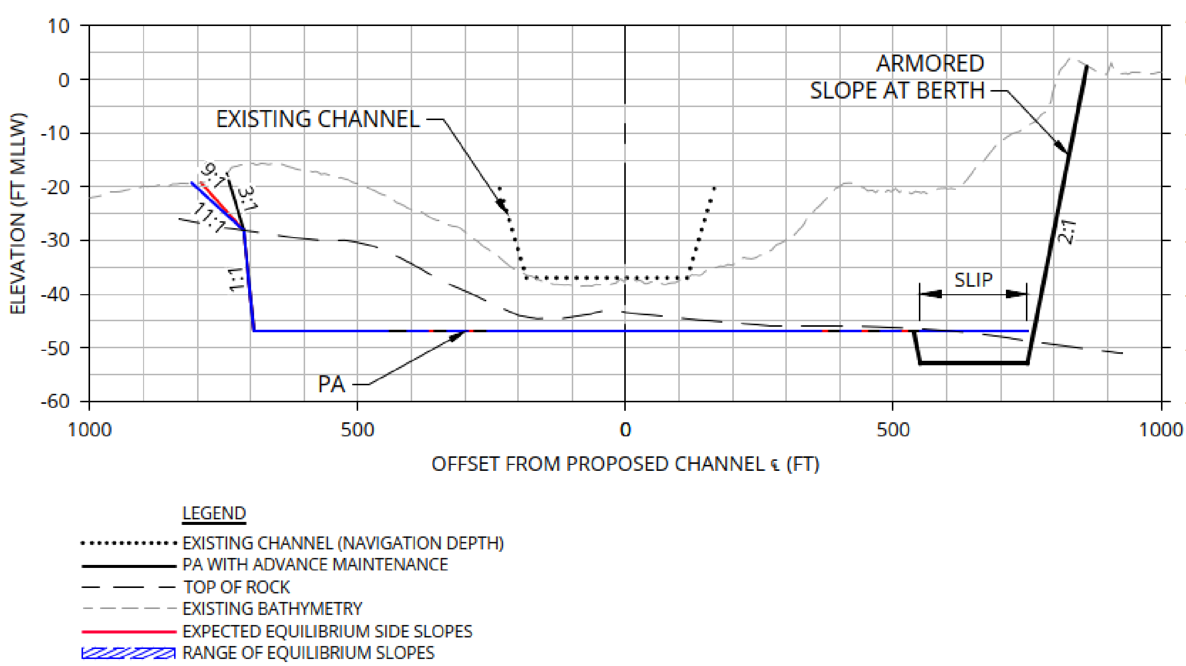


Figure 4-40
Reach 6 Representative Cross Section, PA Condition (Station 273+45)

Cross sections showing the median equilibrium side slopes adjacent to the infrastructure within this reach can be seen in Sub-appendix 13 (*Cross Sections)*, Stations 4+33+84 (FIG 132) through 5+30+78 (FIG-153). Physical effects of side slope equilibration on adjacent infrastructure is discussed in detail in Section 7.

4.2.8 Reach 7

Reach 7 is located directly upstream of Reach 6, from RM 5.6 (Station 297+50) to RM 6.4 (Station 340+00). Similar to Reach 6, this portion of the channel is straight and dominated by tidal currents that flow parallel to the channel alignment. Therefore, the same sediment transport mechanism is applicable to this reach. Reach 7 differs from Reach 6 in that it is wider (and, therefore, current velocities are less), contains different sediment, and contains deeper bedrock. The bedrock is at an elevation that is unlikely to impact side slope measurements, but is at a shallower depth than the bottom of the PA, and will limit the depth over which side slope equilibration occurs under the PA.

One of the pile dikes, CB-6.4, is located within this reach (although named for RM 6.4, it is actually located at RM 6.3); the pile dikes are described in Section 4.2.9.

4.2.8.1 Measured Side Slopes

High-quality cross-channel surveys conducted by USACE from 2008-2016 were used to measure the side slopes at seven cross sections within this reach. The measured side slopes range from 11.2:1 to 24.8:1 on the left side of the slope, and from 3.9:1 to flat on the right side of the slope. The median side slopes are 15:1 and 32:1 on the left and right side of the channel, respectively. Figure 4-41 shows the cross-section locations. The measured cross sections at these locations are shown in Table 4-7.

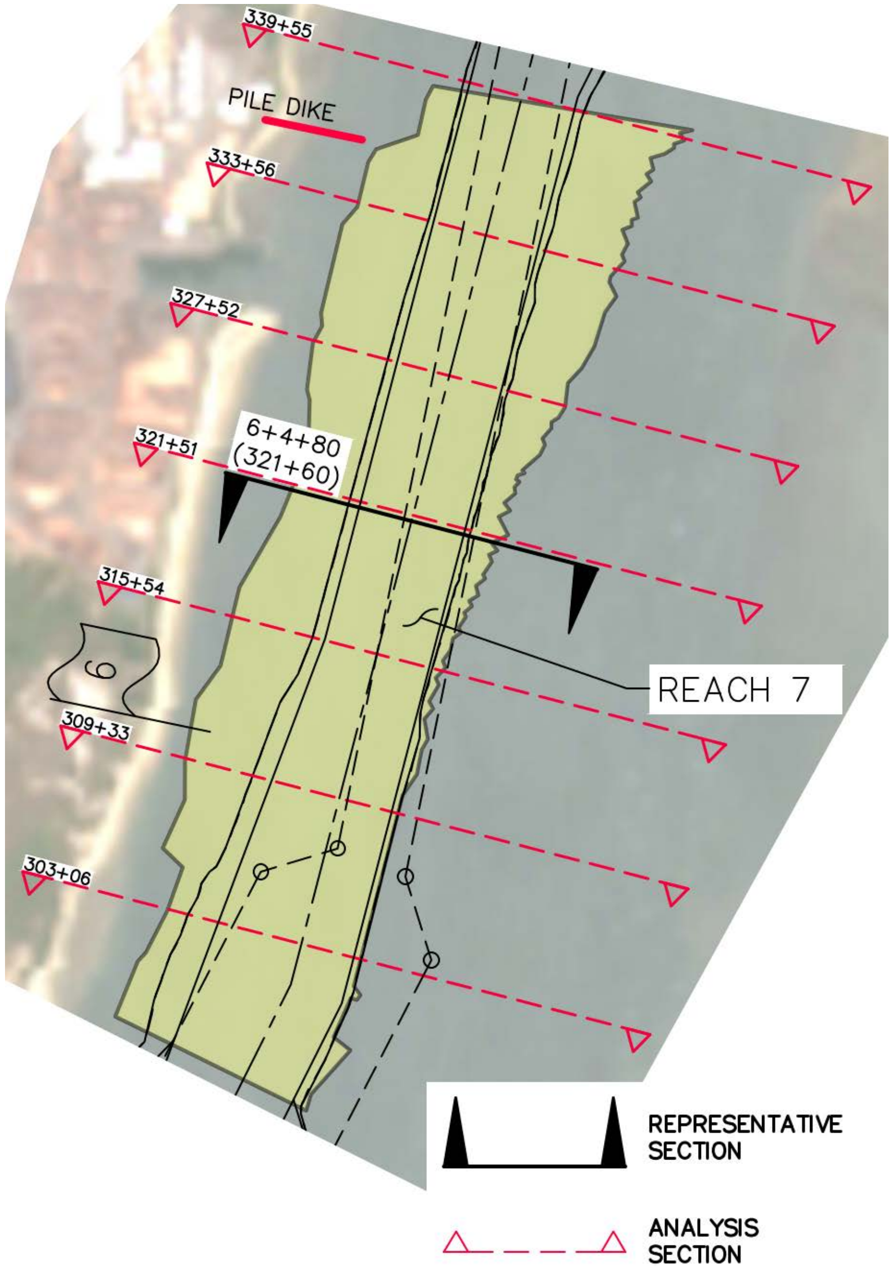


Figure 4-41
Cross Section Locations in Reach 7

Table 4-7
Side Slope Measurements in Reach 7

Station	Left Slope	Right Slope
303+06	11.2	3.9
309+33	11.8	29.5
315+54	15.5	28.4
321+51	15	32.1
327+52	18.2	30.3
333+56	23.9	47.4
339+55	24.8	Flat
Median	**15**	**32**

Investigation of Table 4-7 shows that the right slope at Station 303+06 appears much steeper than the others. This steep slope is located just upstream of the T-dock; geotechnical investigations associated with the construction of this structure revealed a relatively shallow depth to rock (as shallow as -27 ft MLLW) in this area (OIPCB 1980). This rock may influence the steepness of the measured slope.

The left slopes in Table 4-7 become flatter upstream, as the slopes in this reach transition from the 11:1 slopes of Reach 6 to the 20:1 slopes near Jarvis Turn (Reach 8). Using the median side slope angle in this reach can be used to calculate the volume, similar to the average area method used in engineering. As Figure 4-41 shows, the left slope of Reach 7 is not adjacent to any infrastructure. Therefore, the equilibrium side slope is only used to calculate volumes and a median value is acceptable.

A representative cross section from this reach, taken at Station 321+60 (RM 6.1), is illustrated in Figure 4-42; this cross section is shown at a 10x vertical exaggeration. This cross section shows the left side slope at 15:1, and a compound slope on the right side of the channel. The measured side slope values (Table 4-7) on the right side of the channel reflect the lower, flatter slope. The steeper portion of the compound slope is located approximately 500 ft away from the existing channel. The compound slope has been modeled as the future condition. This is consistent with the approach introduced in Section 3.2.2 that accepts the measured slopes as stable.

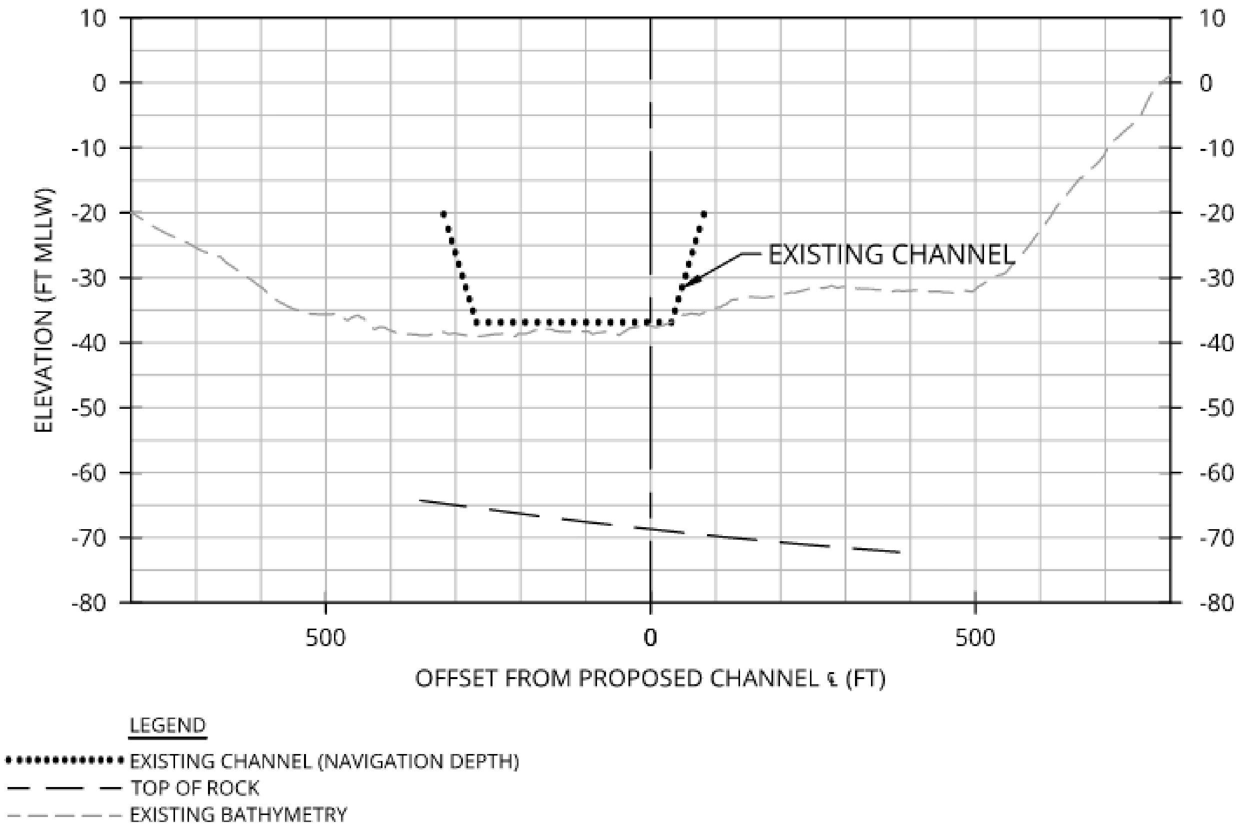


Figure 4-42
Reach 7 Representative Cross Section, Existing Condition
(Station 321+60/RM 6.1)

4.2.8.2 Substrate Conditions

The elevation of the bedrock varies throughout this reach, although it is generally within 10-20 ft of the existing channel bottom. Sediment is medium dense to dense sand. Equilibration at depths shallower than the rock depth are expected.

4.2.8.3 Long-Term Morphological Behavior

The morphology in Reach 7 is similar to the morphology of Reach 6. The Fredsoe model was calibrated to the measured side slopes using local current velocities and sediment conditions. The reader is referred to Section 4.2.7.3 for a description of model calibration and setup.

The calibrated model performs well. Using sediment properties from a boring located within this reach, the model reproduces side slopes within this reach. Calibration results for Reach 7, along with measured side slopes, can be seen in Figure 4-43. For model calibration, an equilibration timescale of 8 years was considered, and results are displayed at two-year intervals. This is consistent with the timescale of equilibration stated in Section 3.2.4.2.

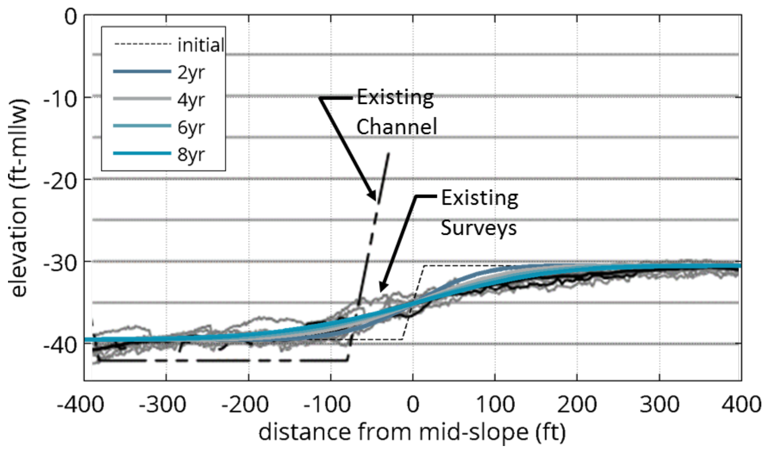


Figure 4-43
Calibration to Existing Conditions at Station 321+51

The calibrated model is then applied to the dredge prism of the modified channel. For the deeper channel, the sediment parameters are adjusted to reflect the deeper sediment (i.e., the characteristics describing sediment at the bottom of the boring was used) (Oregon International Port of Coos Bay 2016b). The modeled slope height spans from the channel bottom to the top of the adjacent banks in this scenario. The measured side slope was then applied to bottom of the slope (i.e., the bottom of the channel or the location where bedrock intersects the channel).

The model was run at different cross sections, each of which had a unique set of geotechnical conditions (see Sub-appendix 5: boring B-13 contained dense shell fragments while B-14 did not). The modeled side slopes varied from 9:1 to 17:1. The final value used for the future equilibrium side slope used to calculate capital dredge volume is 13:1, which is the median of the modeled side slope results. Because this method is used to estimate the expected equilibrium slope angles, the median of the model output slopes was used.

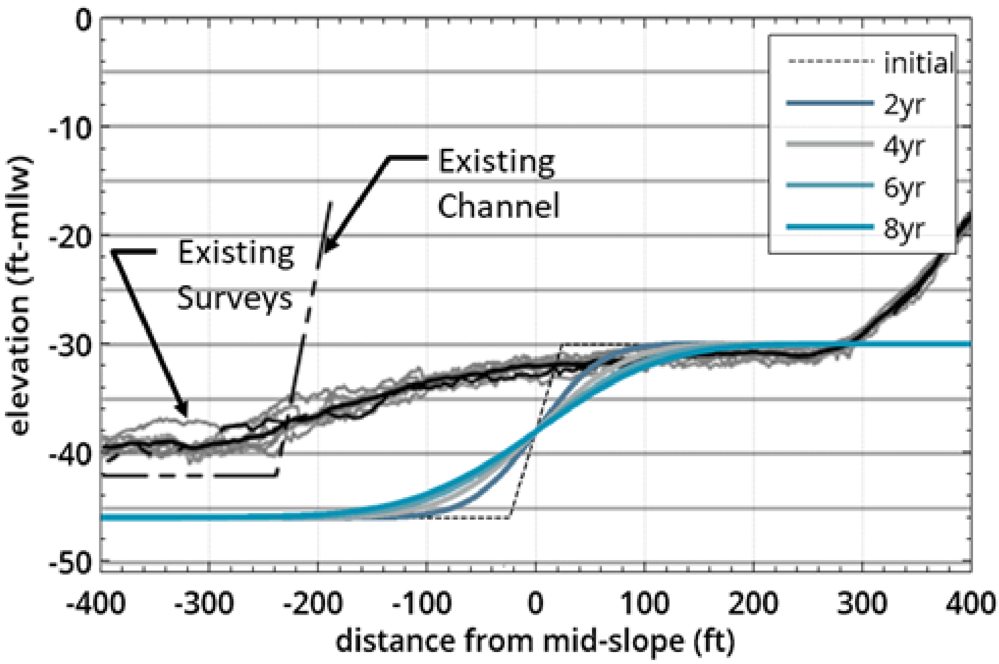


Figure 4-44
Estimated Side Slopes for the PA at Station 321+51 (output after 8 years shown)

4.2.8.4 Reach 7 – Conclusion

The future equilibrium side slopes measured from the toe of the dredged channel in Reach 7 are 15:1 on the left side and 32:1 on the right side. These values are used to estimate project effects and future O&M volumes. The right side contains a compound slope, with a steeper slope overlying the shallow 32:1 slope. Under implementation of the channel modification, future equilibrium side slopes based on long-term morphological behavior are 13:1 on both sides of the channel. For the flatter 32:1 slope on the right side, the slope is expected to retain its compound nature, where the steep upper slope would extend downwards to intersect with the milder bottom slope. A representative cross section of the equilibrium side slopes under the PA shown at a 10x vertical exaggeration can be seen in Figure 4-45. True-scale depictions of the left and right slopes can be seen in Figure 4-46 and Figure 4-47, respectively. Cross sections in this reach at both exaggerated and true scale for the construction and median side slope angles can be seen in Stations 5+32+68 through 6+19+32 in Sub-appendix 15.

