Appendix C.
90% Engineer's Opinions of Probable Construction Cost

19-Jun-26

Sitka Sedge Tidal Wetland Restoration - 90% Design - Estimated Construction Cost

Construction Category		*Cost*
Mobilization/Demobilization	$	886,000
Access, Traffic Control, Water Mangement, Survey	$	847,000
Demolition, Clearing, and Grubbing	$	205,000
Tidal Channel and Ditch Fill Earthwork	$	584,000
Setback Dike Embankment, Imported Fill, Access Rd	$	2,555,000
Setback Dike Drainage Structure, Tide Gate, MTR	$	1,841,000
Backup Drainage Culverts and Flap Gates	$	113,000
Seeding and Revegetation	$	210,000
Wood Habitat Structures	$	100,000
Prefabricated Pedestrian Bridge and Foundations	$	2,407,000
15% Contingency for 90% Design Stage	$	1,462,000
Sitka Sedge TWR Total Construction Cost	$	11,210,000
Construction Engineering and Permit Compliance Services (20%)	$	2,242,038

Reneke Creek Bridge and Stream Realignment - 90% Design - Estimated Construction Cost

Construction Category		*Cost*
Mobilization/Demobilization	$	276,927
Access, Traffic Control, Water Management, Survey	$	314,695
Demolition, Clearing, and Grubbing	$	83,235
Roadway Improvements	$	403,119
Roadway Bridge and Pile-Supported Foundation	$	755,430
Stream Channel and Ditch Fill Earthwork	$	550,449
Wood Habitat Structures	$	127,500
Seeding and Revegetation	$	534,844
15% Contingency for 90% Design Stage	$	456,930
Reneke Creek/Sandlake Road Total Construction Cost	$	3,503,129
Construction Engineering and Permit Compliance Services (20%)	$	700,626

Total Estimated Construction Cost for SSTW and Reneke Creek	$	14,713,129
Total Recommended Funding Amount for Construction and Consultant Services:	$	17,655,793

SITKA SEDGE TIDAL WETLAND RESTORATION PROJECT — 19-Jun-20

90% DESIGN - PRELIMINARY ENGINEER'S OPINION OF PROBABLE CONSTRUCTION COST

Dike Breach, Pedestrian Bridge, Setback Dike, Interior Work - Does not include Sandlake Rd Culvert

Item #	Spec Section	Item Description	Unit	Quantity	Unit Price	Item Cost
1	00210	Mobilization	LS	1	$ 669,181	$ 669,181
2	00221	Temporary Work Zone Traffic Control, Complete	LS	1	$ 50,000	$ 50,000
3	00345	Temporary Water Management / Work Area Isolation	LS	1	$ 300,000	$ 300,000
4	00345	Temporary Drainage Culverts	LS	1	$ 80,000	$ 80,000
5	00353	Temporary Work Access and Containment - general (not ped bridge)	LS	1	$ 150,000	$ 150,000
6	00280	Erosion and Sediment Control	LS	1	$ 130,000	$ 130,000
7	00300	Concrete Washout Facility	LS	1	$ 12,000	$ 12,000
8	00280	Pollution Control Plan	LS	1	$ 5,000	$ 5,000
9	00325	Construction Survey Work - General	LS	1	$ 80,000	$ 80,000
10	00310	Remove Existing Box Culvert/Tide Gate and Reconstruct Embankment	LS	1	$ 25,000	$ 25,000
11	00320	Clearing and Grubbing	LS	1	$ 180,000	$ 180,000
12	00330	Dike Breach Channel Excavation	CY	6,156	$ 22	$ 135,432
13	00330	Tide Gate Channel Excavation	CY	1,006	$ 22	$ 22,132
14	00330	Central Channel Excavation	CY	5,366	$ 22	$ 118,052
15	00330	Secondary Tidal Channel Excavation (across site)	CY	7,931	$ 22	$ 174,488
16	00330	Ditch Fill Embankment	CY	8,946	$ 10	$ 89,461
17	00330	Excess fill embankment (unsuitable overbuild and mounds)	CY	3,646	$ 10	$ 36,461
18	00330	Select Native Topsoil Embankment (Breach Channel)	CY	222	$ 20	$ 4,444
19	00330	Select Native Topsoil Embankment (Setback Dike)	CY	2,802	$ 20	$ 56,041
20	00330	Setback Dike Embankment (Zone B)	CY	24,340	$ 15	$ 365,099
21	00330	Import Contractor-Furnished Fill (Setback Dike)	CY	45,877	$ 40	$ 1,835,084
22	00350	Subgrade Reinforcement Geogrid (Setback Dike 4 layers)	SY	34,008	$ 5	$ 182,044
23	00350	Subgrade Geotextile (Access Rd)	SY	2,552	$ 4	$ 10,068
24	00641	1-1/2-0" Aggregate Base (Access Road on Setback Dike)	TON	1,166	$ 84	$ 97,915
25	00445	36 Inch Concrete Culvert Pipe (Backup)	FOOT	160	$ 285	$ 45,600
26	1062	36 Inch Flap Gate	EA	2	$ 12,000	$ 24,000
27	00540	Structural Concrete Class 4000 - Culvert Headwalls	CY	18	$ 2,000	$ 35,556
28	00390	Loose Riprap Class 100	CY	40	$ 60	$ 2,400
29	Special	Backup Culvert Trash Racks/Beaver Exclusion	EA	2	$ 2,500	$ 5,000
30	00350	Riprap Geotextile, Type 2	SY	387	$ 4	$ 1,548
31	00330	Tide Gate Preloading Mound Embankment	CY	5,067	$ 10	$ 50,667
32	00905	Settlement Monitoring Plates	LS	1	$ 31,000	$ 31,000
33	00350	Subgrade Reinforcement Geogrid (Tide Gate Structure Foundation)	SY	948	$ 8	$ 7,591
34	00350	Separation Geotextile (Tide Gate Structure Foundation)	SY	474	$ 4	$ 1,896
35	00390	Filter Blanket	SY	387	$ 30	$ 11,600
36	00390	Loose Riprap Class 700 (modified - For Tide Gate Structure)	CY	387	$ 80	$ 30,987
37	00641	French Drain Exc and Filter Backfill (based on geotech during construction)	CY	859	$ 65	$ 57,778
38	00810	Sheet Pile Cutoff Walls	LS	1	$ 301,860	$ 301,860
39	00530	Reinforcement, Grade 60	LBS	388,573	$ 0.39	$ 111,373
40	00540	General Structural Concrete, Class 4000 (Tide Gate Structure)	CY	237	$ 4,000	$ 947,173
41	00641	Aggregate Base (Tide Gate Structure Foundation)	TON	1,024	$ 140	$ 143,361
42	00641	4 Inch Drain Pipe	LF	150	$ 13	$ 1,950
43	01550	Cl.-4ft Chainlink Fence	LF	148	$ 60	$ 8,880
44	01550	4 Foot x 48 Inch Vinyl Coated Chain Link Single Gates	EA	3	$ 1,600	$ 4,800
45	01095	NMM - NMTR3 - Mussel Tidal Regulator (fabricate and deliver)	EA	1	$ 65,000	$ 65,000
46	01095	NSRG10-flrc right hand Tide Gate Assembly	EA	1	$ 80,000	$ 80,000
47	01095	NMM - Tide Gate and MTR Installation	LS	1	$ 34,000	$ 34,000
48	01095	General Contractor - Assist NMM with Tide Gate/MTR Install	LS	1	$ 20,000	$ 20,000
49	01095	Beaver Exclusion Fencing and Flow Device	LS	1	$ 40,000	$ 40,000
50	01030	Temporary Seeding - Native EC Mix - Disturbed Areas	AC	8	$ 5,500	$ 44,000
51	01030	Permanent Seeding - Native EC Mixes (Setback, Staging, Dike Slopes)	AC	5.1	$ 5,500	$ 28,179
52	01040	Salvage, Store, and Transplant plants in impacted areas	LS	1	$ 15,000	$ 15,000
53	01040	Setback Dike Shrub Planting	AC	2.1	$ 50,000	$ 103,125
54	01040	Tidal Marsh Planting	AC	0.65	$ 30,000	$ 19,500
55	01091	WHS - Bank Structure	EA	12	$ 6,500	$ 78,000
56	01091	WHS - Single Keyed Log w Root Wad	EA	10	$ 1,200	$ 12,000
57	01091	WHS - Install Single Log on Mainlplain or Channel	EA	10	$ 1,000	$ 10,000
58	00582	Ped Bridge - Furnish Prefabricated Truss (140' span)	EA	1	$ 663,000	$ 663,000
59	00582	Ped Bridge - Assemble and Install Prefabricated Truss (140' span)	EA	1	$ 92,150	$ 92,150
60	00582	Ped Bridge - Mobilize Crane/Derik	LS	1	$ 150,000	$ 150,000
61	Special	Ped Bridge - Temporary Access Improvements and Staging	LS	1	$ 720,000	$ 720,000
62	00540	Ped Bridge - General Structural Concrete, Class 4000 (Abutments)	CY	35	$ 2,000	$ 70,000
63	00340	Ped Bridge - Glass Fiber Reinforced Polymer Reinforcement	FT	5100	$ 10	$ 51,000
64	00320	Ped Bridge - Furnish Pile Driving Equipment	LS	1	$ 75,000	$ 75,000
65	00320	Ped Bridge - Furnish (FP 20 x0.50) Steel Piles	FOOT	420	$ 165	$ 70,200
66	00320	Ped Bridge - Prestoring Piles	FOOT	420	$ 800	$ 315,000
67	00320	Ped Bridge - Drive Piles	EA	8	$ 3,500	$ 21,000
68	00320	Ped Bridge - Steel Pile Protective Coating	LS	1	$ 15,000	$ 15,000
69	00330	Ped Bridge - Structure Excavation	CY	40	$ 75	$ 3,000
70	00641	Ped Bridge - Granular Structure Backfill	CY	30	$ 250	$ 7,500
71	01088	Ped Bridge - Metal Handrail, 3 Rails	FOOT	48	$ 150	$ 7,200
72	00330	Trail Approach Embankment	CY	222	$ 65	$ 14,400
73	00641	Trail Resurfacing - 1/8" minus	TON	170	$ 80	$ 13,594
74	00350	Riprap Geotextile, Type 2	SY	295	$ 4	$ 1,179
75	00390	Filter Blanket	SY	295	$ 30	$ 8,841
76	00390	Loose Riprap Class 700	CY	387	$ 80	$ 30,987