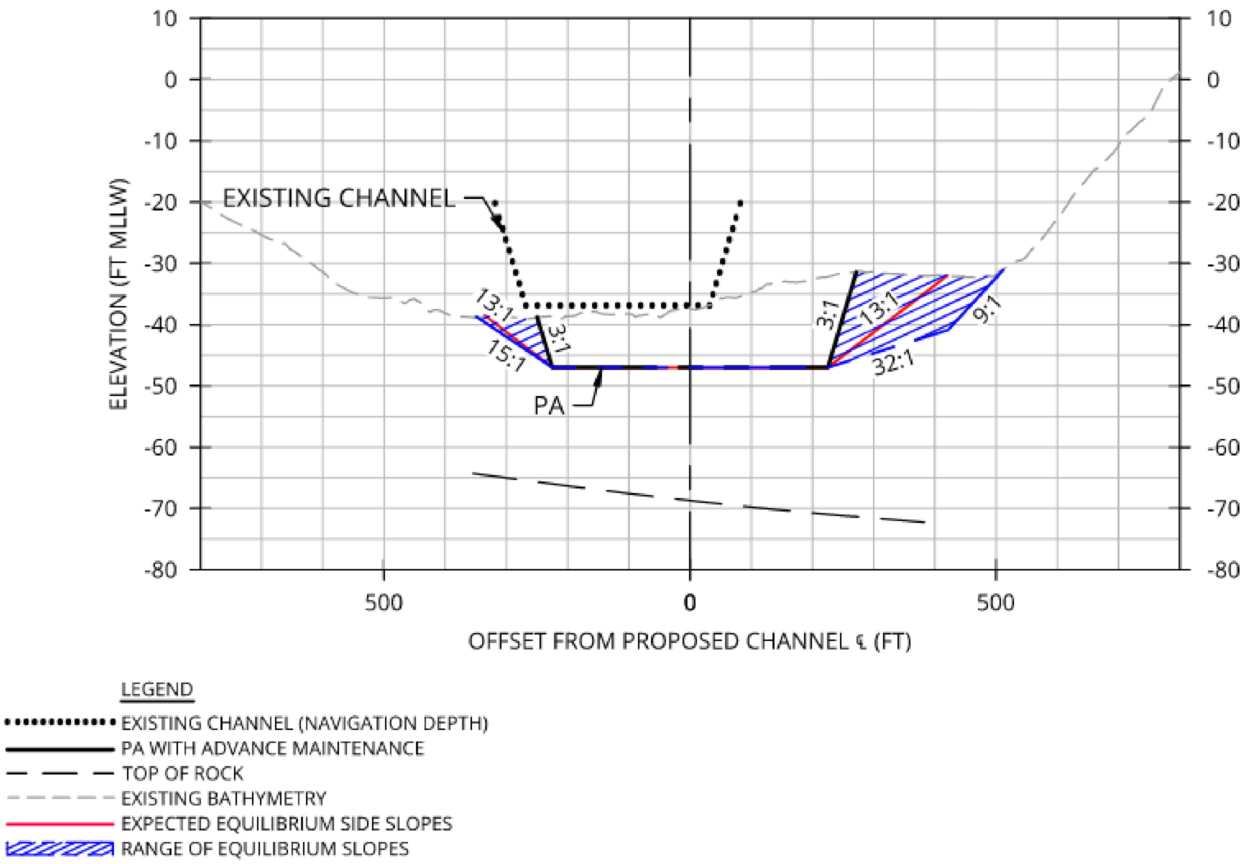


Figure 4-45
Reach 7 Representative Cross Section, PA Condition (Station 321+60)

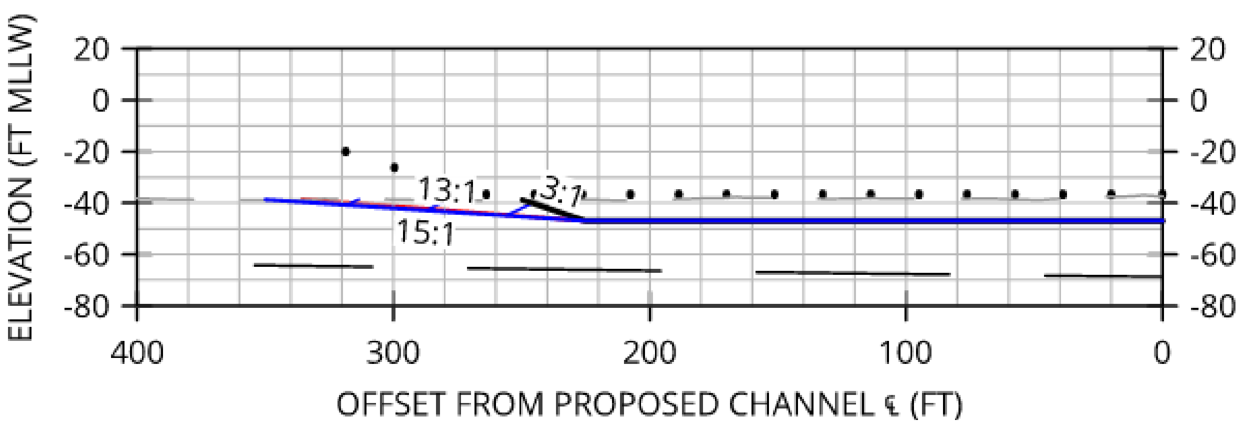


Figure 4-46
Future Equilibrium Side Slope in Reach 7, Left Slope

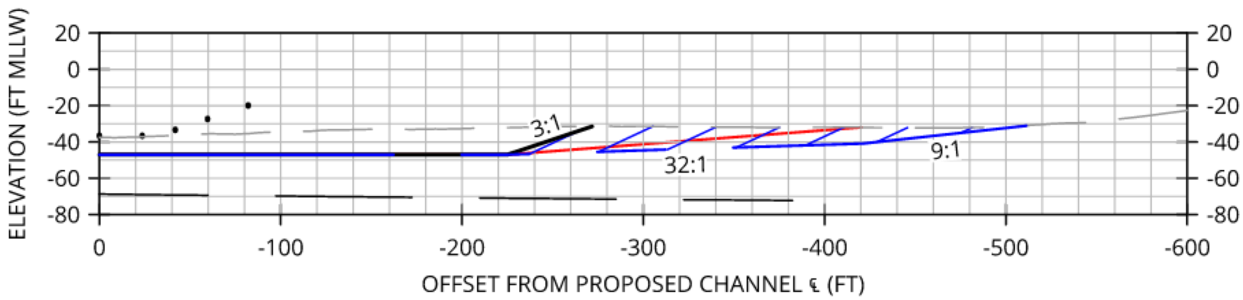


Figure 4-47
Future Equilibrium Side Slope in Reach 7, Right Slope

Future side slope equilibration is unlikely to affect pile dike CB-6.4; the pile dike is more than 300 ft away from the channel, and 200 ft away from where the side slope is expected to intersect the present bathymetry. A cross section showing the expected and median equilibrium side slopes emerging from the channel bottom can be seen in Figure 4-48. Additional cross sections at this pile dike can be seen in Sub-Appendix 13 (*Cross Sections)*, sections 6+19+41 through 6+19+91. Physical effects of side slope equilibration on the pile dikes are discussed in detail in Section 7.8.

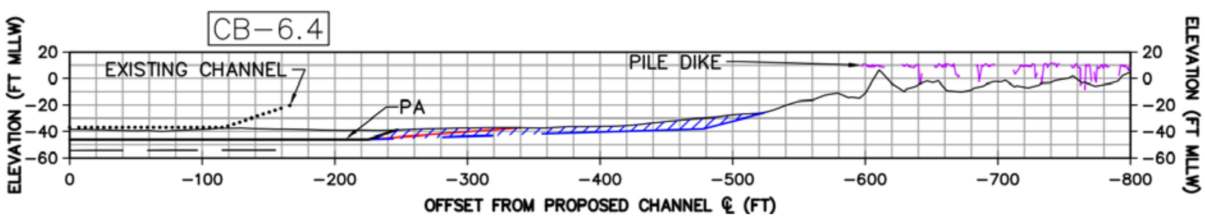


Figure 4-48
Future Side Slope Equilibration in the Vicinity of the Pile Dike CB-6.4

4.2.9 Reach 8

Reach 8 extends from RM 6.4 (Station 340+00) to RM 7.3 (382+00), encompassing Jarvis Turn. The curved nature of this reach leads to a horizontal distribution of currents with higher velocities along the outer bend and lower velocities along the inner bend. In addition, it appears that the natural thalweg has been migrating towards the North Spit, which causes erosion of the outer bank and shoaling adjacent to the airport. This erosion, while still ongoing, appears to have been reduced by the installation of five pile dikes in 1957.

The surveyed riverbed is shallower than the authorized depth on the left side of the channel, and equal to or deeper than the authorized depth on the right side of the channel. Rock is not present within this reach.

The pile dikes represent critical infrastructure to Coos Bay, they have been instrumental in maintaining the present alignment of the channel (i.e., they have prevented the channel from continuing to erode the North Spit). There are five pile dikes total, located to the right of the channel at RM 6.4, 6.6, 6.8, 7.0, and 7.3 (note that these structures are named after their river mile, e.g., the pile dike located at RM 6.6 is named CB-6.6). Four of them (excluding CB-6.4) are located within this reach. Equilibration of the side slopes is not expected to affect these structures.

As noted by recent surveys (Sub-appendix 2), the pile dikes are in a deteriorated condition. The survey found submerged piles that had been detached from the structure, mold and rot, debris attached to the structure, rusting hardware, and missing piles. This report considers the potential effect of channel deepening on the existing structure and foundation; it does not consider the deteriorated state of the structures.

4.2.9.1 Measured Side Slopes

High-quality cross-channel surveys conducted by USACE from 2009-2016 were used to measure the side slopes at six cross sections within this reach; these cross sections were selected because of their orientation perpendicular to the channel. The measured side slopes range from 13.9:1 to 26.3:1 on the left side of the slope, and from 1.9:1 to 12.8:1 on the right side of the slope. The median side slopes are 20:1 and 6:1 on the left and right side of the channel, respectively. Cross section locations are shown in Figure 4-49. The measured cross sections at these locations are shown in Table 4-8.

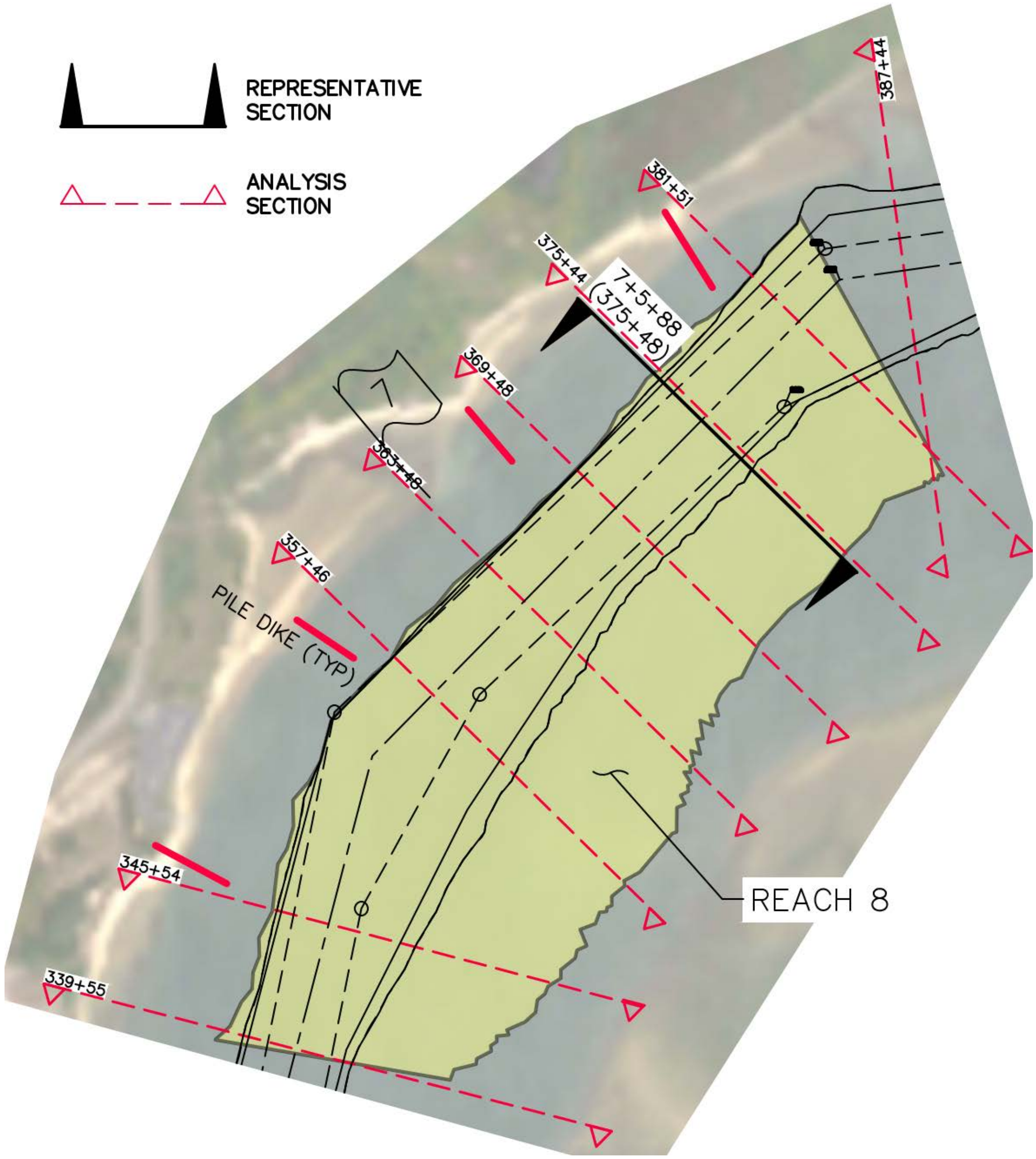


Figure 4-49
Cross Section Locations in Reach 8

Table 4-8
Side Slope Measurements in Reach 8

Station	Left Slope	Right Slope
345+54	21.5	7.5
357+46	26.3	3.5
363+48	22.8	4.1
369+48	18.3	1.9
375+44	13.9	12.8
381+51	14.6	12.8
Median	**19.9**	**5.8**

A representative cross section from this reach, taken at Station 375+48 (RM 7.1), is illustrated in Figure 4-50; this cross section is shown at a 10x vertical exaggeration. This cross section shows the left slope encroaching into the existing navigation channel; the channel's tendency to migrate towards the right bank results in shoaling along the left slope. While the individual shoals are stable at side slopes as steep as 6.6:1, the median side slope angle is 20:1. On the right side of the channel, the channel is naturally deep, and erosion at the bottom of the slope maintains a steep slope.

A pile dike (CB-7.0) is located within close proximity to Station 369+48; this structure results in the steep slope measured at this transect. Generally, the side slopes adjacent to the pile dikes were found to range from 1.8:1 to 2.3:1. These steep slopes could be due to the presence of rock that is stable at such slopes (see Section 7.7).

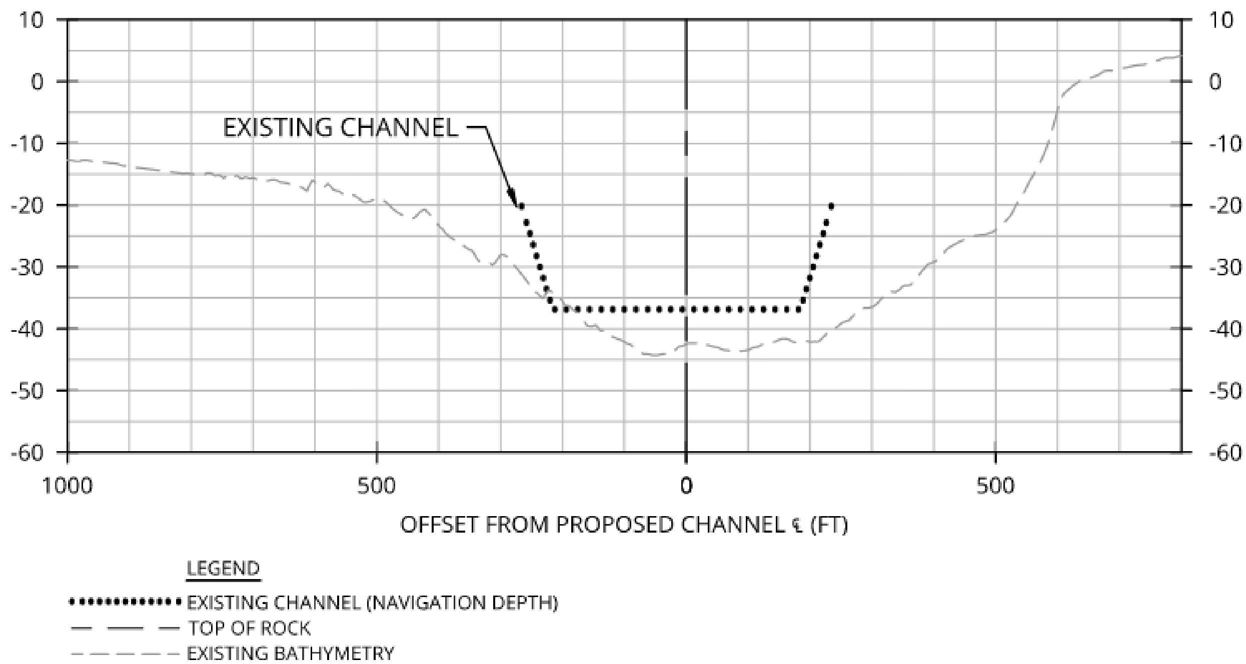


Figure 4-50
Reach 8 Representative Cross Section, Existing Condition (Station 375+48/RM 7.1)

4.2.9.2 Substrate Conditions

Rock is not expected to be present beneath the existing or proposed navigation channel. The substrate in this reach is sand with a median grain size of approximately 0.25 mm (Table 1B, Sub-appendix 5).

4.2.9.3 Long-Term Morphological Behavior

Using historic (pre-2016) navigation charts to trace morphological changes indicates that the channel has been shifting towards the pile dikes on the outer bend of the turn. The -18 ft MLLW contour line (3 fathoms) was digitized from Navigation Charts dating back to 1916. The evolution of this line can be seen in Figure 4-51. As shown in the figure, the outer bend underwent significant movement from 1916 through 1953, after which the location of the -18 ft contour was stabilized; stabilization appears to coincide with installation of the pile dikes. On the left bank, the channel appeared to similarly migrate inwards until the pile dikes were constructed. Subsequently, the contour has shifted outwards. This shift outwards may be a result of channel dredging.

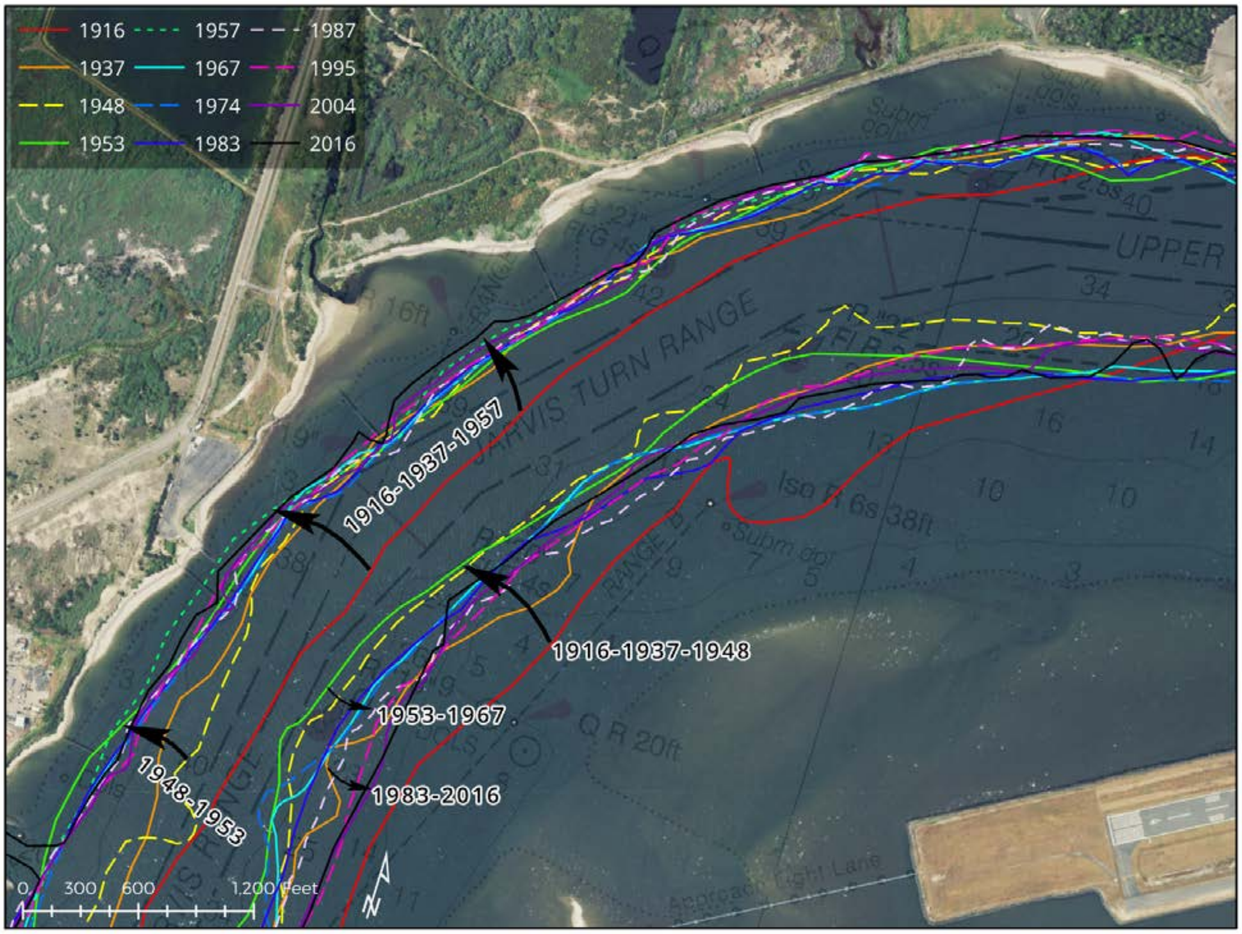


Figure 4-51
Migration of -18 ft MLLW Contour in the Jarvis Turn

The morphology differs significantly on the left and right sides of the channel. On the left side, shoaling is the dominant process. Figure 4-52 shows how shoaling appears at the bottom of the left slope, without any loss of material from the top of the slope. From 2009 to 2015, nearly 10 ft of material were deposited. Therefore, the sediment is likely sourced from elsewhere within the channel, potentially from upstream or from the erosional right bank. On this slope, a dynamic equilibrium is established between the processes of shoaling and removal of sediment via annual maintenance dredging. The slopes measured on the shoals themselves tend to be steeper than the angle over the entire side slope, indicating that the sediment may be morphologically stable at steeper angles. It should be noted that the slope angles in Table 4-8 describe the average angle over the entire slope, not the steep slopes measured on the shoals.

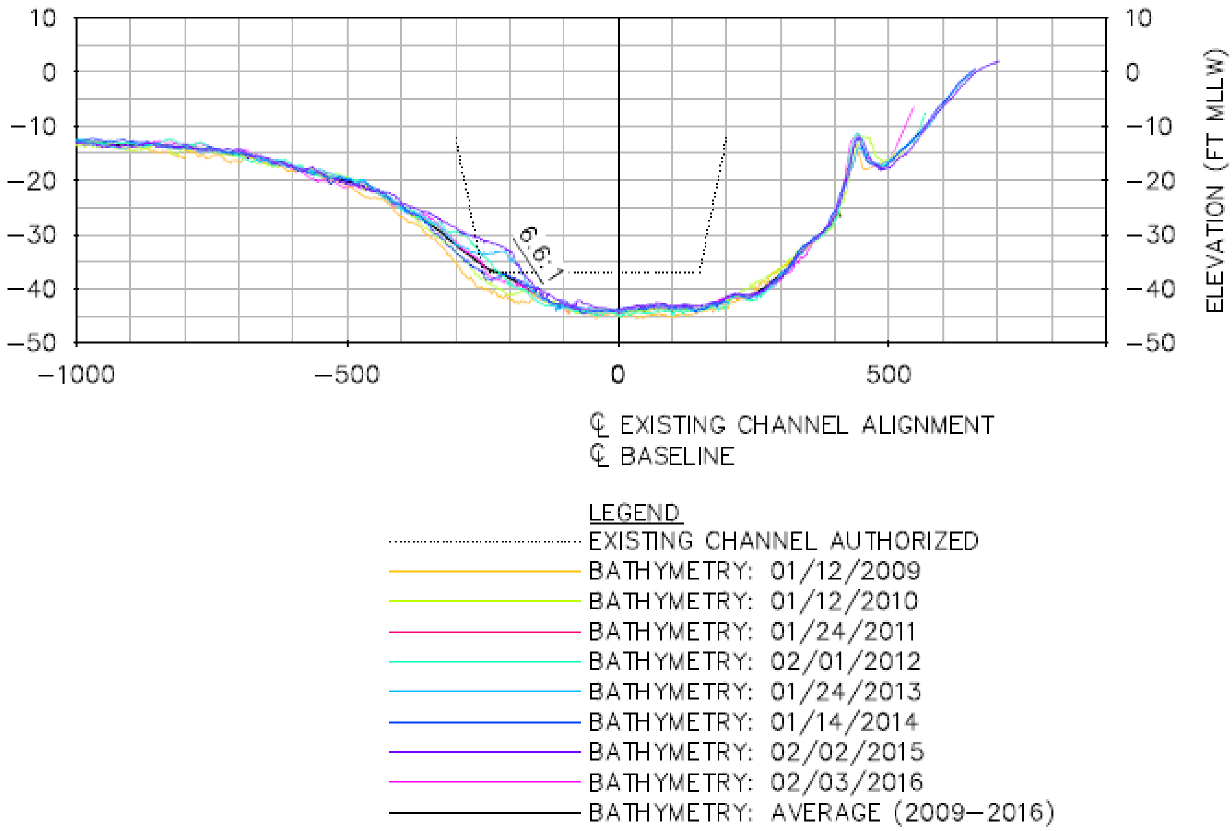


Figure 4-52
Shoaling on the Inner Slope of Station 375+44 (RM 7.1)

In this reach, it is expected that shoaling will continue after dredging; sediment from a different location (likely the outer bank of the bend) will fill in the bottom of the channel, while sediment at the top of the slope will remain stable. As this process dominates the left slope in Reach 8, side slope equilibration is not expected.

The right slope within this reach is erosional. Figure 4-53 shows erosion on the right bank of the channel. As this figure shows, the channel is progressively deepening each year, starting in 1997 and ending in 2016 (the last year of data). Over this time period, the elevation of the channel decreased by 10 ft. The result of this erosion is the progressive steepening of the slope. The measured side slope, which is a product of the ongoing erosion, is assumed to represent the future equilibrium side slope.

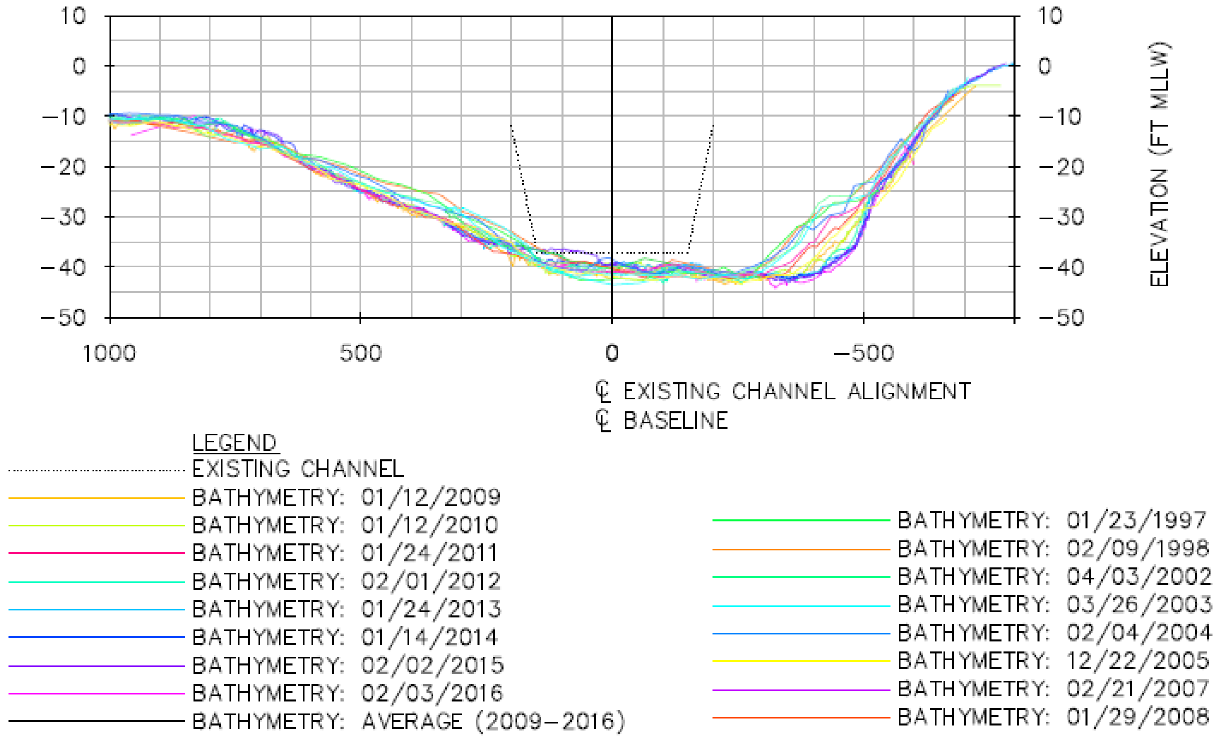


Figure 4-53
Erosion of on the Outer Slope at Station 345+54 (RM 6.5)

4.2.9.4 Reach 8 – Conclusion

Reach 8 will be dredged to a 4:1 slope on the left and a 3:1 slope on the right. After initial dredging, the left slope will fill in as a result of shoaling and will require regular maintenance dredging. Loss of material from the top of the cut, and the resultant side slope equilibration, is not expected on this slope. However, the resultant left slope could be as mild as 20:1, and a 20:1 slope has been used as the future equilibrium condition to evaluate equilibration volumes and channel effects. On the right slope, active erosion contributes to continual steepening of the side slopes. The median measured side slope is 6:1. This side of the channel is expected to re-equilibrate to this slope after dredging. In the vicinity of the turning basin, equilibration is expected to begin from the edge of the basin.

The future equilibrium side slopes based on long-term morphologic considerations in this reach can be seen in the representative cross section, depicted in Figure 4-54; this cross section is shown with a 10x vertical exaggeration. True scale representations of the left and right slopes are depicted in Figure 4-55 and Figure 4-56, respectively. The left and right equilibrium side slopes for this reach can be seen in exaggerated true scale in sections 6+21+12 through 7+10+56 (17 sections total) in Sub-Appendix 13 (*Cross Sections)*. The airport would be on the far left of the sections 7+0+00 through 7+10+56, however it is so far from the daylight of the 20:1 slopes that it is not present in these figures.