Construction Subtotal	$ 8,747,992
15% Contingency	$ 1,462,199
GRAND TOTAL CONSTRUCTION COST	$ 11,210,191
Construction Engineering and Permit Compliance Services (20%)	$ 2,242,038

NOTES:

1. This preliminary opinion of probable construction cost is based on the Draft 90% Design which will be further refined as the project advances towards implementation. Geotechnical investigation in the marsh location of the setback dike and tide gate structure was deferred to the start of construction, with earthwork volumes and subgrade stabilization measures based on subsurface data from nearby borings on site.

2. The above cost opinion assumes 2021/2022 dollars (projected construction timing) and does not include future escalation or financing costs.

3. Any estimate of construction cost prepared by RSH represents its judgement as a design professional and is supplied for general guidance to the Client. Since RSH has no control over the cost of labor and material, competitive bidding, or market conditions, RSH does not guarantee the accuracy of such estimates as compared to Contractor bids or actual costs to the Client.

RENEKE CREEK BRIDGE AND STREAM RESTORATION

19-Jun-25

90% DESIGN - PRELIMINARY ENGINEER'S OPINION OF PROBABLE CONSTRUCTION COST

Culvert Replacement, Bridge Installation, Roadway Improvements, Stream and Habitat Improvements

Item #	Spec Section	Item Description	Unit	Quantity	Unit Price	Item Cost
TEMPORARY FEATURES AND APPURTENANCES						
1	00210	MOBILIZATION	LS	ALL	10%	$ 276,927
2	00221	TEMPORARY PROTECTION AND DIRECTION OF TRAFFIC	LS	1	$ 40,000	$ 40,000
3	00222	TEMPORARY SIGNS	SQFT	200	$ 30	$ 6,000
4	00224	TEMPORARY PLASTIC DRUMS	EACH	45	$ 75	$ 3,375
5	00225	TEMPORARY PAVEMENT BARS	SQFT	24	$ 30	$ 720
6	00226	TEMPORARY BARRIER	FOOT	650	$ 35	$ 22,750
7	00226	TEMPORARY IMPACT ATTENUATOR, NARROW SITE SYSTEM	EACH	2	$ 5,000	$ 10,000
8	00226	MOVING TEMPOARY IMPACT ATTENUATORS, NARROW SITE SYSTEM	EACH	2	$ 1,500	$ 3,000
9	00226	MINIMUM DEFLECTION BRIDGE TEMPORARY BARRIER	FOOT	90	$ 65	$ 5,850
10	00227	PORTABLE TRAFFIC SIGNAL	EACH	2	$ 15,000	$ 30,000
11	00250	TEMPORARY WATER MANAGEMENT/WORK AREA ISOLATION	LS	1	$ 70,000	$ 70,000
12	00253	TEMPORARY WORK ACCESS AND CONTAINMENT - RENEKE CREEK AREA	LS	1	$ 35,000	$ 35,000
13	00280	EROSION AND SEDIMENT CONTROL	LS	1	$ 33,000	$ 33,000
14	00290	POLLUTION CONTROL PLAN	LS	1	$ 5,000	$ 5,000
ROADWORK						
15	00305	CONSTRUCTION SURVEY WORK	LS	1	$ 50,000	$ 50,000
16	00310	REMOVAL OF STRUCTURES AND OBSTRUCTIONS	LS	1	$ 30,000	$ 30,000
17	00310	ASPHALT PAVEMENT SAW CUTTING	FOOT	647	$ 5	$ 3,235
18	00320	CLEARING AND GRUBBING	LS	1	$ 50,000	$ 50,000
19	00330	EMBANKMENT IN PLACE	CUYD	776	$ 65	$ 50,440
20	00350	SUBGRADE GEOTEXTILE	SQYD	2200	$ 5	$ 11,000
21	00390	FILTER BLANKET	SQYD	124	$ 30	$ 3,717
22	00390	LOOSE RIPRAP, CLASS 700	CUYD	124	$ 80	$ 9,911
23	00445	12 INCH CULVERT PIPE, 10 FT DEPTH	FOOT	12	$ 200	$ 2,400
BRIDGES						
24	00510	STRUCTURE EXCAVATION	CUYD	239	$ 75	$ 17,925
25	00510	GRANULAR STRUCTURE BACKFILL	CUYD	197	$ 150	$ 29,550
26	00520	FURNISH PILE DRIVING EQUIPMENT	LS	1	$ 50,000	$ 50,000
27	00520	FURNISH PP 16 X 141 STEEL PILES	FOOT	1,088	$ 150	$ 163,200
28	00520	DRIVE PP 16 X 0141 STEEL PILES	EACH	16	$ 2,000	$ 32,000
29	00520	HP 16 X 141 STEEL PILE SPLICES	EACH	5	$ 1,000	$ 5,000
30	00520	REINFORCED PILE TIPS	EACH	16	$ 500	$ 8,000
31	00530	REINFORCEMENT, GRADE 60	LB	6,950	$ 2.90	$ 20,155
32	00540	FOUNDATION CONCRETE, CLASS 4000	CUYD	47	$ 1,500	$ 70,500
33	00545	REINFORCED CONCRETE BRIDGE APPROACH SLABS	SQYD	145	$ 700	$ 101,500
34	00550	21 INCH PRECAST PRESTRESSED SLABS	FOOT	399	$ 900	$ 359,100
35	00587	THRIE BEAM RAIL	LS	1	$ 30,000	$ 30,000
BASES						
36	00641	AGGREGATE BASE	TON	1,280	$ 50	$ 64,000
WEARING SURFACES						
37	00744	LEVEL 2, 1/2 INCH ACP MIXTURE	TON	700	$ 150	$ 105,000
38	00749	EXTRA FOR ASPHALT APPROACHES	EACH	4	$ 1,000	$ 4,000
PERMANENT TRAFFIC SAFETY AND GUIDANCE DEVICES						
39	00810	GUARDRAIL, TYPE 3	FOOT	50	$ 90	$ 4,500
40	00810	GUARDRAIL TRANSITION	EACH	4	$ 3,600	$ 14,400
41	00810	GUARDRAIL TERMINALS, NON-FLARED	EACH	4	$ 3,600	$ 14,400
42	00860	LONGITUDINAL PAVEMENT MARKINGS - PAINT	FOOT	1,293	$ 3	$ 3,879
RIGHT OF WAY DEVELOPMENT AND CONTROL						
43	01030	TEMPORARY SEEDING- DISTURBED AREAS	AC	0.5	$ 5,500	$ 2,750
44	01030	PERMANENT SEEDING- all zones	AC	12.2	$ 6,000	$ 73,062
45	01040	REVEGTATION - UPLAND RIPARIAN FOREST	AC	5.7	$ 50,000	$ 285,676
46	01040	REVEGETATION - SWAMP FOREST	AC	3.5	$ 50,000	$ 173,334
WATERWAY ENHANCEMENTS						
47	01091	ENGINEERED STREAMBED MATERIAL (ESM)	TON	2425	$ 150	$ 363,679
48	00330	CHANNEL AND DITCH FILL EMBANKMENT	CUYD	1314	$ 12	$ 15,788
49	00330	HABITAT BERM AND FLOODPLAIN ROUGHNESS EMBANKMENT	CUYD	1442	$ 12	$ 17,304
50	00330	STREAMBED EXCAVATION	CUYD	3933	$ 35	$ 137,670
51	01091	WOOD HABITAT STRUCTURES	LS	1	$ 127,500	$ 127,500
					Construction Subtotal	**$ 3,048,199**
					15% Contingency	*$ 456,830*
					GRAND TOTAL CONSTRUCTION COST	**$ 3,503,129**
					STREAM PORTION (APPROXIMATE):	**$ 1,596,304**
					BRIDGE AND ROADWAY PORTION (APPROXIMATE):	**$ 1,906,825**
					Construction Engineering and Permit Compliance Services (20%)	**$ 700,626**