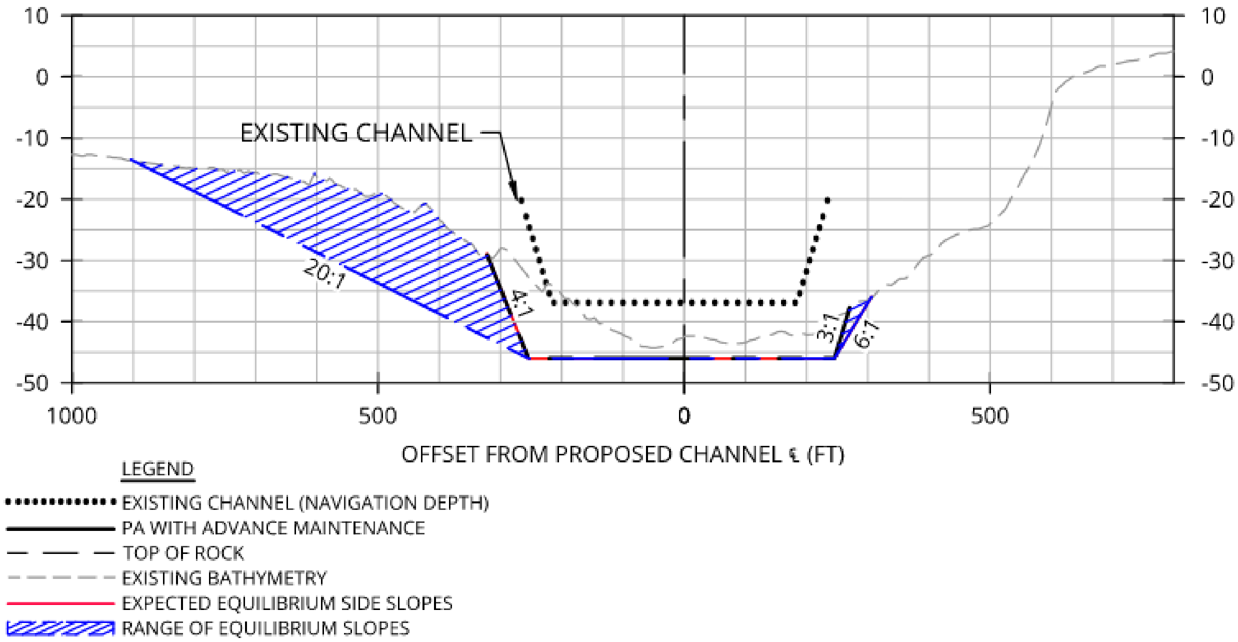


Figure 4-54
Reach 8 Representative Cross Section, PA Condition (Station 375+48)

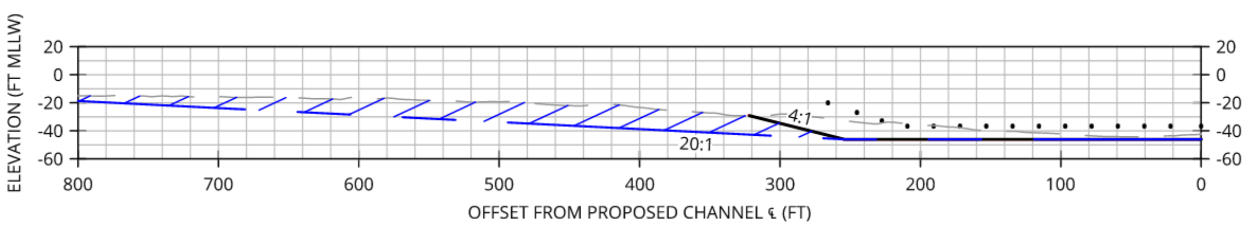


Figure 4-55
Future Equilibrium Side Slope in Reach 8, Left Slope

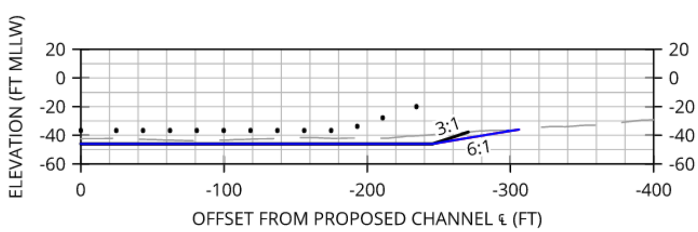


Figure 4-56
Future Equilibrium Side Slope in Reach 8, Right Slope

As noted above, the pile dikes represent critical pieces of infrastructure within this reach. Cross sections at each of the pile dikes, which include the equilibrium side slopes, are shown in Figure 4-57. Cross sections at each pile dike are also included in Sub-appendix 13 (*Cross Sections)*. These cross sections are shown in true scale, and only depict the portion of the slope in the vicinity of the pile dikes.

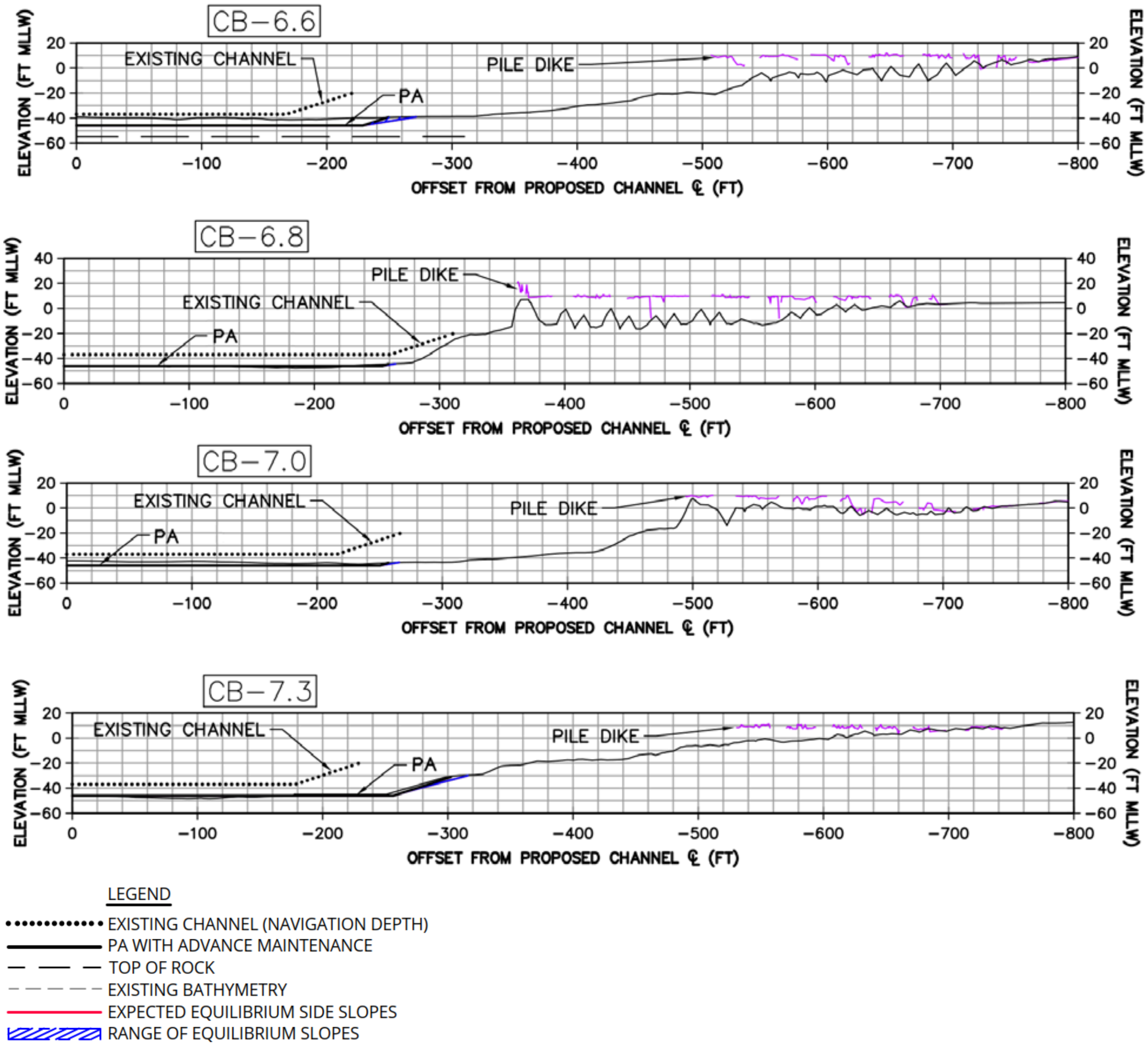


Figure 4-57
Future Side Slope Equilibration in the Vicinity of the Pile Dikes

This figure shows that the channel is naturally deep in this area, and that only limited dredging is required. Because the dredge cut is expected to be shallow, equilibration is expected to be limited. The limited equilibration that may occur has been shown to terminate more than 50 ft from the rock apron at the base of the pile dikes. Therefore, no effects on the pile dikes are expected.

In addition, the pile dikes are protected by a rock revetment provided along the length of each one structure and by a rock apron around the channel-most pile of each structure. The rock apron has a radius of 50 ft and a thickness of 3 ft. At the end of the structure, the stone revetment has a thickness of 6 ft, and extended 15 ft beyond the tip of the outer-most pile. As explained in Section 7.8.3, sufficient rock from the original placement is still present at the pile dikes to protect them against any potential (and unexpected) effects.

4.2.10 Reach 9

Reach 9 is the furthest upstream reach, located from the upstream extent of Reach 8 at RM 7.3 (Station 382+00) to the end of the proposed channel modifications at RM 8.2 (Station 430+00). This reach encompasses the upstream portion of the Jarvis Turn. The morphology of this reach is similar to that of Reach 8, in that it tends to be depositional on the inner bend (left side of the channel) and erosional on the outer bend (right side of the channel). This reach differs from Reach 8 in that the sediment is looser, potentially leading to geotechnical instability while performing deep dredge cuts.

The head cut at the upstream reach of dredging is not anticipated to equilibrate. The head cut has been designed at an approximately 30:1 side slope that will be dredged during construction (See Sub-appendix 8, Sheet C-204). This slope is not expected to equilibrate.

4.2.10.1 Measured Side Slopes

High-quality cross-channel surveys conducted by USACE from 2009-2016 were used to measure the side slopes at seven cross sections within this reach. The measured side slopes range from 11.9:1 to 32.9:1 on the left side of the slope, with a median side slope of 19:1. On the right side, the slopes range from 1.9:1 to 14.9:1 with a median side slope of 3:1. Since these slopes are milder than the upstream sections, they were included in the median slope calculation to be conservative. Cross section locations are shown in Figure 4-58. The measured cross sections at these locations are shown in Table 4-9.

On the left side of the channel, the airport is present. The airport is located 900 ft away from the proposed Turning Basin at the closest point. The large distance between the airport and the proposed dredging is sufficient to avoid potential effects at the airport even after future channel equilibration.

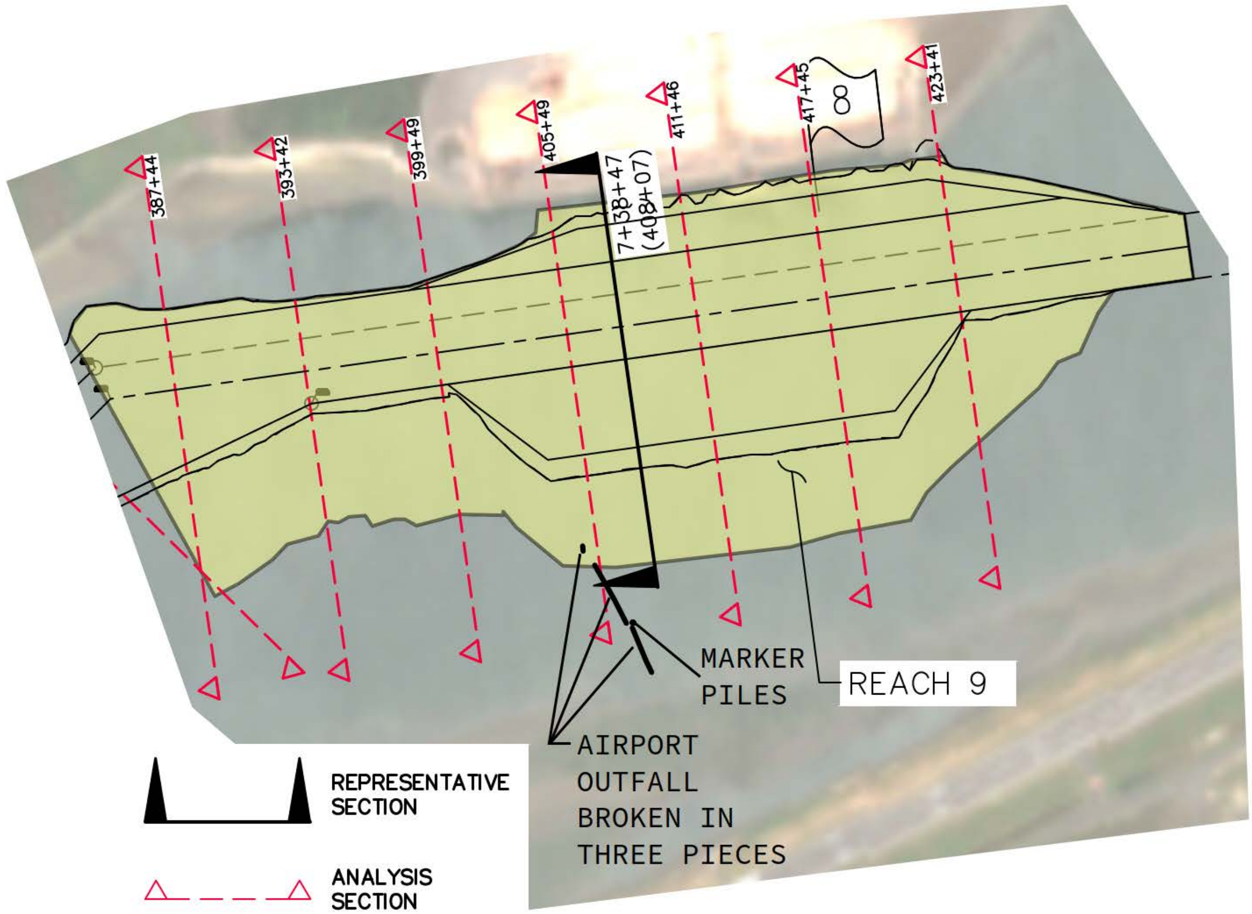


Figure 4-58
Cross Section Locations in Reach 9

Table 4-9
Side Slope Measurements in Reach 9

Station	Left Slope	Right Slope
387+44	18.9	9.9
393+42	17.7	14.9
399+49	19.9	8.7
405+49	27.5	2.9
411+46	32.9	2.2
417+45	17	1.9
423+41	11.9	2.3
Median	**18.9**	**2.9**

A representative cross section from this reach, taken at Station 408+07 (RM 7.7), is illustrated in Figure 4-59; this cross section is shown at a 10x vertical exaggeration. This cross section shows the left slope encroaching into the existing navigation channel; the channel's tendency to migrate towards the right bank results in shoaling along the left slope.

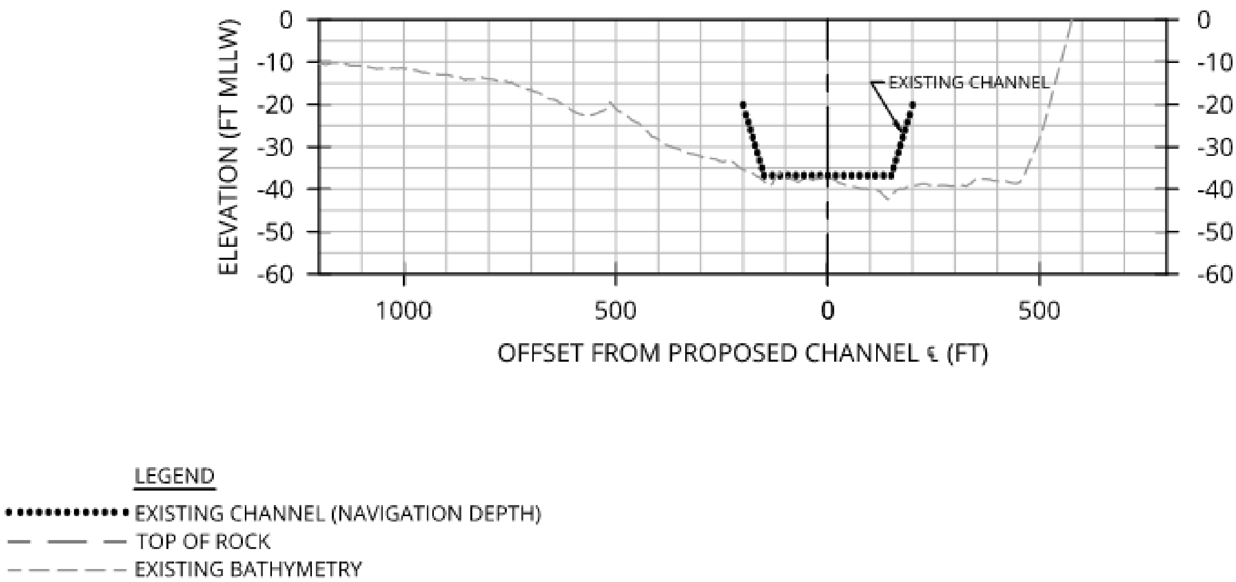


Figure 4-59
Reach 9 Representative Cross Section, Existing Condition
(Station 408+07/RM 7.7)

4.2.10.2 Substrate Conditions

Rock is not expected to be present beneath the existing or proposed navigation channel; loose to medium dense sand is present.

4.2.10.3 Long-Term Morphological Behavior

The morphology on the left slope is similar to the morphology on the left slope of Reach 8; that is, the slope is depositional. Because sediment tends to rapidly fill in from the bottom of the slope, equilibration is not expected, although the side slope equilibration may result in a slope as mild as 20:1. Please see the discussion in Section 4.2.9.3 for a detailed explanation.

On the right slope, the long-term slope is stable at 3:1. Therefore, no equilibration is expected along this slope.

4.2.10.4 Reach 9 – Conclusion

Reach 9 will be dredged to a 5:1 slope on the left side. After initial dredging, the left slope will fill in as a result of shoaling and will require regular maintenance dredging. Loss of material from the top of the cut, and the resultant side slope equilibration, is not expected on this slope. On the right side of the channel, the median measured side slopes are 3:1 – this represents the expected future equilibrium condition.

The future equilibrium side slopes as predicted by long-term morphological analysis in this reach can be seen in the representative cross section, depicted in Figure 4-60; this cross section is shown with a 10x vertical exaggeration. True scale representations of the left and right slopes are depicted in Figure 4-61 and Figure 4-62, respectively. True scale cross sections of this reach are provided in Sub-appendix 13 (*Cross Sections),* Sections 7+21+12 through 8+00+00 (10 sections total). The side slopes daylight more than 900 feet away from the airport so no impact on airport infrastructure is expected. Physical effects of future side slope equilibration on airport are summarized in Section 7.9.

The Airport Outfall can be seen in Sections 7+30+95 through 7+31+48. The daylight of the 20:1 Slope comes closest to the structure in 7+31+21, where it daylights 120 ft from the structure. The 120 ft between the daylight and the structure is flat. Even if the top portion of the daylight would round off, there is still 100 ft of sediment protecting the outfall. Therefore, no effects are anticipated. Physical effects of side slope equilibration on outfalls are summarized in Section 7.10.

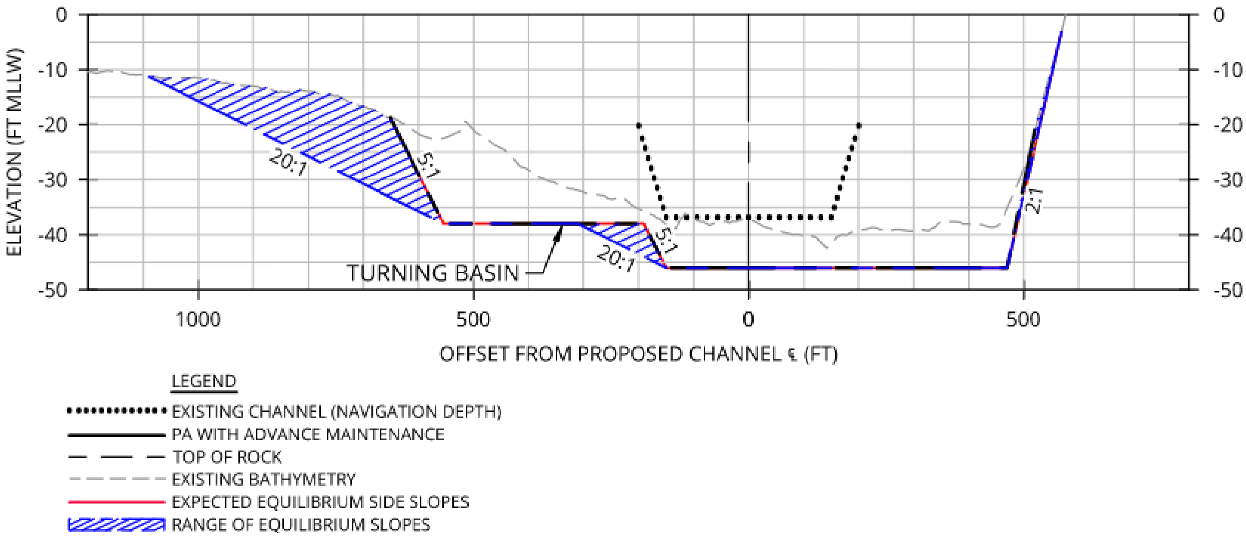


Figure 4-60
Reach 9 Representative Cross Section, PA Condition (Station 408+07)

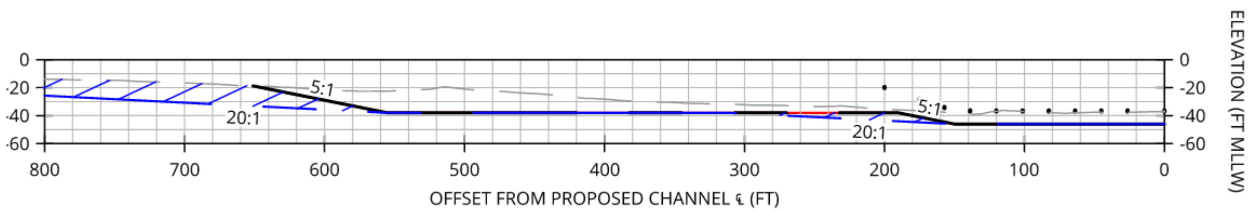


Figure 4-61
Future Equilibrium Side Slope in Reach 9, Left Slope

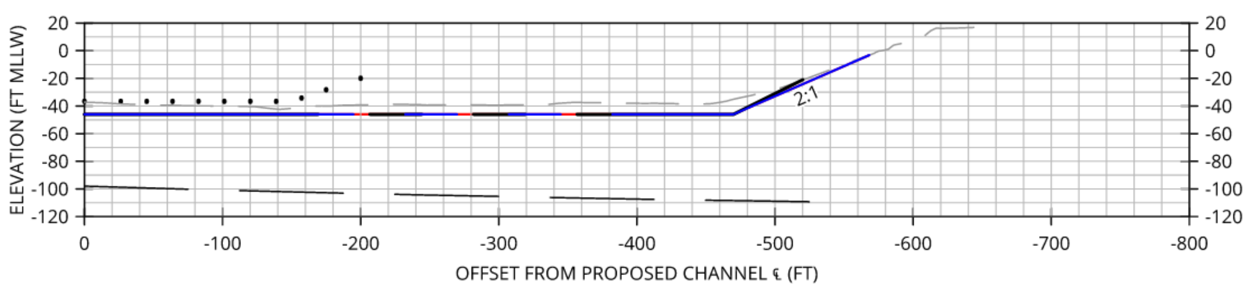


Figure 4-62
Future Equilibrium Side Slope in Reach 9, Right Slope

4.3 Summary of Side Slope Analysis Results

The methodology for predicting future side slope equilibration follows a sequential process that analyzes the measured side slopes and relates these slopes to the geotechnical and morphological processes on both banks in each reach. Certain processes that result from channel modification are identified, and analysis methods estimating how these processes will impact a wider, deeper channel are used to predict equilibrium side slopes. Other morphological processes that are independent of channel modification are identified, and these processes are expected to continue into the future and are not expected to drive side slope equilibration.

Table 4-10 summarizes the results of the side slope analysis methodology discussed herein. The columns under “Measured Side Slopes” heading contain the range of side slope angles based on the measured side slopes and also indicate areas where dredging will occur in rock and future side slope equilibration is not expected. The column “Geotechnical Considerations and Determination of Side Slopes for the Constructed Condition” contains the geotechnical processes that are critical to Assessment 3. The Column “Morphological Behavior and Determination of Future Equilibrium Side Slopes” contains the results of Assessment 4. This column summarizes the morphological processes that occur in each reach. Columns under “Channel Condition Side Slopes” heading contain the estimated equilibrium side slopes for the Constructed Channel condition, Future Equilibrium Side Slope for Capital Dredging Estimate, and Future Equilibrium Side Slope for Effects Analysis and O&M Estimate resulting from the analysis in the prior sections.

Table 4-10

Results of Side Slope Equilibration Analysis

REACH	**SIDE**	**MEASURED SIDE SLOPES**			**Geotechnical Considerations and Determination of Side Slopes for the Constructed Condition**	**Morphological Behavior and Determination of Future Equilibrium Side Slopes**	**CHANNEL CONDITION SIDE SLOPES**		
		Flattest	*Median*	*Steepest*			*Constructed Channel Condition*	*Future Equilibrium Side Slope for Capital Dredging Estimate*	*Future Equilibrium Side Slope for Effects Analysis and O&M Estimate*
Reach 1 (RM -0.85 - 0.1)	Left (South)	flat	16:1	5.7:1	Relative density information in this reach is not available. Loose sands are assumed to be present within this reach and the risk of flow failures indicates a 4:1 slope should be dredged.	This reach experiences currents perpendicular to the channel alignment. These currents drive cross-channel bedload that transport sediment from the top of the slopes into the channel. The expected equilibrium side slope anges were based on Raaijmakers' (2005) forumal, which resulted in a side slope angle of 9:1.	4:1	9:1	16:1
	Right (North)	flat	29:1	5.7:1			4:1	9:1	29:1
Reach 2 (RM 0.1 - 0.3)	Left (South)	34:1	22:1	18:1	Relative density information in this reach is not available. Loose sands are assumed to be present within this reach and the risk of flow failures indicates a 4:1 slope should be dredged.	This slope is adjacent to (left slope) or contains (right slope) a large scour hole; sediment is constantly removed from the bottom of the slope, effectively steepening the slope. The expected side slope is estimated to be equivalent to the median measured side slope	4:1	22:1	22:1
	Right (North)	31:1	15:1	11:1			4:1	15:1	15:1
Reach 3 (RM 0.3-0.9	Left (South)	35:1	22:1	14:1	Relative density information in this reach is not available. Loose sands are assumed to be present within this reach and the risk of flow failures indicates a 4:1 slope should be dredged.	This reach experiences shoaling. Immediately after dredging, sediment from offshore is deposited, further stabilizing the slope. Review of hisptoric pre- and post-dredge surveys indicates that sediment does not erode from the top of the slope. Therefore, the expected side slope is the same as the construction side slope (4:1).	4:1	4:1	22:1
	Right (North)	35:1	15:1	11:1			4:1	4:1	15:1

REACH	SIDE	MEASURED SIDE SLOPES			Geotechnical Considerations and Determination of Side Slopes for the Constructed Condition	Morphological Behavior and Determination of Future Equilibrium Side Slopes	CHANNEL CONDITION SIDE SLOPES		
		Flattest	*Median*	*Steepest*			*Constructed Channel Condition*	*Future Equilibrium Side Slope for Capital Dredging Estimate*	*Future Equilibrium Side Slope for Effects Analysis and O&M Estimate*
Reach 4 (RM 0.9 - 2.0)	Left (South)	18:1	13:1	8:1	Loose sand may be present on this slope and the risk of flow failures will be considered. Because dredge cuts are expected to be relatively shallow, a 3:1 slope will likely be stable.	Rock is present through the majority of this reach. Portions that contain sediment are expected to be erosive, and therefore expected side slopes are assumed to be equal to the median side slopes (13:1).	3:1	13:1	13:1
	Right (North)	22:1	18:1	11:1		Due to the curved nature of this reach, this slope is expected to shoal. The time series of bathymetries do not provide evidence of these processes; the expected side slopes are assumed to be equal to the median side slopes (18:1).	3:1	18:1	18:1
Reach 5 (RM 2.0 - 4.5)	Both	N/A - Rock			The presence of relatively dense sands over rock within this reach indicates that the risk of flow failures is low. A 3:1 slope will likely be stable in the sand overburden.	N/A	N/A - Rock		
Reach 6 (RM 4.5 - 5.6)	Both	25:1	11:1	6:1	The presence of relatively medium dense to dense sands within this reach indicates that the risk of flow failures is low. A 3:1 slope will likely be stable.	Currents parallel to the channel are expected to initiate bed load with a downward component. The downward component will transport sediment from the upper portion of the slope into the channel, flattening the slope. The expected side slope (9:1) is predicted by the Fredsoe (1978) method.	3:1	9:1	11:1

REACH	SIDE	MEASURED SIDE SLOPES			Geotechnical Considerations and Determination of Side Slopes for the Constructed Condition	Morphological Behavior and Determination of Future Equilibrium Side Slopes	CHANNEL CONDITION SIDE SLOPES		
		Flattest	*Median*	*Steepest*			*Constructed Channel Condition*	*Future Equilibrium Side Slope for Capital Dredging Estimate*	*Future Equilibrium Side Slope for Effects Analysis and O&M Estimate*
Reach 7 (RM 5.6 - 6.4)	Left (East)	25:1	15:1	11:1	The presence of relatively dense sands within this reach indicates the risk of flow failures is low. A 3:1 slope will likely be stable.	Currents parallel to the channel are expected to initiate bed load with a downward component. The downward component will transport sediment from the upper portion of the slope into the channel, flattening the slope. The expected side slope (13:1) is predicted by the Fredsoe (1978) method. This slope differs from Reach 6 because of the different sediment and hydrodynamic conditions.	3:1	13:1	15:1
	Right (East)	flat	32:1	4:1			3:1	13:1	32:1
Reach 8 (RM 6.4 - 7.4)	Left (East)	26:1	20:1	14:1	The presence of relatively loose sands within this bank indicates that flow failures will be a significant geotechnical consideration. Dredging at 4:1 is recommended to reduce the risk of flow failure	Shoaling is expected to occur on this side slope; historic bathymetry surveys do not show loss of material from the top of the slope. Therefore, the expected side slope is equivalent to the construction side slope (4:1).	4:1	4:1	20:1
	Right (East)	13:1	6:1	2:1	The presence of relatively dense sands within this bank indicates the risk of flow failures is low. A 3:1 slope will likely be stable.	Erosion is expected to continue after dredging, steepening the slope. Therefore, the expected side slope is equivalent to the median side slope.	3:1	6:1	6:1
Reach 9 (RM 7.2 - 8.2)	Left (South)	33:1	19:1	12:1	The presence of relatively loose sands within this bank indicates that flow failures will be a significant geotechnical consideration. Dredging at 5:1 is recommended to reduce the risk of flow failure	Shoaling is expected to occur on this side slope; historic bathymetry surveys do not show loss of material from the top of the slope. Therefore, the expected side slope is equivalent to the construction side slope (5:1).	5:1	5:1	20:1
	Right (North)	15:1	3:1	2:1	The likely presence of relatively dense sands in this bank indicates the risk of flow failures is low. A 3:1 slope will likely be stable.	This slope is presently stable at a 3:1 slope, and this is the projected long-term angle.	3:1	3:1	3:1

5. IMPLEMENTATION OF EQUILIBRIUM SIDE SLOPES

The side slopes presented in Table 4-10 represent the side slope angles that were calculated to fit the bulk properties of each reach. However, due to discontinuities in the bathymetric data and heterogeneities in the subsurface such as rock outcroppings, the slopes presented cannot be applied uniformly within each reach. The discussion presented in this section explains how the slope angles were used to develop a 3D model that could be used for further analyses.

The complete extents of the PA channel, including the side slopes, is represented by the zone of the equilibration surface. The dredge depth components modeled are shown in Table 5-1. AMD width was not considered in this report, although a rock buffer width of 25 ft (50 ft near Guano Rock) is included to provide adequate clearance for dredge equipment. These depths were determined based on conversations with USACE. The turning basin is located in Reach 9 and is dredged to -38 ft MLLW.

Table 5-1
Modeled Depths and Side Slope for Each Reach

	Station Range	Future Equilibrium Slope Left | Right (downstream view)	Depth Components for Zone of Equilibration Mapping			
			Auth.	AM	Rock Buffer	
					Width	Depth
Reach 1	-45+00 – 7+00	16:1 | 29:1	57	6	No Rock	
Reach 2	7+00 – 17+00	22:1 | 15:1	57	6	No Rock	
Reach 3	17+00 – 47+50	22:1 | 15:1	45 – 57'	1 – 6	25'	1'
Reach 4	47+50 – 105+00	13:1 | 18:1	45	1	25'	1'
Reach 5	105+00 – 237+50	11:1 | 11:1	45	1	25'	1'
Reach 6	237+50 – 297+50	11:1 | 11:1*	45	1	25'	1'
Reach 7	297+50 – 340+00	15:1 | 32:1	45	1	25'	1'
Reach 8	340+00 – 382+00	20:1 | 6:1	45	1	No Rock	
Reach 9	382+00 – 430+00	20:1 | 3:1	45	1	No Rock	

**2:1 slopes are used in rock and loose sediment for the proposed Container Berth slopes*

Table 5-2
Zone of Equilibration Model Details and Input-Data Sources

Project Depth – Entrance Channel	-57	ft-MLLW
Project Depth – inner channel	-45	ft-MLLW
Advanced Maintenance (AM) – Entrance Channel	6	ft
AM – inner channel	1	ft
Rock Buffer	1	ft
Rock Overwidth (at Guano Rock)	25	Ft (50 ft)
Overdepth	0	ft
Empire Turning Basin (including AM)	-47	ft-MLLW
Container Berth Slip (including AM)	-52	ft-MLLW
RM 8 Turning Basin Depth (including AM)	-38	ft-MLLW
Roseburg Berth	-38	ft-MLLW
Rock Free Surface Version	GHDv2	Mar-24
Bathy Model	v5	May-24
Channel Side Slope Condition	Future Equilibrium	

5.1 Concepts in Implementation

The channel with future equilibrium sides slopes was modeled using Autodesk Civil 3D and Autodesk Subassembly Composer. The model template was configured in the Subassembly Composer and written so that the template could adapt to a variety of conditions. Figure 5-1 shows a simplified version of the model template logic that was created within the Subassembly Composer. The channel template model was then loaded into Civil3D, the inputs were defined (i.e., substrate material, presence of a compound slope, or manual daylight point/structure – as explained below), and the full channel was built.