NOTES:

1. This preliminary opinion of probable construction cost is based on the Draft 90% Design which will be further refined as the project advances towards implementation.
2. The above cost opinion assumes 2027/2028 dollars (anticipated construction timing) and does not include future escalation or financing costs.
3. Any estimate of construction cost prepared by ESA represents its judgement as a design professional and is supplied for general guidance to the Client. Since ESA has no control over the cost of labor and material, competitive bidding, or market conditions, ESA does not guarantee the accuracy of such estimates as compared to Contractor bids or actual costs to the Client.

Appendix D.
Sitka Sedge Tidal Wetland Restoration - No Rise Rationale Memo and No Rise Certification

FLOODWAY "NO-RISE / NO-IMPACT" CERTIFICATION

This document is to certify that I am duly qualified engineer licensed to practice in the State of

Oregon . It is to further certify that the attached technical data supports
(State)

the fact that proposed Sitka Sedge Tidal Wetland Restoration will not impact the base flood
(Name of Development)

elevations, floodway elevations, and floodway widths on Sand Lake at published
(Name of Stream)

cross sections in the Flood Insurance Study for, Tillamook County, OR , dated September 28, 2018
(Name of community) *(Date)*

and will not impact the base flood elevations, floodway elevations, and floodway widths at the

unpublished cross-sections in the area of the proposed development.

STAMP



March 9, 2025

SEAL, SIGNATURE AND DATE

Hunter White, P.E.
Name

Engineer / Hydrologist
Title

1615 SE 3rd Ave, Suite 400

Portland, OR 97214
Address

FOR COMMUNITY USE ONLY:		
Community Approval ☐ Approved ☐ Disapproved		
Community Official's Name	Community Official's Signature	Title

Environmental Science Associates
1615 SE 3rd Ave
Suite 400
Portland, OR 97214
(503) 274-2010

memorandum

date	March 9, 2026
to	Christer LeBrecque – Tillamook Estuaries Partnership
from	Hunter White, P.E. and Elizabeth Schalo, P.E.
subject	**Sitka Sedge Tidal Wetland Restoration – No-Rise Rationale Memorandum**

1.0 Introduction and Overview

Oregon Parks and Recreation Department (ORPD), in partnership with Tillamook Estuaries Partnership (TEP) and Tillamook County Public Works (TCPW), is working towards implementing a series of improvements and modifications to the infrastructure at Sitka Sedge State Natural Area (SSSNA) to restore tidal hydrology, aquatic organism passage, and habitat conditions in the area known as Beltz Marsh while maintaining equivalent or better flood protection for the community of Tierra Del Mar (TDM). The project is known as the Sitka Sedge Tidal Wetland (SSTW) Restoration project and is slated for construction beginning in Summer 2026 pending receival of necessary permits and construction funding.

The primary goals of the Sitka Sedge Tidal Wetland project are to:

- Reconnect Beltz Marsh to tidal and estuarine processes in the adjacent Sand Lake Estuary,
- Improve fish passage conditions into Beltz Marsh and the three creeks (Reneke, Beltz, and No Name/East Marsh) to address limiting factors preventing recovery of ESA-listed Oregon Coast coho salmon and benefit other native fish species,
- Cause no negative impacts to neighboring properties in Tierra Del Mar (TDM) and, to the extent feasible, improve TDM's flood protection, drainage capacity, and resilience to climate change and sea-level rise, and
- Maintain or improve recreational access to the Sitka Sedge State Natural Area trail network.

In 2017-2019, OPRD contracted with consultants to perform a range of site assessments, develop preliminary restoration and infrastructure improvement alternatives, and conduct preliminary hydrologic analysis to select a preliminary preferred alternative including a dike breach to reconnect Beltz Marsh to Sand Lake, coupled with a setback dike near the south end of Beltz Marsh to maintain flood protection for the adjacent community of Tierra Del Mar and TCPW-owned Sandlake Road (PGG, 2019). In 2023-2024, ESA completed a detailed alternatives analysis, 2D hydrodynamic modeling, and 30% Design for the Sitka Sedge Tidal Wetland (SSTW) Restoration

project, with several presentations to technical advisory teams and at public townhall meetings. The methodology for development of the hydrologic and hydraulic models as well as analysis of model results of design alternatives are described in the ***Sitka Sedge Tidal Wetland – Hydraulic Analysis Memorandum*** (ESA, 2024) and **Sitka Sedge Tidal Wetland Restoration 30% Basis of Design** (ESA, 2025). The ***Sitka Sedge Tidal Wetland – Hydraulic Analysis Memorandum*** is included in the Appendix. **Figure 1** shows the project location.

ESA is currently contracted for the engineering and permitting services for the Final Design phase of the Sitka Sedge Tidal Wetland Restoration project. The project is currently at a 60%-Design stage, and the project team is utilizing the 60% Plans and project design details to apply for state, federal, and local permits. The hydraulic analysis and modeling have also been updated to reflect the 60% design configuration of the proposed dike breach and setback dike location.

The purpose of this memorandum is to provide evidence and rationale to prepare a No-Rise Certification for the proposed actions associated with the Sitka Sedge Tidal Wetland Restoration project. The following information demonstrates that the proposed project actions will not cause any rise to the Base Flood Elevation in the Sand Lake Estuary and will reduce peak water levels in the area south of the proposed setback dike.

2.0 No-Rise Criteria

Title 44 of the Code of Federal Regulations (CFR) 60.3(d)(3) states that a community shall,

> "Prohibit encroachments, including fill, new construction, substantial improvements, and other development within the adopted regulatory floodway unless it has been demonstrated through hydrologic and hydraulic analyses performed in accordance with standard engineering practice that the proposed encroachment would not result in any increase in flood levels within the community during the occurrence of the base flood discharge." Once a project is determined to be within a floodway, the next step is to demonstrate that the development will cause no rise in the base flood. This can be done through a hydraulic study, commonly called a no-rise or zero-rise analysis, and results in a no-rise certification (FEMA 2023).

Figure 2 is an excerpt of the FEMA Flood Insurance Rate Map (FIRM) of the project area. The FIRM demonstrates the project area is not located within a floodway. The project is located with a FEMA Zone AE without a floodway. The Base Flood Elevation (BFE) of 11.8' NAVD88.