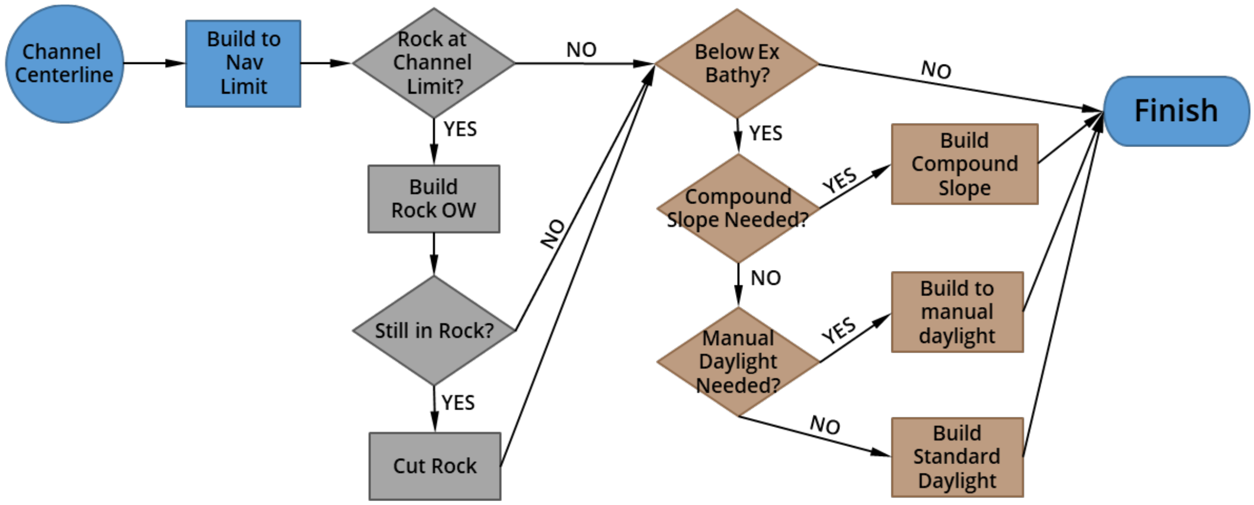


Figure 5-1
Simplified Civil 3D Subassembly Model

The following daylighting techniques are incorporated into the model in order to develop equilibrated slopes:

- **Standard Daylighting.** The daylight point (location where a proposed slope intersects the proposed bathymetry) is computed from channel limit (plus applicable advance maintenance, additional depth buffer for rock, and additional width buffer for rock) up to the bathymetric surface using the equilibrium side slope identified in Table 4-10; this is illustrated in Figure 5-2. Alternative approaches were only utilized when this standard approach was not suitable or representative of actual site conditions. Standard daylighting was applied more than 80% of the time.
- **Compound Slope.** The slope of a stable, steeper upper-bank or a rock structure is allowed to extend down until it intersects the side slope as prescribed in Table 4-10. In these regions, the zone of equilibration extends up to the bank toe, or structure toe; this configuration is shown in Figure 5-3.
- **Manual Daylighting.** A daylight line may be manually adjusted to account for actual site conditions not fully captured by the model, the bathymetry, and/or the rock model. The manual daylight is developed by examining cross sections and employing engineering judgement. Between side slope reaches, manual daylight lines were often used to produce a smoother, realistic transition. The manual daylight was also used to filter jagged, unrealistic daylight lines that may be cause by survey artifacts or transient sand waves.

In areas with little or no sediment above the rock surface, slope equilibration was not modeled. In the model, this was defined as cuts computed to be less than 1 ft. Often, thin overburdens calculated in the model may not actually be present, but a result of comparing the coarsely gridded bathymetric surface and rock models.

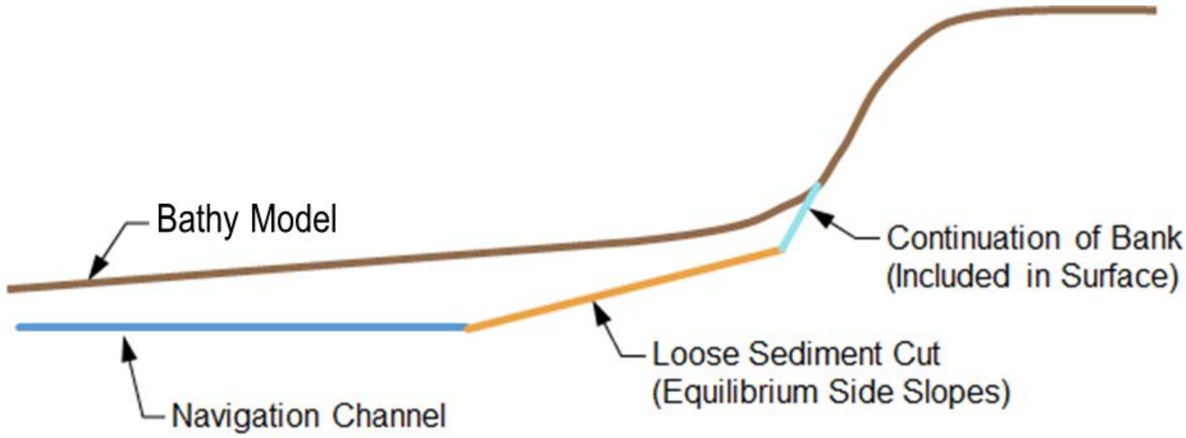


Figure 5-2
Typical Section in Rock

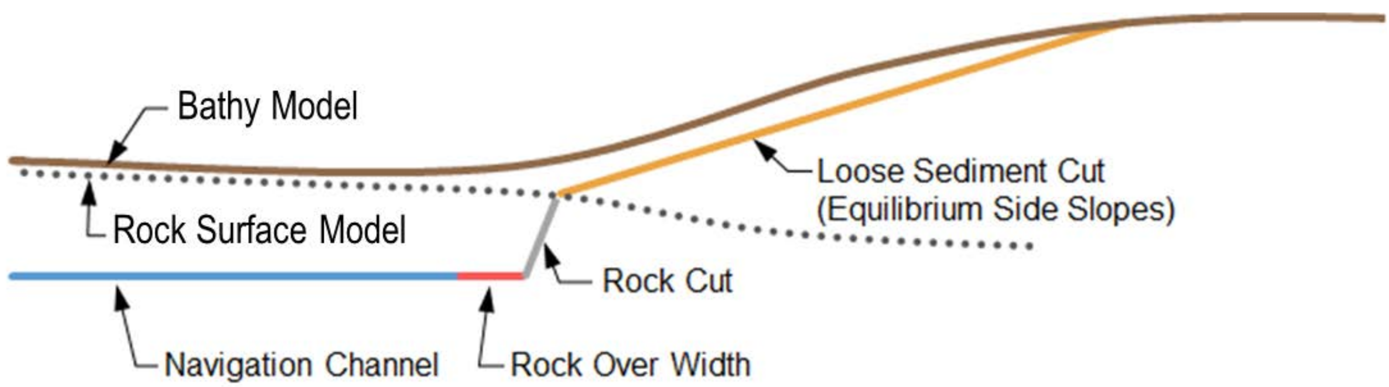


Figure 5-3
Typical Section with a Compound Slope

5.2 Specific Results

This section notes where the three configurations above were applied within the channel. Standard daylighting is used unless noted.

5.2.1 Reach 1 (Station -45+00 – 7+00)

Standard daylighting was implemented throughout Reach 1 using future equilibrium side slopes of 19:1.

5.2.2 Reach 2 (Station 7+00 – 17+00)

Right (north) side

- The future equilibrium side slope is 15:1.
- A rock toe apron is proposed over most of this side of the reach (from 8+00 to 17+00) to address for potential jetty toe scour if future equilibrium side slopes are realized in the future. Under potential future conditions, the rock toe apron adjusts, protecting the existing structure from toe scour. This adjusted apron is expected to have a slope equal to the existing toe of jetty slope. Therefore, the compound slope approach is appropriate.

Left (south) side

- The future equilibrium side slope is 22:1.
- Limited manual daylighting (filtering) is employed on this side of the channel. This consists of smoothing the daylight point so that the sudden, steep variations in bathymetry do not drive severe fluctuations in the daylight line location. The steep variations are driven by discontinuities where two surveys are merged and mobile sand waves. Therefore, the manual daylighting smooths over data irregularities and temporary transient features.
- The daylight line of the future equilibrium side slopes is 50 to 200 ft from the South Jetty's relic apron, and closest to the jetty at station 30+00.

5.2.3 Reach 3 (Station 17+00 – 47+50)

Right (north) side

- The future equilibrium side slope is 15:1.
- The compound slope template from Reach 2 continues into Reach 3, terminating at 31+00.

Left (south) side

- The future equilibrium side slope is 22:1.
- On left (south) side, manual daylighting (filtering) was used to smooth the erratic, jagged standard daylight line caused by sand waves in the bathymetry model.
- In the Guano Rock vicinity, the rock surface is visible on the surface, with little-to-no overburden. A 50-ft overwidth beyond the channel limits is employed to improve navigation and the maintainability of the channel. Here, the dredging cut is all rock and the side slopes daylight at a 1:1 slope.
- Reach 3 ends within the Guano Rock region.

5.2.4 Reach 4 (Station 47+50 – 105+00)

Right (north/west) side

- The future equilibrium side slope is 18:1; standard daylighting is utilized except where noted as follows.
- Between 48+00 and 54+00 the future equilibrium side slopes reach the North Jetty. A toe apron is proposed to address the possible effects of side slope equilibration on the jetty. The compound slope approach is used to represent an adjusted toe apron meeting the equilibrium side slope.
- From 70+00 to 82+00 the side slopes are created using manual daylighting in order to smooth the position of the daylight line, and transition into a portion of the channel that does not require dredging.
- Near 87+00, manual daylighting was used to transition to a compound slope template.
- A compound slope section spans from 90+00 to 97+00. Here the stable bank will extend down and meet the equilibrium side slope.

- At 97+00, the present bathymetry is sufficiently deep and no dredging is required until Reach 5.

Left (south/east) side

- Begins in the Guano Rock region, which extends through 53+00.
- From 53+00 to the corner at 65+00, standard daylighting is used with a future equilibrium side slope of 13:1.
- At the southern angle point of the channel limits near 69+00, manual daylighting is used to smooth the transition into a large rock cut.
- From 69+00 to 91+00 the future equilibrium side slope is 9:1 to match the present bathymetry.
- From 91+00 to the end of Reach 4, standard daylighting is used with a future side slope of 13:1.

5.2.5 Reach 5 (Station 105+00 – 237+50)

In Reach 5, manual daylighting is used sporadically (4 locations, 400 ft total) to limit unrealistic spikes in the areas of effect. Often these spikes are caused by the model finding sediment overburdens which do not actually exist. The false overburden is a result of the rock surface and bathy surface being on different grids.

Right side (northwest side)

- The future equilibrium side slope is 11:1.
- Manual daylighting is used near 108+50 where the side slope would stop at a relic stone mound. The relic mound is located at the site of an old material offloading pier.
- From 202+00 to 207+00, manual daylighting is used to account for high points in the rock surface outside the channel.

Left side (southeast side)

- The future equilibrium side slope is 11:1.
- With the exception of a few locations where smoothing of the slope transitions was needed, standard daylighting is used.

5.2.6 Reach 6 (Station 237+50 – 297+50)

Right side (northwest side)

- The future equilibrium side slope is 11:1.
- From 280+00 to 285+50, the compound slope template is used.
- Upstream of 285+50, manual daylighting is used to transition out of the compound slope template.
- The proposed container slip is present within this reach. The container slip will be armored following construction. The slopes will be dredged to 2:1 in rock and sand.

Left side (southeast side)

- The future equilibrium side slope is 11:1 and standard daylighting is used.

5.2.7 Reach 7 (Station 297+50 – 340+00)

Right side (northwest side)

- The future equilibrium side slope is 32:1 and a compound slope template is used from Station 303+00 to 332+00.
- Manual daylighting is used at the beginning (up to Station 304+50) and end of the reach (starting at Station 335+00) to transition into the compound slope template.

Left side (southeast side)

- The future equilibrium side slope is 15:1 and standard daylighting is used.
- Manual daylight is used to transition into Reach 8.

5.2.8 Reach 8 (Station 340+00 – 382+00)

Standard daylighting is used throughout Reach 8. The future equilibrium side slope is 4:1/20:1 on the left side, and the future equilibrium side slope 6:1 on the right side.

5.2.9 Reach 9 (Station 382+00 – 430+00)

Standard daylighting is used throughout Reach 9. The future side slope is 5:1 and 20:1, respectively, on the left side, and future equilibrium side slope is 3:1 on the right side.

6. EQUILIBRIUM DREDGE QUANTITIES

This section notes expected shoaling volume as a result of future side slope equilibration. Shoaling volumes are calculated by developing a 3D model of the design channels future equilibrium side slope conditions. Both the "expected" future equilibrium side slopes based on long-term morphological conditions, as used to calculate capital dredge quantities, and the "conservative" future equilibrium side slopes used to estimate project effects and calculate future O&M dredge quantities are analyzed and compared to the present bathymetry. Shoaling volumes beyond the initial construction are calculated for both of the future equilibrium conditions. These total volumes can be converted to annualized volumes by fitting the total volume to an exponential curve. In Reaches 1 through 3 (RM -1 through 1), slope equilibration was assumed to occur within one year due to the high-energy environment in these reaches. In the upstream reaches, shoaling was assumed to occur over approximately 8 years (Section 3.2.4.3). According to the Corps engineering experience, this decrease can be approximated by an exponential decay curve (Babcock 1989).

6.1 Proposed Alteration (PA)

The range of sediment volume that is estimated to result from equilibration of the PA side slopes is presented in Table 6-1. Future side slope equilibration is expected to result in 1.44 mcy, but may be as high as 4.71 mcy.

Table 6-1
Future Side Slope Equilibration Volumes by River Mile (kcy)

RM:	-1 – 0	0 – 1	1 – 2	2 – 3	3 – 4	4 – 5	5 – 6	6 – 7	7 – 8.2	TOTAL
Expected Value (for Capital Dredge Estimate)	178 k	289 k	556 k	3 k	16 k	82 k	170 k	149 k	-	1,443 k
Conservative Value (for Future O&M Estimate)	591 k	1,075 k	561 k	7 k	35 k	171 k	251 k	1,112 k	905 k	4,707 k

The conservative values have been incorporated into the O&M section of the Engineering Design Report (Section 9). This section incorporates the construction schedule, such that equilibration begins after dredging occurs in any given year.

7. PHYSICAL EFFECTS OF SIDE SLOPE EQUILIBRATION

This section uses the future equilibrium side slopes to estimate physical effects to federal infrastructure and adjacent resources. A map showing all of the areas where sedimentation patterns may be affected, using the future equilibrium side slope estimates, is presented in Section 7.1. The subsequent sections, 7.2 through 7.10, analyze infrastructure immediately adjacent to the channel. This analysis shows that portions of the North Jetty (head and trunk) are sufficiently close to the channel that portions of the structure may be undercut; therefore, a rock apron has been proposed to protect against the potential effects of undercutting. Effects are not expected at the North Jetty root/rail spur, the South Jetty, the *Rossell* shipwreck, the pile dikes, the SWORA, or the outfalls.

7.1 Zone of Equilibration

The zone of equilibration is defined as the region within the side slope daylight; essentially, it represents the entire area that may be influenced by dredging. The zone of equilibration for the PA with future equilibrium side slopes is shown in Figure 7-1. The surfaces presented in these figures also include the elevation of the channel/equilibrium side slope surface at each location within the zone of equilibration. The zone of equilibration surface was imported into ArcGIS and submitted to the project design team as a 3-dimensional triangular irregular networks (TIN) surface. This zone addresses the full length of the channel and beyond until the future equilibrium side slopes daylight to bathymetry elevation model. The acreage covered by the equilibrating side slopes of the future equilibrium side slopes beyond the constructed condition is 350 acres for the PA.

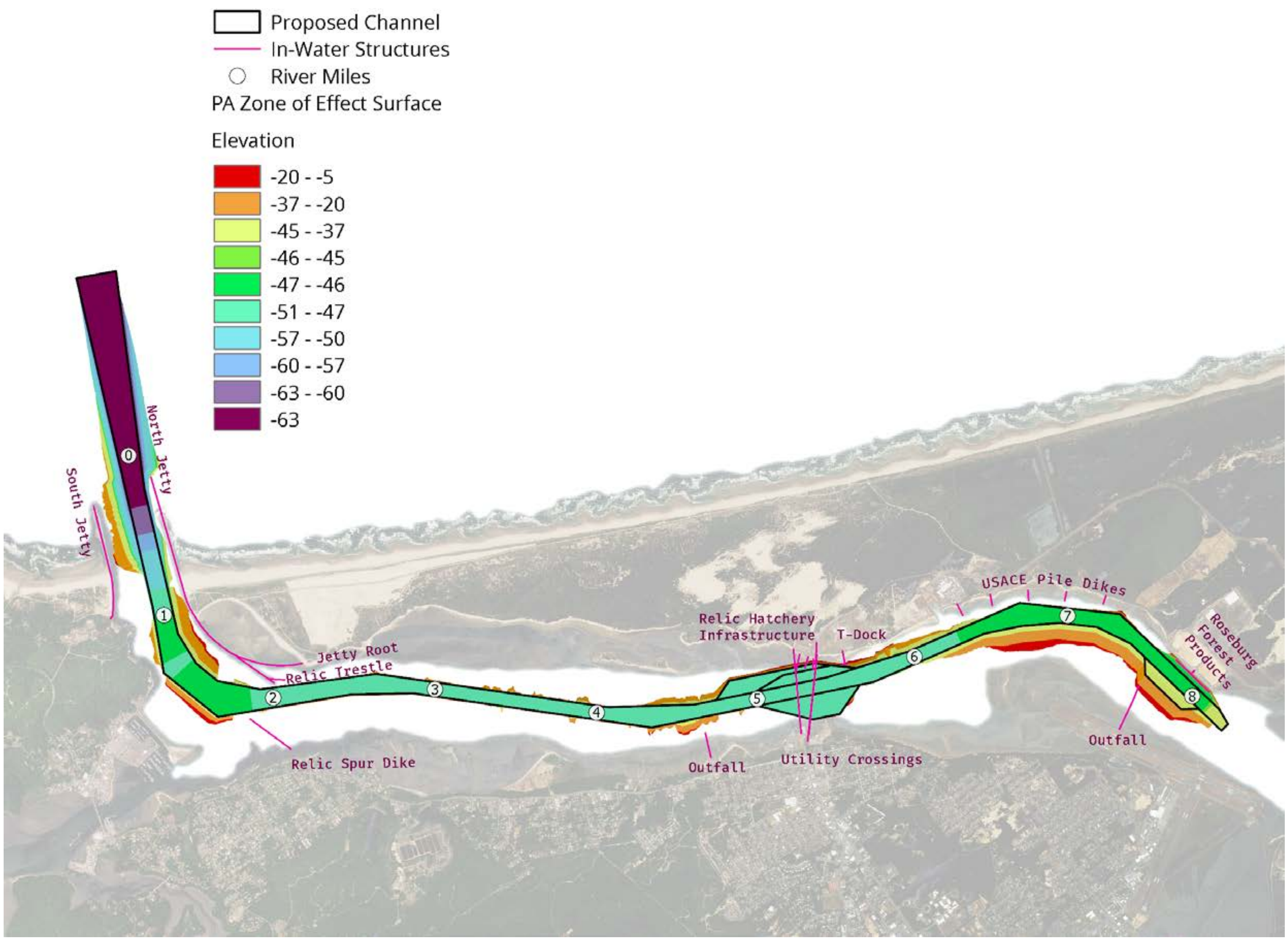


Figure 7-1

Zone of Equilibration (PA)

7.2 Geotechnical Global Stability of the Equilibrium Side Slopes Adjacent to Infrastructure

The approach used in the evaluations for geotechnical stability of the equilibrium side slopes adjacent to key infrastructure is presented in Section 3.3 of this report. The long-term effects of the proposed channel modifications on the global stability of select infrastructure were evaluated for the following infrastructure: the North Jetty, the South Jetty, the relic trestle located downstream of approximately RM 2, the berth and access channel at approximately RM 4.95, the Arago dock at approximately RM 5.36, the North Bay Marine Industrial Site T-dock located at approximately RM 5.55, the pile dike at RM 6.8, and the Southwest Oregon Regional Airport.