Title 44 of the CFR 60.3(c)(10) states that a community shall,

> "Require until a regulatory floodway is designated, that no new construction, substantial improvements, or other development (including fill) shall be permitted within Zones A1-30 and AE on the community's FIRM, unless it is demonstrated that the cumulative effect of the proposed development, when combined with all other existing and anticipated development, will not increase the water surface elevation of the base flood more than one foot at any point within the community." Once a project is determined to be within an AE Zone, such as this project, the next step is to demonstrate that the development will not cause a rise in the base flood greater than 1 foot. This can be done through a hydraulic study, and may also be called a no-rise or zero-rise analysis, and results in a no-rise certification, although the project may increase base flood elevation up to one foot under federal regulations.

FEMA Region X published its ***Procedures for "No-Rise" Certification For Proposed Development in the Regulatory Floodway*** in 2013. It details the technical data that should support a "no-rise" certification as being "based upon hydraulic analyses that utilize the same model used to prepare the effective Flood Insurance Study (FIS) report and Flood Insurance Rate Map (FIRM) unless it is demonstrated that the 'effective' hydraulic model is unavailable or its use is inappropriate. If an alternative hydraulic model is used, the new model must be calibrated to reproduce the FIS profiles within 0.5 feet" (FEMA, 2013).

The project area is shown on the FIRM as a backwater of Sand Lake (FEMA, 2018a). The FIS for Tillamook County lists Sand Lake as having its Base Flood Elevation (BFE) as a non-coastal stillwater elevation (FEMA 2018b). However, the Tillamook Coastal Study, which informs the Tillamook County FIS, states the following:

> "Sand Lake is one estuary which had not previously been subjected to detailed coastal or riverine analyses. This particular estuary is periodically influenced by coastal backwater flooding due to extreme coastal water levels. For the purposes of establishing a new BFE in the estuary, we used the still water level (SWL) to map the coastal backwater effect of the 1% and 0.2% flood events into Sand Lake (Figure 8-10)."

Figure 3 reproduces the study's Figure 8-10. The Tillamook Coastal Study also notes the SWL is "the sum of the predicted astronomical tide listed in Tide Tables, plus the effects of processes such as an El Niño or storm surge that can elevate the measured tide above the predicted tide" (DOGAMI, 2014).

The Tillamook County Coastal Study demonstrates that the BFE for the project area is developed from solely coastal influences including the predicted astronomical tide and coastal storms. This project, or any amount of fill in Sand Lake, does not influence the predicted astronomical tide or coastal storm processes and therefore, cannot impact the BFE. Hydraulic modeling as described in the FEMA Region X ***Procedures for "No-Rise" Certification For Proposed Development in the Regulatory Floodway*** is not needed to demonstrate this further.

3.0 No-Rise Analysis

Although this project would not affect the BFE, it may affect other hydrologic conditions and scenarios. ESA has utilized its two-dimensional (2D) HEC-RAS hydraulic model to evaluate potential rise in a 1% flood event (ie 100-Year Event) as a result of the Project.

Four hydraulic models are typically developed for a No-Rise analysis (FEMA, 2013):

1. Duplicate Effective (DE) – The DE model is used to duplicate FEMA effective water surface elevation data. This model reflects the topography and inputs at the time the effective model was created, as a form of quality control for FEMA.

2. Corrected Effective (CE) – The CE model is used to correct errors in the duplicate effective model, add any additional cross sections, or incorporate more detailed topographic information than that used in the current effective model.

3. Existing Conditions (EC) – The EC model is used to reflect current topography, structure data (i.e., bridges and culverts), and hydraulic roughness.

4. Project Conditions (PC) – The PC model is used to reflect project conditions topography, structures, and hydraulic roughness.

As described in Section 2.0, the BFE in the project area was developed from solely coastal influences and not from riverine influences. Therefore, there is not a FEMA effective HEC-RAS (Hydrologic Engineering Center River Analysis System) model for the area. As such, ESA was not able to develop a Duplicate Effective or Corrected Effective model for the No-Rise Analysis. ESA created an Existing Conditions model with the best available data. ESA also developed a Project Conditions model to compare against the Existing Conditions model and draw conclusions for any potential rise.

Existing Conditions and Project Conditions Models

ESA developed and utilized a two-dimensional (2D) HEC-RAS hydraulic model to evaluate the site's existing condition, inform and evaluate alternative design concepts, and evaluate the preferred alternative for potential rise in water levels. 2D hydraulic models use a depth-averaged approach to represent flow patterns in the horizontal plane, producing maps of depth and velocity across the simulated landscape. HEC-RAS 2D was used for this project to represent the complex, multi-directional flows within Sand Lake, Beltz Marsh, flood-prone areas, and local infrastructure.

A series of hydrologic scenarios were developed for the site to calibrate the model and analyze design alternatives, including the 100-year storm event. Flooding is a concern for TDM and as such, project performance under flood conditions was a priority to analyze. As noted in the Tillamook Coastal Study, the project area had not previously been subjected to detailed riverine analyses. ESA developed hydrologic flows using a variety of methods depending on the contributing watershed. Streamflow inputs for the TDM drainage areas were developed utilizing the Santa Barbara Urban Hydrograph Method. Streamflow inputs for creek watersheds were developed utilizing USGS StreamStats, verified by a watershed scaling of the nearest gaged stream comparable to the project creek.

The downstream boundary of the model is located in Sand Lake approximately 2,000 feet downstream of Beltz Dike. This downstream boundary condition is tidal. The tides data used was water level data measured outside of the Beltz Dike February 1-13, 2017. This data is appropriate for use for the 100-year event scenario because of the high tide (11.9 ft NAVD) included in the dataset that is concurrent with the hydrograph peak and also near the published BFE (11.8 ft NAVD).

The 2D hydraulic model requires topography, hydrology, and roughness as inputs for the model, which are represented within the model mesh. Alternative design concepts were assessed with model depth, water level, and velocity outputs. These scenarios and model development are described in detail in the ***Sitka Sedge Tidal Wetland – Hydraulic Analysis Memorandum*** (ESA, 2024), which is in the Appendix.

4.0 No-Rise Results

Water surface elevations are compared between the Project Conditions model and Existing Conditions model for the 100-year flood event in Table 1. There are no FEMA cross sections or BFE lines in the project area. The water surface elevations were compared at two key locations: in the marsh behind the proposed setback dike (**Figure 4**) and outside of the existing dike in Sand Lake (**Figure 5**). As shown in Table 1 and Figures 4 and 5, the

hydraulic modeling predicts a flood reduction near TDM as a result of the project. 100-year water surface elevations are approximately 1 foot lower under project conditions than existing conditions in the marsh behind the project setback dike (Figure 4). This demonstrates the project is not anticipated to cause a rise in 100-year flood levels but rather, a reduction. Outside of the existing dike, in Sand Lake, the hydraulic model predicts no change in water level comparing the existing conditions to the project conditions.

TABLE 1. EXISTING AND PROJECT 100-YEAR FLOOD WSE COMPARISON

	FEMA BFE (feet NAVD88)	Existing Conditions (feet NAVD88)	Project Conditions (feet NAVD88)	Project minus Existing (feet)
Inside Project Setback Dike (Figure 4)	11.8	12.05	11.03	-1.02
Outside Existing Dike (Sand Lake) (Figure 5)	11.8	11.86	11.86	0.00

The results of these analyses demonstrate that the proposed Project will not cause an increase in 100-year flood event elevations in Sand Lake or Sitka Sedge Wetland relative to Existing Conditions. This conclusion is based on a comparison between Existing and Project Conditions model results, which shows no increase in water surface elevations under Project Conditions for the 100-year flood event. We therefore request that the County Floodplain Administrator approve this No-Rise certification.

5.0 References

Corps of Engineers, 2023. HEC-RAS River Analysis System: Hydraulic Reference (Version 6.4.1). U.S. Army Corps of Engineers Hydrologic Engineering Center.

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Environmental Science Associates (ESA), 2025. Sitka Sedge Tidal Wetland Restoration 30% Basis of Design. Revised January, 2025.

ESA, 2025. Sitka Sedge Tidal Wetland – 60% Design Plans. September, 2025.

Federal Emergency Management Agency (FEMA) Region X, 2013. Procedures for "No-Rise" Certification for Proposed Developments in the Regulatory Floodway. October 2013.

FEMA, 2018a. National Flood Insurance Program Flood Insurance Rate Map, Tillamook County, Oregon, Map Number 41057C0715F. Effective Date September 28, 2018.

FEMA, 2018b. Flood Insurance Study, Tillamook County, Oregon and Incorporated Areas. Flood Insurance Study Number 41057CV001A. Effective Date September 28, 2018.

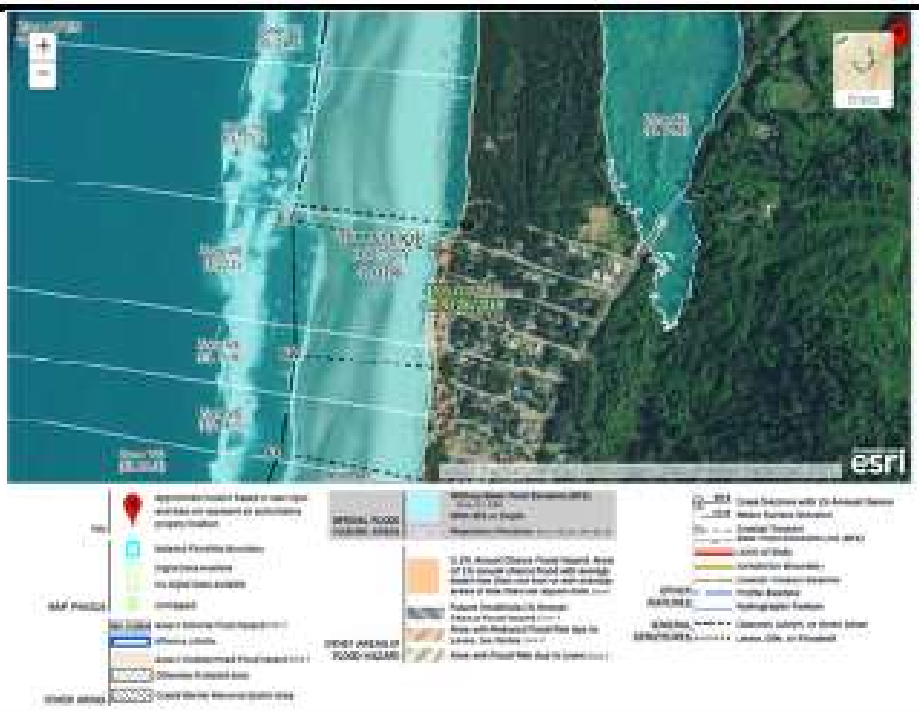
FEMA, 2023. Guidance for Flood Risk Analysis and Mapping, Floodway Analysis and Mapping. November, 2023.

FEMA, 2025. Email Correspondence with FEMA Regional Service Center (RSC) Lead, Josha Crowley. January 7, 2025.

Oregon Department of Geology and Mineral Industries (DOGAMI), 2014. Coastal Flood Insurance Study, Tillamook County, Oregon. June 30, 2014.



Figure 1
Project Location Map

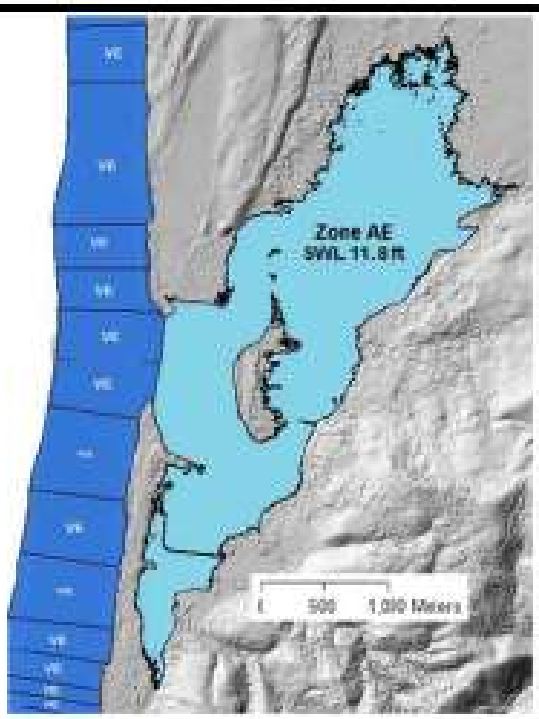


Note: *This figure shows* the FEMA Flood Insurance Rate Map of the project area, including the mapped 100-year floodplain boundary which inundates the lowest lying portions of Tierra Del Mar and overtops the low point in Smallfate Road at the East Marsh crossing. The road low point elevation is 11.5' NAVD88 compared to the 100-year floodplain elevation of 11.8'.

Figure 2
FEMA Flood Insurance Rate Map Excerpt from North End of TDM

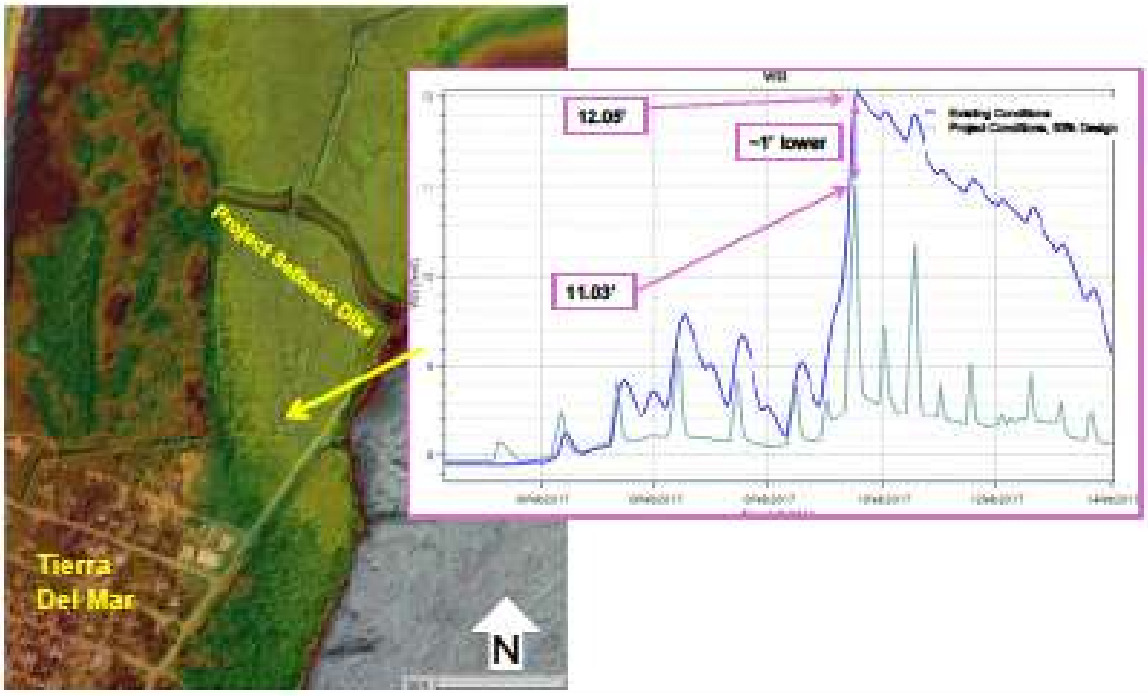
Sitka Sedge Tidal Wetland Restoration
No-Rise Memorandum

ESA



Note: This figure is recreated from the Coastal Flood Insurance Study, Tillamook County, Oregon, Figure 8- 10. It shows the FEMA floodplain in the project area is a backwater of Sand Lake and is a still water level (SWL) of 11.8 ft. The Tillamook Coastal Study notes the SWL is "the sum of the predicted astronomical tide listed in Tide Tables, plus the effects of processes such as an El Niño or storm surge that can elevate the measured tide above the predicted tide."