The locations of infrastructure in relation to the slope stability sections used in the analyses are shown in Attachment A, Figures A-1 through A-7. A summary of the global factor of safety at each infrastructure location before and after the proposed channel modifications is provided in Table 7-1 below. The geometry and sediment properties used in the analyses are presented in Attachment A, Figures A-8 through A-18.

Table 7-1
Global Factor of Safety at Each Infrastructure Location before and after the Proposed Channel Modifications

Infrastructure	Location (River Mile)	Global Static Slope Stability Factor of Safety		
		Before Channel Modification	After Channel Modification and Future Side Slope Equilibration	Relative Change
North Jetty	RM 0.27	1.7	1.6	-0.1
	RM 0.38	1.7	1.6	-0.1
	RM 0.53	1.9	1.7	-0.2
South Jetty	RM 0.38	2.3	2.3	0
	RM 0.49	2.4	2.4	0
Relic Trestle	RM 2.05	2.8	2.8	0
Empire Terminal	RM 5.36	1.5	1.5	0
T-Dock	RM 5.55	N/A[1]	3.8	N/A[1]
Pile Dike	RM 6.80	1.5	1.5	0
Regional Airport	RM 8.02	N/A[1]	2.2	N/A[1]

Notes:

1. *Comparison is not provided as pre-channel modification does not have significant slope heights/angles at this cross section.*

Overall, the results of the analyses indicate no changes or minor changes in the global stability of the long-term future equilibrium side slopes adjacent these infrastructure locations. In general, the change in global stability is a function of the proximity of the channel modifications to the infrastructure. In most cases, this distance was large enough to have minimal effect on the infrastructure, with the exceptions of the North Jetty. The proposed channel is relatively close to the North Jetty, and the long-term equilibrium side slopes have a risk of undermining a portion of the existing jetty rock and resulting in an unacceptably low factor of safety; however, as described in other sections of this report, a rock apron has been proposed at this location to mitigate this risk. The global slope stability analyses considered the proposed rock apron at the North Jetty. The post-dredging analyses including the rock apron resulted in a small decrease in the factor of safety at the North Jetty. It should be emphasized that the analyses only considered existing conditions before any channel modifications or equilibration and conditions after channel modification and long-term slope equilibration. Interim conditions were not included in the analyses, and periods of localized instability should be expected at the location of the rock aprons as the channel side slopes equilibrate and the rock aprons settle into their assumed long-term configuration.

It should be noted that the North Jetty has experienced ongoing deterioration at the head of the jetty, which is evident in the historical topography and bathymetry (USACE, 2012). Although an analysis of the jetty deterioration is outside the scope of this work, it may have occurred due to wave loading or other factors. In review of this work, it has been noted that the existing mudline is at or below the lowest historical elevation of jetty rock provided to the project team. In the areas where the falling apron has been proposed, the falling apron will also reduce the risk of scour undermining the jetties.

7.3 North Jetty

The majority of the North Jetty is not at risk of undercutting from side slope equilibration. From channel Station 0+33+00 through 0+46+00 (North Jetty Station 56+50 through 70+50), the minimum distance between the daylight and the North Jetty toe is 60 ft. Moreover, currents are expected to decrease in speed (Sub-appendix 4 – *Offshore and Entrance Dynamics*), alleviating potential for undermining the North Jetty. Similarly, upstream of Channel Station 1+03+00 (North Jetty Station 48+00), the minimum distance from the daylight point to the toe of the jetty is 125'. Current velocities are also expected to decrease in speed in this area (Sub-appendix 4 – *Offshore and Entrance Dynamics*), alleviating potential for undermining the North Jetty. Cross sections along the North Jetty, at 100 ft intervals, be seen in Sub-appendix 13 (*Cross Sections)*.

From North Jetty Station 70+50 through 86+50 and North Jetty Station 48+00 through 58+00, the future equilibrium side slopes daylight within 50' of the jetty toe. Because of the critical nature of this structure, and the catastrophic implications of failure, the entire range of side slopes (i.e., the median measured slopes) are used to quantify the daylight point and the required protection. Although the actual side slopes are not expected to achieve these angles, a rock apron has been designed to protect against potential undermining of the North Jetty. The threat to the North Jetty varies along the length of the structure, based on the distance between the channel and the structure, the proposed depth of the channel bottom, and the expected equilibrium channel side slope angle.

The depth for which slope protection is necessary is driven by the amount that the equilibrium side slopes daylight beneath the North Jetty. The required slope protection depths are summarized in Table 7-2. It should be noted that the depths listed in Table 7-2 represent the maximum depth within the range of stations (i.e., from North Jetty Station 77+50 – 86+50, the required toe protection under the PA is 14 ft or less).

Table 7-2
Daylight depth beneath the North Jetty toe

North Jetty Station Range	Required Toe Protection Depth PA (ft)
77+50 – 86+50	14
70+50 – 77+50	9
48+00 – 70+50	4

Under the PA, toe protection will be designed for a depth of 15 ft from North Jetty Station 77+50 through 86+50 and for a depth of 10 ft from North Jetty Station 70+50 through 77+50 and from North Jetty Station 48+00 through 58+00. These design depths are greater than the undercut depth in Table 7-2 to be conservative. The design of toe protection is detailed in Section 6.10 of the *Engineering Appendix*.

7.4 North Jetty – Rail Spur

The rail spur is the portion of the North Jetty that extends from the main structure towards the channel; it is located at approximately RM 1.9 through 2.1. It is closest to the channel at RM 2.1, where it is approximately 200 ft away from the channel. Over time, the presence of this structure has resulted in significant scour, with a permanent scour hole surrounding the structure. This scour hole ranges from 52 to 63 ft deep (Figure 4-25), or deeper than the proposed dredge cut. Therefore, side slope equilibration is not expected to affect this portion of the North Jetty. Detailed cross sections at the rail spur are shown in Sub-Appendix 13 (*Cross Sections)*.

7.5 South Jetty

The South Jetty is sufficiently far from the proposed channel modifications that side slope equilibration is not expected to undercut the structure. Near the jetty head, the equilibrated channel is sufficiently far from the structure that no undercutting is likely to occur, as shown in Figure 4-10. At other locations, the sides slopes are expected to intersect the seabed at a shallower elevation than the bottom of the structure, as shown in Figure 4-18. By comparison, the base of the structure is located at approximately -25 ft MLLW, based on historic bathymetry surveys (USACE 1924), while the slopes daylight near -20 ft MLLW. Therefore, effects to this structure are not anticipated. The daylight of the future equilibrium side slopes relative to the South Jetty can be seen in sections 0+14+00 through 0+41+00 of Sub-Appendix 13 (*Cross Sections)*.

7.6 William T. Rossell

The wreck of the *William T. Rossell* is located between the Entrance Channel jetties, approximately 140 ft south of the existing FNC and 85 feet south of the PA Channel Limits. The bottom elevations in the area of the wreck are approximately -45 ft to -50 ft MLLW with an existing slope of approximately 22H:1V. The maximum elevation over the highest remaining superstructure is approximately -30 ft MLLW with the elevation of most of the visible wreck in the -35 ft to -45 ft MLLW range. Since its sinking in 1957, the majority of the wreck has become buried at depths of up to 15 ft to 20 ft. The bottom of the hull at the stern (furthest from the channel) is located at an elevation of approximately -60 ft MLLW and the bottom of the hull at the bow (closest to the channel) is at approximately -70 ft MLLW. The wreck has a break in the hull at approximate mid-ship and possibly a break in the hull forward closer to the bow.

As explained in Section 4.2.4 and in Sub-appendix 4 (*Offshore and Ocean Dynamics)*, the area in the vicinity of the *Rossell* wreck is highly depositional and experiences rapid shoaling following dredging events. The long-term equilibrated channel side slope is the resulting slope configuration after the effects of dredging no longer affect channel side slope morphology. In regions of high shoaling, such as the area around the wreck, where channel maintenance dredging occurs on an annual basis, there is no trend of material from the top of, or above, the dredge slope shifting down slope. Long-term slope equilibration is not anticipated to result in any significant bed lowering in the area adjacent to the wreck. Therefore, no adverse effects to wreck stability will result from long-term slope equilibration.

Cross-sections showing constructed condition and future equilibrium side slopes extending from the PA near the *Rossell* are presented in Sub-Appendix 13 (*Cross Sections)*, Stations 0+17+00 and 0+19+00. The PA constructed condition side slope of 4H:1V (including advanced maintenance dredging) will daylight 50 ft or more from the wreck. Therefore, no adverse effects to wreck stability will result from construction dredge activities. While the range of future equilibrium side slopes are shown undercutting the wreck, no significant bed lowering due to slope equilibration is expected in the area adjacent to the *Rossell* wreck. However, applying the conservative future equilibrium side slope, which assumes the median measured side slope of 22H:1V extending up and out from the dredge toe at the advance maintenance dredge depth, shows the potential for undercutting the vessel. Although this slope is not applicable for assessment of potential effects on the wreck, the amount of undercutting represented by the future equilibrium slopes would not represent sufficient erosion to mobilize the wreck on a 22H:1V slope to a point where it would have an effect on the channel or future dredging activities. This is illustrated in Figure 7-2.

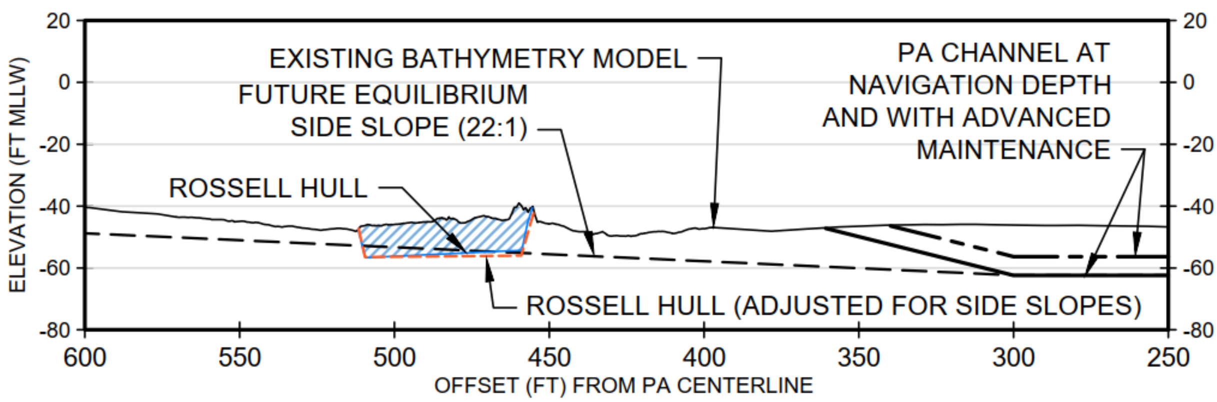


Figure 7-2
Side Slope Equilibration at the wreck of the *Rossell*

7.7 Private Docks

No significant effects are anticipated at the private docks at Cape Arago Dock and the Hollering Place. Detailed cross sections showing the future equilibrium side slopes in the vicinity of these docks can be seen in sections 5+10+56 through 5+22+32 of Sub-appendix 13 (*Cross Sections)*. While the daylight lines are close to the existing structures, the effective elevation change is minimal. The resultant mudline elevation at the outer pile due to side slope equilibration (-15 ft MLLW) is still shallower than the design slip depth of -22ft MLLW. Therefore, no adverse impacts to the structure are anticipated. Other docks are not located near the side slope equilibration zone of equilibration.

Detailed cross sections showing the disused aquaculture (salmon capture/release) facility can be seen in sections 5+12+97 through 5+19+53 of Sub-Appendix 13 (*Cross Sections)*. This facility is being demolished as a part of the container terminal construction.

7.8 Pile Dikes

The results of the equilibrium side slope analysis indicated that channel modification, combined with future side slope equilibration, is unlikely to affect the pile dikes. Figure 4-57 shows that the channel is naturally deep in this area, and that only limited dredging is required. Because the dredge cut is expected to be shallow, equilibration is expected to be limited. Detailed cross sections at the pile dikes can be seen in Sub-Appendix 13 (*Cross Sections)*. The specific sections corresponding to each pile dike are noted below:

- Pile Dike 6.4: 6+19+41 to 6+19+91
- Pile Dike 6.6: 6+29+14 to 6+29+59
- Pile Dike 6.8: 6+40+15 to 6+40+62
- Pile Dike 7.0: 6+51+06 to 6+51+56
- Pile Dike 7.3: 7+10+16 to 7+10+56

The limited equilibration that may occur has been shown to terminate more than 50 ft from the rock apron surrounding the pile dikes. Therefore, no effects are expected.

In the event that future side slope equilibration was to cause any additional erosion in the vicinity of the structures, this section investigates the present state of the rock apron structure, and ultimately concludes that the additional mitigation beyond the existing rock apron is not necessary to protect the pile dikes from side slope equilibration. Pile Dike CB-6.8 is shown because it is located closest to the channel.

7.8.1 Pile Dike History

The pile dikes were constructed in 1957. At the time of construction, the base of the structures was armored with rock to enhance the long-term stability of the structures. The immediate vicinity of the structures had been historically eroding, justifying the placement of such protection.

The as-built plans indicate two armor stone protection structures were constructed (USACE 1957). A rock revetment, whose thickness was equal to 1/3 of the depth, was placed within 7.5 ft of each side of the structure. In addition, a rock apron was placed around the channel-side pile. This apron was placed in a circle with a radius of 50 ft and a thickness of 3 ft. The armor stone distribution can be seen in Figure 7-3.

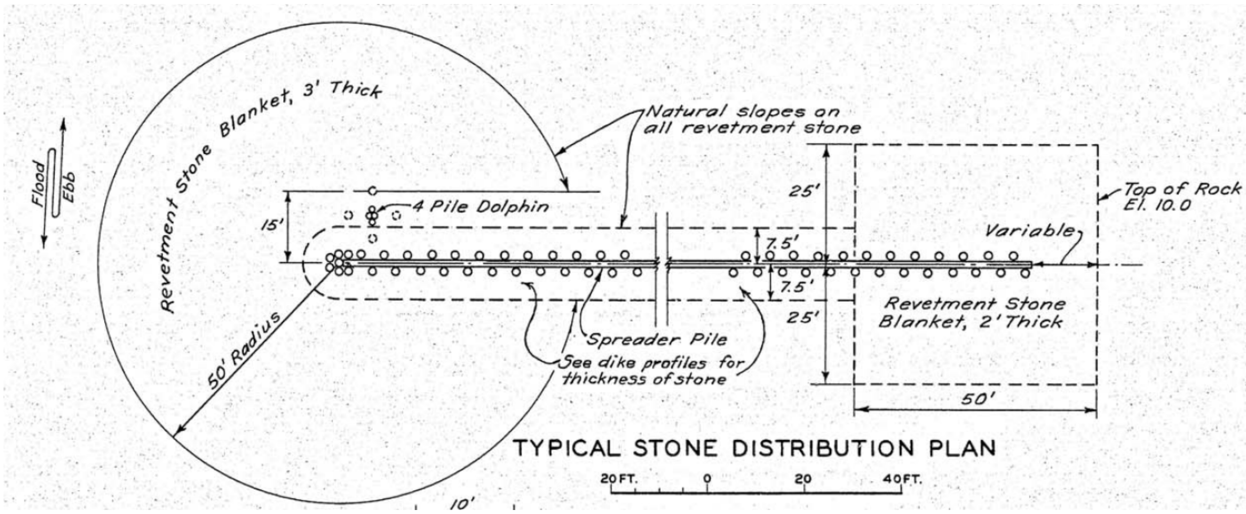


Figure 7-3
Armor Stone Distribution during Pile Dike Construction

7.8.2 Present State of the Rock Apron

Side scan sonar imagery allows us to photograph the floor of the estuary; At CB-6.8, this provides valuable insight to the functionality and performance of the rock apron. Underwater imagery that depicts the rock apron can be seen in Figure 7-4. As this figure shows, the revetment has remained in its original location, and is still within close proximity of the structure. The rock apron has tended to shift. Presently, rock appears to extend down the slope adjacent to the channel, indicating movement of the original apron. However, a significant portion of the original apron is present just offshore of Lateral Marker 19 (i.e., the original location of the tip of the pile).