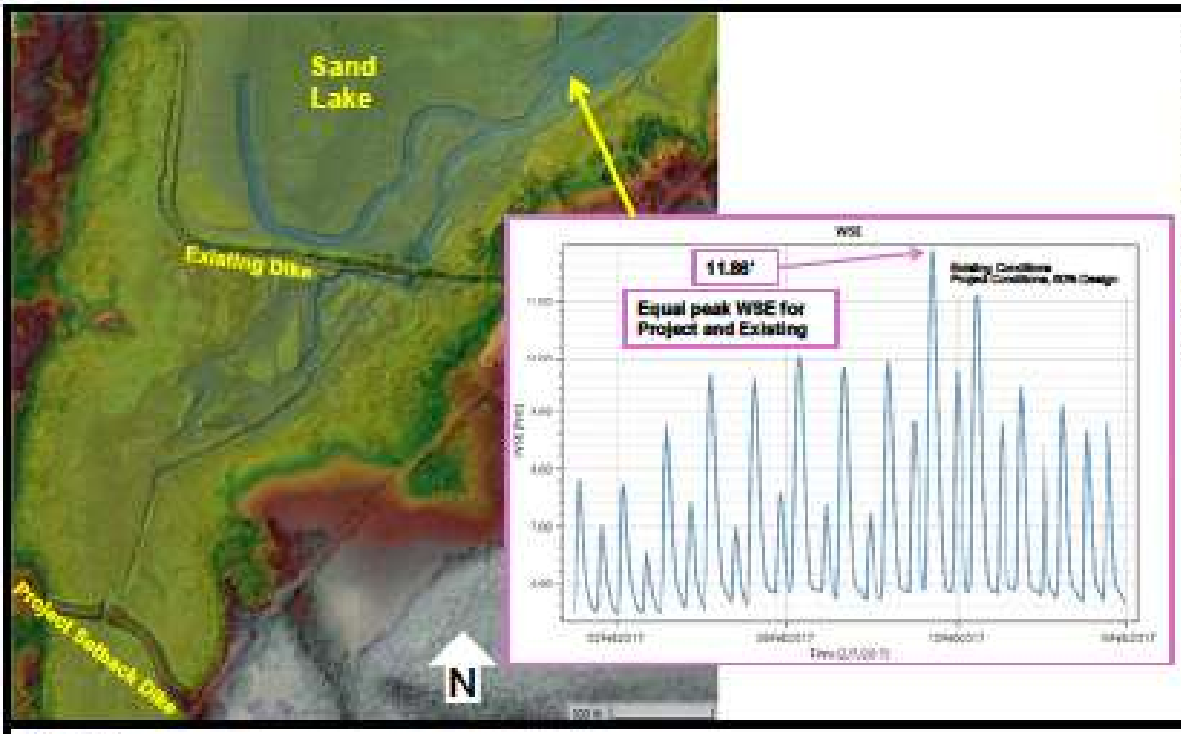
Figure 3
Coastal backwater flooding mapped from SWLs for Sand Lake. The 0.2% chance flooding is too small to be visible at this scale. (DOGAMI, 2014)
Sitka Sedge Tidal Wetland Restoration
No-Rise Memorandum



Note: **This figure shows the water surface elevations for a 100-year flood event concurrent with high winter tides. Existing Conditions results in a predicted peak WSE of approximately 12 ft. The Project Conditions is predicted to peak at 11 ft. This simulation shows the Project is expected to reduce 100-year peak water levels, near Tierra Del Mar and significantly reduce the duration of high water inundation compared to existing conditions.**

Figure 4
Hydraulic Model Results – 100-Year Water Surface Elevation 60% Design vs Existing Conditions, Inside Project Setback Dike
Sitka Sedge Tidal Wetland Restoration
No-Rise Memorandum

ESA



Note: **This figure shows the water surface elevations for a 100-year flood event concurrent with high winter tides. Existing Conditions results in a predicted peak WSE of 11.86 ft outside of the Existing dike, in Sand Lake. The Project Conditions is predicted to peak 11.86 ft. This simulation shows the Project is expected to result in no change to 100-year peak water levels outside of the Existing dike, in Sand Lake, compared to existing conditions.**

Figure 5
Hydraulic Model Results – 100-Year Water Surface Elevation 60% Design vs Existing Conditions, Outside of Existing Dike (Sand Lake)
Sitka Sedge Tidal Wetland Restoration
No-Rise Memorandum

ESA

Appendix E.
Sitka Sedge Tidal Wetland Restoration – Fish Passage and Tide Gate Design Memorandum

ESA

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Memorandum

date	October 27, 2026
to	Mac Barr, Greg Apke, Robert Bradley, Mike Sinnott – Oregon Department of Fish and Wildlife (ODFW)
cc	Christer LaBrecque – Tillamook Estuaries Partnership (TEP) Tracy Johnson, Noel Bacheller, Jason Elkins – Oregon Parks and Recreation Department (OPRD) Olivia Deans, Jennifer Jones, Cheryl Butner – National Oceanic and Atmospheric Administration (NOAA)
from	Hunter White, PE – Environmental Science Associates (ESA)
subject	Sitka Sedge Tidal Wetland Restoration – Fish Passage and Tide Gate Design

1 – INTRODUCTION AND SUMMARY

Oregon Parks and Recreation Department (OPRD) in partnership with Tillamook Estuaries Partnership (TEP) is planning a tidal wetland restoration and fish passage improvement project at Sitka Sedge State Natural Area (SSSNA). ESA is working with these project partners to restore tidal processes and enhance habitat conditions in the area with SSSNA known as Beltz Marsh, including improving fish passage and access to the three tributary streams that drain into Beltz Marsh, including Reneke Creek, Beltz Creek, and No Name Creek. The site is currently separated from Sand Lake by an existing earthen dike (Beltz Dike) with a single drainage outlet through a 4'x4' concrete box culvert. The culvert once featured a wooden, top-hinged tide gate that prevented tidal water from flowing into Beltz Marsh, but the wooden boards are no longer present, allowing a muted tidal connection and limited fish passage through the culvert. However, the culvert is undersized for the flows that pass through the culvert resulting in very high velocities that are likely a partial fish passage barrier or create turbulent conditions that are detrimental to fish passing through the culvert.

The Sitka Sedge Tidal Wetland (SSTW) project includes proposed infrastructure and habitat improvements to restore tidal processes and improve fish passage conditions between Sand Lake and Beltz Marsh and to the tributary streams that drain into the marsh. An overview of the proposed SSTW restoration project elements is shown in Figure 1.

Proposed Dike Breach and Pedestrian Bridge:
The proposed restoration project includes breaching the existing Beltz Dike at a location west of the existing box culvert by excavating a new primary tidal channel through the dike that connects two apparent historical tidal channels inboard and outboard of the dike embankment. Historical aerial photo analysis indicates that this location was likely the original primary tidal channel location prior to construction of Beltz Dike in the early 1900s. To maintain recreational access via the Beltz Dike Trail that travels along the top of the existing dike embankment, the project includes a new 140-foot-span prefabricated pedestrian bridge over the dike breach channel. The breach channel and associated bridge span were sized based on tidal channel sizing guidelines,

hydraulic analysis and evaluation of water velocities relative to fish passage criteria and erosion potential, and alternatives analysis to restore fish passage while maintaining trail access at SSSNA. Modeled velocities through the 140-foot-span and dike breach are expected to stay below 2 fps even during peak drain out during the 100-year storm scenario. The existing drainage box culvert will be fully removed, and the dike embankment in this location will be reconstructed to repair erosion and maintain trail access into the future.

Proposed Setback Dike, Tide Gate Structure, and Backup Culverts:
To maintain flood protection and prevent peak high tide water levels from reaching the adjacent community of Tierra Del Mar (TDM) after breaching the existing dike, the project includes constructing a new earthen setback dike across the south end of Beltz Marsh. The location of the setback dike was designed to minimize the area of direct wetland impacts and minimize the area of wetland to remain behind the setback dike, while providing adequate water storage volume behind the setback dike to contain stormwater runoff from Tierra Del Mar and the No Name Creek watershed to result in a reduction of peak water levels at the northern edge of Tierra Del Mar compared to existing conditions. Beltz Creek and Reneke Creek will flow into the tidally reconnected portion of Beltz Marsh north of the setback dike. No Name Creek will drain into the area south of and behind the setback dike, draining beneath Sandlake Road and into a network of drainage ditches that are impounded by existing beaver dams.

Combined flows of No Name Creek and stormwater inputs from Tierra Del Mar, via the Sandlake Road Ditch and TDM North Ditch, and direct precipitation on the land/water surface south of the setback dike and north of the TDM drainage ditches will be conveyed through the setback dike via a proposed concrete tide gate structure consisting of a 12-foot-span box culvert, fitted with an 8-foot-tall by 10-foot-wide side-hinged aluminum tide gate with a Muted Tidal Regulator (MTR) control system. The MTR system will be operated to force the tide gate to stay open during most tidal water levels (approximately 91% of the year) but close during peak high tides to prevent the highest of tides from reaching private properties in TDM. This proposed tide gate structure with MTR controls will provide fish passage through the setback dike and along the No Name Creek pathway. *Additional tide gate fish passage analysis is summarized below within this Introduction and Summary and described in greater detail in Section 5.*

As a conservative backup measure to alleviate concerns of TDM landowners, the setback dike also includes two 36-inch diameter backup culverts that will be set at higher elevations to drain off high water levels that may accumulate behind the setback dike in the unlikely event that the tide gate structure became blocked due to beaver activity or other emergency conditions. These backup culverts are not intended to provide fish passage and would only activate with flow infrequently during extreme storm events or emergency conditions. These backup culverts would be fitted with traditional top-hinged flap gates mounted on the downstream concrete headwalls to prevent extreme peak tides from passing upstream through the backup culverts.

Beaver Management System at Setback Dike Tide Gates:
Due to the presence of heavy and persistent beaver activity, the new tide gate structure will be protected by a beaver exclusion system to prevent beaver from accessing and blocking the culvert or tide gate and a Beaver Deceiver flow device with a bypass pipe to ensure water flows through the exclusion fencing in the even that beaver build a dam or rack debris against the exclusion fencing. All beaver exclusion fencing and flow device filters would be constructed with 6"x6" mesh fencing (cattle panel fencing) with minimum gap spacing of 6 inches to meet ODFW fish passage criteria for cutthroat trout-only streams, as ODFW has determined is appropriate for the documented and anticipated fish use in No Name Creek.

Summary of Fish Passage Analysis at Proposed Tide Gate Structure:
Oregon's fish passage rules are set forth in Oregon Administrative Rules (OAR) Chapter 635, Division 412 – Fish Passage. Section 635-412-0035 states "tide gates associated with fish passage structures shall meet the requirements of OAR-412-0035(2)", which provides maximum velocity and minimum depth criteria, "for an

average of at least 51% of tidal cycles". As referred to in this memorandum, 'passability' means the tide gates are open, and velocity and depth criteria are being met.

In order to achieve at least 51%-passability, it would be necessary to keep the tide gate open at all times when water levels are at 5.5' (the minimum water level in Sand Lake at the project location due to bed features in the lake that prevent full drain out). This minimum threshold will be easily met with the proposed tide gate's MTR Water Management Plan, which proposes to use an MTR water level setting of 8.0' NAVD88. Due to existing beaver dams upstream of the proposed setback dike and tide gate structure, water levels upstream of the beaver dams never fall below approximately 7.5' to 8' during summer and winter conditions, respectively. Therefore, regularly allowing a water level of 8.0' to occur upstream of the proposed tide gate will just barely overtop the existing beaver dam and will not substantially change the water levels that occur in the southern portion of Beltz Marsh or on private properties in Tierra Del Mar.

Following tidal reconnection via the proposed dike breach, tidal water levels outboard of the setback dike will rise and fall between elevations of 5.5' and approximately 12'. Tidal water levels immediately upstream of the proposed setback dike and tide gate will be managed to rise and fall between elevations of 5.5' to 8.0' during typical tidal conditions, with higher water levels occurring upstream of the setback dike during periods of stormwater runoff when the tide gate is closed. With a minimum low water surface elevation of 5.5', we have selected a culvert invert elevation of 5.0' so that there will always be a minimum of 6 inches of water depth (0.5') in the tide gate structure.

Extensive hydraulic modeling of the proposed tide gate structure during a wide range of conditions demonstrates that water velocities through the tide gate structure and tide gate opening are expected to stay below 2 feet per second (fps) during typical tidal conditions, reach just over 2 fps during a 2-year storm event, and peak just below 4 fps during the peak of a 100-year storm event. Therefore, velocity and water depth requirements for fish passage will be met at all times when the tide gate is open, only exceeding the fish passage velocity threshold of 2 fps infrequently during storm events exceeding a 1-year or 2-year recurrence interval.

The fish passage analysis described in this memo indicates that a tide gate system consisting of a 12-foot-wide concrete box culvert, with a single, 10-foot-wide by 8-foot-tall side-hinged tide gate with Muted Tidal Regulator (MTR) controls will comply with ODFW fish passage velocity criteria.

In summary, we propose a tide gate system with the following characteristics:

- **A concrete tide box structure with wing walls and a 12-foot interior span**
- **A 14' wide x 15' tall concrete headwall equipped with a single 10'Wx8'H side-hinged tide gate**
- **Gate invert elevation at 5.0 feet NAVD88**
- **MTR Controls with a year-around MTR Water level setting of 8.0 feet NAVD88**
- **With these proposed tide gate dimensions and operations, the tide gate would be open and meeting ODFW fish passage criteria for approximately 91% of the time over the course of the full year, with monthly fish passage ranging from 84% of January to 94% of June and July.**

Sandlake Road Crossings at Reneke, Beltz, and No Name Creeks:
Reneke Creek, Beltz Creek, and No Name Creek all originate in watersheds east of Sandlake Road and enter Beltz Marsh after passing through undersized culverts beneath Sandlake Road. The SSTW project designs include replacing the undersized culvert at Reneke Creek and Sandlake Road with a new 49-foot-span roadway bridge. This Reneke Creek/Sandlake Road crossing fish passage improvement is designed to meet Stream Simulation fish passage. Construction of the bridge and associated roadway work may be constructed separately from the rest of the SSTW project, awaiting additional grant funding.

Culvert replacements at the Sandlake Road crossings of Beltz Creek and No Name Creeks are designed to a 30%-level. These crossing designs will be finalized in the future after additional design funding is obtained, and these culvert replacements will be permitted and constructed separately in the future. These crossings are not included in the SSTW project and are not seeking fish passage authorization at this time.

The Sandlake Road culvert crossings are not covered in this memorandum and will be submitted separately for fish passage authorization.

The following sections include site descriptions, site maps illustrating site conditions and context, water level monitoring data, and a summary of hydraulic and fish passage analysis performed. All figures are included in Appendix A. ODFW Fish Passage Plan for a Road Crossing-Stream Structure and Beaver Coexistence Flow Device Fish Passage Approval Application forms will be submitted to ODFW with the formal application. The 90% Plans are also provided for additional project information and context.

2 – PROJECT DESCRIPTION

2.1 - Existing Site Conditions and Setting

In 2014, OPRD acquired property north of Pacific City, including and adjacent to portions of Sand Lake Estuary. The property established as Sitka Sedge State Natural Area (SSSNA) consists of approximately 102 acres of marsh, including approximately 70 acres of tidal marsh, sand, and mudflats, 32 acres of freshwater marsh, and 22 acres of forested or shrub swamp. The property includes approximately 78 acres of estuary altered by the half-mile long Beltz Dike which runs east/west across the property.) Tierra Del Mar (TDM) is a residential community located immediately south of Beltz Marsh (Figure 2). Relative to surrounding areas, TDM is flat, with elevations of developed portions ranging from around 14 to 23 feet NAVD88. TDM is triangular in shape, narrowing to the south. It is surrounded by the Pacific Ocean and associated foredune to the west, additional vegetated dunes and Beltz Marsh to the north, and forested bedrock hills to the east. A marshy swale referred to as No Name Creek or "East Marsh" occurs between TDM and the bedrock hills to the east (PGG, 2019).

Park property contains 3 major streams: Reneke Creek, Beltz Creek, and "no-name" creek. All of these streams are tributary to the tidal marsh and Sand Lake Estuary.

SSSNA includes a parking lot and trailhead located adjacent to Sandlake Road immediately north of the Reneke Creek area. The trailhead provides recreational access to a network of hiking trails and beach access via the Beltz Dike Trail situated on top of the earthen levee that separates Beltz Marsh from Sand Lake.

Prior to diking, Beltz Marsh was formerly a shallow embayment fully connected to the Sand Lake Estuary via tidal channels and mudflats. Topographic modifications including construction of the Beltz dike, ditching in the marsh, construction of Sandlake Road, quarry activities, and development of the Tierra Del Mar community continue to affect site hydrology (PGG, 2019).