A cross-section can also provide valuable information about the state of the rock apron; it can be seen in Figure 4-57. This figure shows the bathymetry adjacent to the channel sloping sharply downwards, at an approximately 1.8:1 to 2.3:1 slope at the different pile dikes. As shown in Figure 7-4, this sloped area is likely comprised of rock. This portion of the rock apron has settled in response to the erosion near the channel. Therefore, the rock apron has performed successfully.

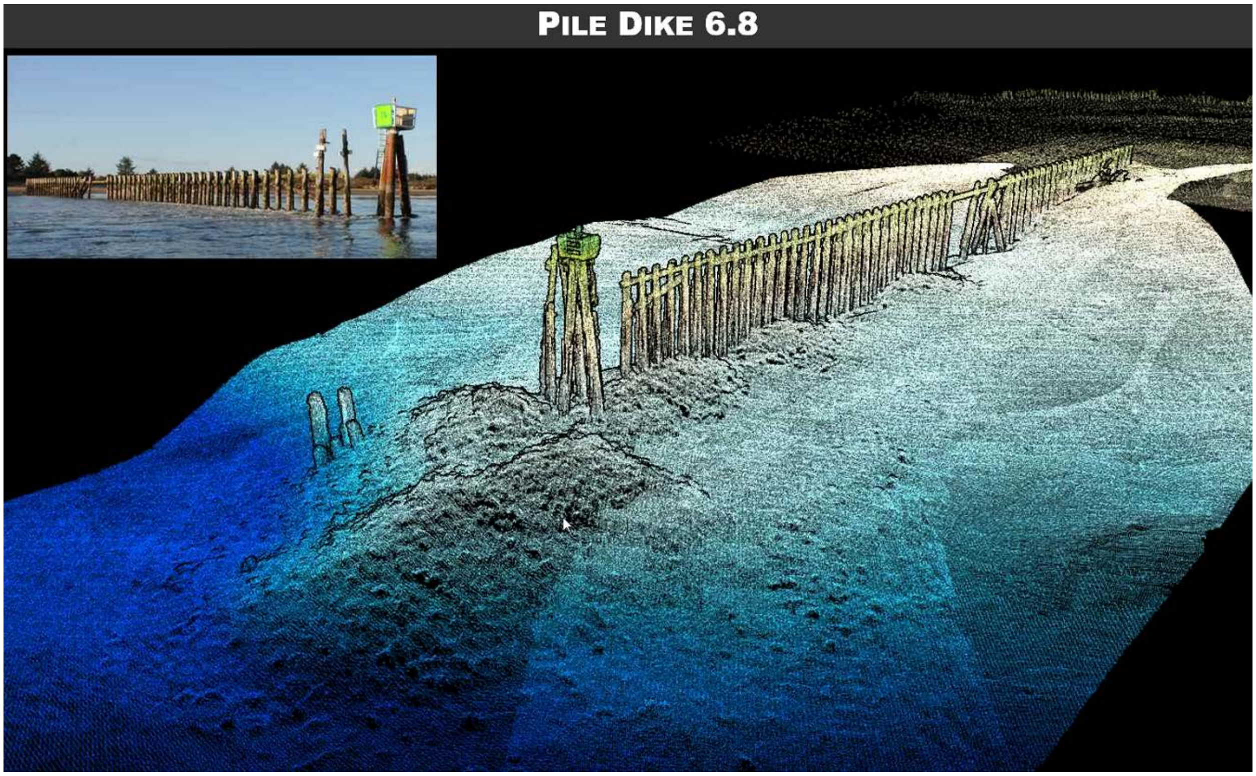


Figure 7-4
Side Scan Sonar Imagery at CB-6.8

7.8.3 Rock Apron Remaining

While a portion of the rock apron has begun to settle, a significant volume of rock still exists to protect against future scour. The portion of the rock apron still present at CB-6.8 was estimated using high-quality multi-beam survey data. The survey was digitized, and the elevation of the sand-rock interface offshore of the structure (the entire border of the detectable rock apron limits) was used to generate a surface. This surface estimated the top of the sand/bottom of the rock. Comparing that surface to the multi-beam bathymetry (which includes rock and sand), yielded a volume of 907 cy. By comparison, the placement volume of the rock apron was 873 cy. Because this surface was able to recreate the placement volume to within 4%, it was assumed to provide a reasonable estimate of the remaining rock apron.

The next step was to estimate the portion of the rock apron that was still intact. This was accomplished by determining the portion of the apron that was still at least 3 ft thick (i.e., the original thickness of the rock apron). The portion of the rock apron greater than 3 ft thick can be seen in Figure 7-5. This analysis indicates that 3,445 square feet (44% of the original apron area) still meet or exceed the original placement thickness. This area is approximately 80 ft wide (parallel to the channel alignment) by 40 ft long. This quantity of rock would be available to protect against additional movement of the adjacent slope.

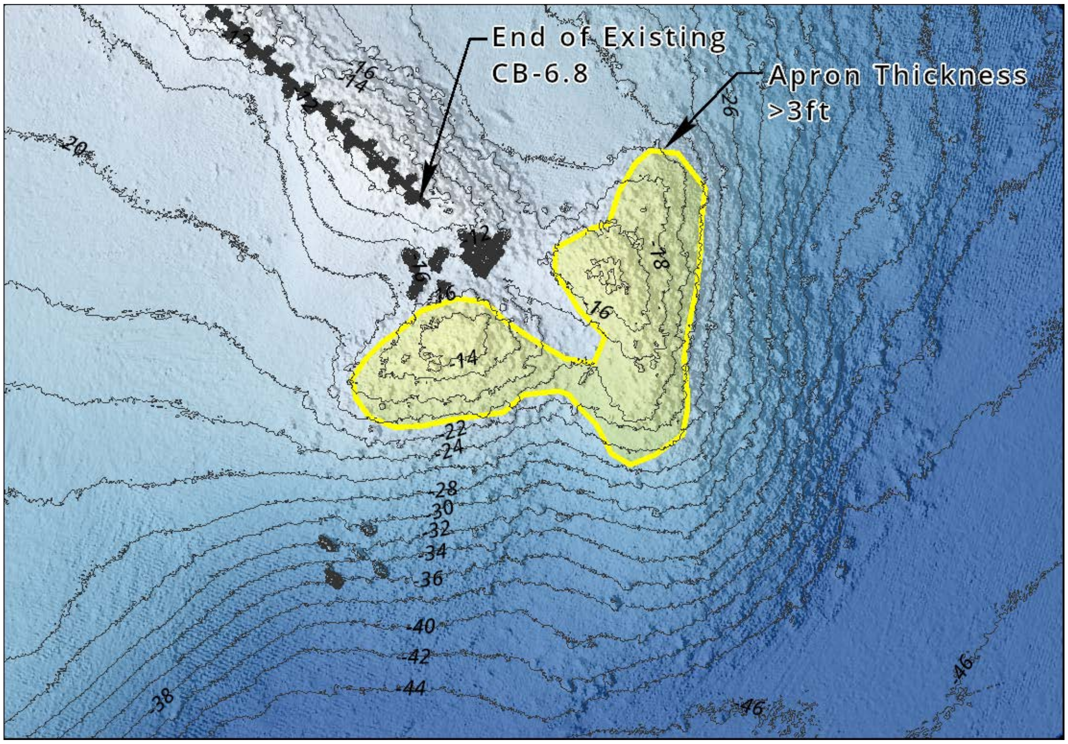


Figure 7-5
Intact Portion of the Original Rock Apron

7.8.4 Future Performance of the Rock Apron

As shown in Figure 4-57, the apron has settled an estimated 5 (C-6.4) to 21 (CB-6.8) ft. Assuming that this represents 60% of the total settling capacity at CB-6.8, the existing rock may have the capacity to settle an additional 14 ft before losing the ability to protect the pile dike structure. The side slope equilibration estimates presented herein do not predict that any of the rock aprons will be further undercut.

7.9 Southwest Oregon Regional Airport

The airport is sufficiently far from the proposed channel modifications. Based on the future equilibrium side slope, equilibration is not expected to undercut the structure. As shown in Figure 4-54 and Figure 4-60, the side slopes are expected to daylight to the present bathymetry well away from the location of the airport. Therefore, effects to this structure are not anticipated. The airport would be present on the far left of section 7+0+00 through the end of Sub-Appendix 13 (*Cross Sections)*. However, structure is a significant distance from the daylight point of the 20:1 slopes it is outside the cross section limits.

7.10 Outfalls

The Empire Outfall can be seen in Sections 4+33+84 through 4+34+35 of Sub-appendix 13 (*Cross Sections)*. The future equilibrium side slopes daylight 150 ft from the outfall. Therefore, no effects from equilibration are expected.

The North Spit Outfall is located at Sections 5+12+38 through 5+12+88 of Sub-appendix 13 (*Cross Sections)*. This outfall is abandoned and will be removed as a part of the container terminal development.

The Airport Outfall can be seen in Sections 7+31+54 through 7+35+60. The daylight of the 20:1 slope encroaches on the outfall. However, when surveyed in December of 2016, the outfall was broken and in three pieces. The outer section of outfall pipe is broken, disconnected, and no effluent flows through the outfall pipe that now lays here. Any encroachment on this outer segment of pipe will not affect how the outfall currently functions. Furthermore, the 20:1 slope is based on the observed median slope for the reach. However, this bank is on the depositional side of the channel. Due to the deeper cut and the depositional environment, the side slope is unlikely to establish itself at a 20:1 slope.

8. RISK MANAGEMENT PLAN

Results of the investigations described in this Section 204(f)/408 Report, in the opinion of the OIPCB, show that all project effects on infrastructure and the natural environment have been managed and are minor and manageable. The Corps of Engineers, through their Section 408 and 404 reviews, will make the Federal determination whether the Proposed Alteration is environmentally acceptable and consistent with Federal policy. As is the case with the implementation of any navigation improvement project in such a dynamic physical environment and within an important and ecologically valuable estuary, there will be inherent residual risk and uncertainty associated project implementation. As such, Risk management will be a critical element of the project.

This sub-appendix presents a geotechnical evaluation of the initial dredge cuts and an assessment of the future equilibrium side slopes that may result from the Coos Bay Channel Modification Project, including the long-term geotechnical stability of the side-slopes adjacent to infrastructure. After the completion of capital dredging, side slopes will continue to evolve until they reach a stable slope angle, after which sedimentation patterns will reach a future equilibrium state. Estimating the expected side slopes is critical for the purpose of predicting the total dredge volumes that may result from channel equilibration process and for the purpose of estimating potential effects to federal infrastructure and other resources. This analysis recognizes the inherent uncertainty in predicting the future equilibrium side slope, and therefore predicts a range of side slope outcomes. Throughout the development of the Section 204(f)/408 Report, potential areas of residual risk regarding the potential for impacts from side slope equilibration area have been identified. While these potential impacts will be further evaluated in the EIS process, preliminary elements of risk identified as warranting quantitative risk management plan are summarized in Table 8-1.

Table 8-1
Risk Management Elements Related to Channel Side Slope Analysis

Issue or Concern	Primary Monitoring	Monitoring Tools	Frequency and Duration of Monitoring	Trigger(s) for Action	Possible Response Actions
North and South Jetty Stability	Bathymetric surveys	Bathymetric surveys to establish baseline Existing variability	Annually – 5-year period post construction. Periodic following major storm events.	Side slope equilibration and/or erosion beyond predicted limits and / or in close proximity to jetty structure	Temporarily suspend dredging operations; Add or enhance rock apron
Other Infrastructure Stability	Bathymetric surveys	Bathymetric surveys to establish baseline Existing variability	Annually – 5-year period post construction. Periodic following major storm events.	Side slope equilibration and/or erosion beyond predicted limits and / or in close proximity to infrastructure and existing shoreline	Temporarily suspend dredging operations; Add or enhance rock armoring, or provide other protective measures

The Risk Management Plan will be developed based on USACE Risk Management guidance.

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Attachment A: Global Slope Stability Analysis Locations And Output Files

List of Figures – Attachment A

ATTACHMENT B: STATIC LIQUEFACTION SCREENING AND PROBABILISTIC ANALYSES

List of Figures – Attachment B

List of Sub-Attachments – Sub-Attachment B

B-1. Static Liquefaction Screening Analyses

The risk of static liquefaction for the initial dredge cuts was assessed using a screening analysis developed by Stoutjesdijk et al. (1994) and method developed by Raaijmakers (2005) that compares the geometric and sediment-density characteristics of the proposed initial dredge cuts to the likelihood of static liquefaction. The Raaijmakers (2005) and Stoutjesdijk et al. (1994) charts are included as Figures B1 and B2, respectively. For the assessment using the Raaijmakers chart, the proposed initial dredge cut slope angle (x-axis) and the assumed relative density of the sediment (y-axis) were plotted and used in conjunction with the dredge cut slope height for each reach. If the point plotted above the line representing the selected dredge cut slope height, the configuration was considered unstable and a more conservative initial dredge cut slope was analyzed. If the point plotted below the line representing the selected dredge cut slope height, the configuration was considered stable. Marginal stability was used to define a scenario where conditions plot within the stable region of the chart under the specified configuration; however, the addition of less than approximately 3 meters of additional dredge cut slope height plotted within unstable region of the chart. The marginal situation was considered unstable and a more conservative initial dredge cut slope angle was analyzed.

For the assessment using the Stoutjesdijk et al. chart, the initial dredge cut slope height (x-axis) and the proposed initial dredge cut slope angle (y-axis) were plotted and used in conjunction with the assumed relative density of the sediment within the reach. If the point plotted below the line representing the assumed relative density, the configuration was considered unstable and a more conservative initial dredge cut slope was analyzed. If the point plotted above the line representing the assumed relative density of the sediment, the configuration was considered stable. As in the Raaijmakers assessment, marginal stability was used to define a scenario where conditions plot within the stable region of the chart under specified configuration; however, the addition of less than approximately 3 meters of additional dredge cut slope height plotted within unstable region of the chart. The marginal situation was considered unstable and a more conservative initial dredge cut slope angle was analyzed.

B-2. Static Liquefaction Probabilistic Analysis

The risk of static liquefaction for the initial dredge cuts was also assessed using a physics-based probabilistic analysis developed by Van den Ham et al. (2014) to quantify the risk of static liquefaction. An example calculation using the Van den Ham assessment is attached. The example calculation represents the assessment completed for the left bank of Reach 3. The parameters used in the example calculation are summarized in Table B-1.

Table B-1
Summary Calculation of Static Liquefaction for Reach 3

Reach	Location	RM	Station	Conditions					Proposed Initial Cut						
									Side Slope			Geotechnical Stability			
				Blows, blows/ft	Unit Weight, pcf	Soil Type	phi, degrees	Soil Relative Density	Inclination (H:V)	Height, ft	Height, m	(Long-Term Static)*	Probabilistic Static Liq., per km per yr	Raaijmakers Static Liq.	Stoutjesdijk Static Liq.
Reach 3	Left Bank	0.49	26+00	3	110	Very Loose	26	0.3	3:1	28	9	1.5	0.03	Marginal	Marginal
									4:1			2.0	0.005	Stable	Stable

Notes: Marginal used to define a scenario where conditions plot within the stable region under specified configuration; however, the addition of less than approximately 3 meters of additional side slope height plots within unstable region.

B-3. Static Liquefaction Analyses Sensitivity

An example evaluation of the sensitivity of the Raaijmakers, Stoutjesdijk et al., and Van den Ham et al. assessments to variations in the assumed relative density of the sediment and changes in slope angles is summarized in Table B-2. In general, the comparison table shows agreement with all three methodologies and the trends shown in the charts for the Raaijmakers (2005) and Stoutjesdijk et al. (1994).

Table B-2
Reach 3 Sensitivity Analyses

Reach	Location	RM	Station	Conditions					Proposed Initial Cut						
									Side Slope			Geotechnical Stability			
				Blows, blows/ft	Unit Weight, pcf	Soil Type	phi, degrees	Soil Relative Density	Inclination (H:V)	Height, ft	Height, m	(Long-Term Static)*	Probabilistic Static Liq., per km per yr	Raaijmakers Static Liq.	Stoutjesdijk Static Liq.
Reach 3	Left Bank	0.49	26+00	1	110	Very Loose	20	0.18	3:1	28	9	1.0	0.1	Unstable	Unstable
									4:1			1.4	0.02	Unstable	Marginal
				2			23	0.26	3:1			1.3	0.05	Unstable	Unstable
									4:1			1.7	0.009	Stable	Stable
				3			26	0.32	3:1			1.5	0.02	Marginal	Marginal
									4:1			2.0	0.005	Stable	Stable
				4			28	0.37	3:1			1.6	0.014	Stable	Stable
									4:1			2.2	0.003	Stable	Stable

Notes: Marginal used to define a scenario where conditions plot within the stable region under specified configuration; however, the addition of less than approximately 3 meters of additional side slope height plots within unstable region.

SUB-ATTACHMENT B-1: EXAMPLE PROBABILISTIC STATIC LIQUEFACTION CALCULATION, VAN DEN HAM ET AL. (2014)