Figure 3 includes an aerial photo of the project area identifying the key site features. Figure 4 depicts an annotated site topography map derived from OPRD DEM (various dates). Key site features include the Beltz Dike, box culvert and tide gate, remnant tidal channel and ditch network, primary beaver dam in the marsh, Sandlake Road prism, Tierra Del Mar community ditches, east marsh complex, and dunes to the west of the marsh. Areas within the marsh having elevations below 5.0' NAVD88 remain inundated most of the time, while areas with elevations above 8.0' NAVD88 are rarely inundated. The low point in Sandlake Road has an elevation of approximately 11.1' NAVD88, overflowing the roadway at 11.6'. Residential areas within Tierra Del Mar

(TDM) have ground elevations ranging from 14.0 to 23.0', or higher. Two private properties in TDM at the southern edge of SSSNA include existing low-lying areas subject to inundation by the highest measured water levels of approximately 12' and backwater flooding during heavy storm events. The TDM ditch and culvert network flows north and discharges into the marsh above the direct influence of the tide (PGG, 2019).

FEMA Flood Insurance Rate Maps from the Tillamook County Flood Insurance Study show the 100-year floodplain elevation in Beltz Marsh and the Sandlake Rd/East Marsh crossing of 11.8' NAVD88. This flood elevation inundates the low point in Sandlake Rd and the low-lying portions of three private properties within TDM.

2.2 - Existing Surface Water Hydrology

2.2.1 - Tides

Tides are the primary driver of surface water level fluctuations in Beltz Marsh. Tidal fluctuations in the Pacific Ocean are transmitted to the marsh via the shallow Sand Lake Estuary and communicated through the existing box culvert and tide gate. The tide gate has deteriorated over time and no longer has a functional gate flap, but the undersized box culvert restricts flow into the marsh and mutes the tidal signal resulting in a reduction in peak tidal water levels.

Pacific Ocean tides are influenced by both astronomical (sun & moon) and meteorological factors (storms). Storms and other large-scale atmospheric phenomenon, such as El Niño, tend to elevate the tides above predicted levels during the winter season. These same storm events are also responsible for delivering heavy rain to the area that frequently corresponds with high tide events. Measurements of ocean tides in the immediate vicinity of Sand Lake Estuary are not available. Long-term measurements at South Beach in Newport, OR, and Garibaldi, OR (within Tillamook Bay), provide representative ocean tides offshore of Sand Lake Estuary. As ocean tides enter the Sand Lake Estuary they are affected by the shallow nature of the estuary, constantly changing sand bed forms near the mouth of Sand Lake, and a complex tidal channel network, resulting in a situation where water levels in the estuary near the Beltz Marsh never fully recede to match low tides in the adjacent ocean. Daily low tides are severely muted and never fall below elevation 5.5 ft NAVD88, while ocean tides regularly fall to 0.0 ft NAVD88 or lower.

Fluctuations behind the dike in Beltz Marsh are primarily controlled by the interaction of tides in Sand Lake with the box culvert and tide gate, and secondarily by inflow from creeks and local drainages. Measurements show that under existing conditions, the missing tide gate and undersized box culvert allows a muted tidal flux to fill the marsh on rising tides. Water passes through the box culvert and water levels behind the dike rise more slowly and lag behind those in Sand Lake due to the limited flow through the structure. Tides in Sand Lake peak and begin to fall before the marsh completely fills and as water levels equilibrate on the ebb tide the flow reverses and water slowly drains back into Sand Lake until the minimum low water elevation of ~5.5 ft NAVD88 is reached. In 2017 when the tide gate was still partially intact, high tide elevations behind the dike were approximately 1.9 feet lower than Sand Lake, as shown in Figure 5. In 2023, since the tide gate was completely missing, high tide water levels behind the dike were measured to be approximately 1.4 feet lower than Sand Lake and approximately 0.5 feet higher than in 2017, as shown in Figure 6.

In the absence of significant freshwater input, low tides behind the dike tend to be similar to those in Sand Lake but are slightly elevated (a few inches) and of longer duration as the marsh slowly drains. Freshwater inputs from creeks and local drainages that arrive on a rising tide accumulate within the marsh and contribute to elevated water levels behind the dike. Following heavy rainfall events, it can take multiple tidal cycles for

accumulated freshwater to be discharged through the existing box culvert. The leaky/missing tide gate is no longer functional, though the size of the box culvert itself limits flux through the dike and thereby causes lower water levels than in Sand Lake at high tide, and higher water levels than in Sand Lake at low tide and following intense rainfall events.

Existing tidal fluctuations in the south end of the marsh are further impeded by the existing beaver dam that impounds water in the ditches and wetlands near Sandlake Road. Water levels remain relatively constant in time at about elevation 8.0 ft NAVD88 during the wet season and drop down to elevation 7.5 ft during summer low flow period. The beaver dam is approximately 3.0 ft high, based on observations by ESA staff in February 2018. Figure 7 depicts typical water level fluctuations in the project vicinity during the summer (no rainfall), and Figure 8 depicts typical water level conditions in the winter during and after a storm event (PGG, 2019).

2.2.2 - Freshwater Inputs

Frequent and intense rainfall is common along the Oregon coast, and in the vicinity of the project site. Mean annual rainfall is about 80 inches per year, with a 2-year peak 24-hour rainfall intensity of about 3 inches (USGS, 2018). Other rainfall records are available at Tillamook and Cloverdale, OR. Periods of intense rainfall correspond to both elevated water levels in Beltz Marsh, and photographs of neighborhood flooding provided by the TDM community from 2016-2017 (pers. com). Given the size of Beltz Marsh, direct rainfall in the marsh could contribute meaningful surface water inflow during high-intensity storm events.

Beltz Creek (0.5 square mile drainage) and Reneke Creek (0.9 square mile drainage) are the primary freshwater sources that discharge into Beltz Marsh under existing conditions. Neither of the creeks has been gaged and few streamflow measurements are available to describe the response of the creek drainages to rainfall events. Therefore, the current understanding of creek hydrology relies on limited observations, comparisons with nearby gaged steams (Tucca Creek), and scaling to approximate discharge from Beltz and Reneke Creeks (PGG, 2019).

Reneke Creek flows west toward Sandlake Road, where it encounters a blocked culvert and then diverts southwest along Sandlake Road. The creek then flows into a partially blocked 38-ft long, 2-ft diameter, culvert that passes beneath Sandlake Road into Beltz Marsh. The culvert invert elevations are low enough to be subject to tidal fluctuations in the marsh during extreme high tides and ESA observed what appeared to be salmonid fish in the pool immediately downstream of the culvert. Upon entering the riparian area west of Sandlake Road, the creek channel meanders through wetlands and then bifurcates into multiple smaller braided channels that separately discharge into the marsh. Signs of erosion and road overtopping indicate that the existing culvert is undersized and does not pass all creek flows beneath the road. Historic photos and ground observations indicate that Reneke Creek was formerly artificially routed through the blocked culvert and then to the north along what is currently a dry swale that historically diverted Reneke Creek around the existing dike (PGG, 2019).

Beltz Creek arrives at Sandlake Road at a higher elevation than Reneke Creek and passes beneath Sandlake Road through an angled 80-ft long, approximately 3-ft diameter, culvert. The downstream end of the culvert is perched about 2.5 ft above the adjacent creek channel and well above the influence of the tide. The creek channel then flows through a former rock quarry and splits into at least two smaller channels before entering the marsh and ditch network.

The primary drainage ditch from TDM (identified as the East Ditch or Sandlake Road Ditch) flows north through a series of small culverts that parallel Sandlake Road. The final culvert beneath Roma Ave. consists of a 40-foot long 12-inch diameter corrugated smooth interior wall HDPE pipe with an outlet invert elevation of 11.8 ft

NAVD88. The ditch continues north and connects with other ditch networks upstream of the existing beaver dam within the marsh. Flows through the ditch along Sandlake Road are limited by the capacity of the culverts and condition of the ditch. A secondary drainage swale within TDM (identified as the North Ditch in Figure 4) runs parallel to the dune line and flows northeast through TDM, including beneath roads and structures, before turning in an east-northeast direction north of Roma Ave.

Finally, drainage from the East Marsh located on the east side of Sandlake Road and across from TDM, flows into the south end of Beltz Marsh via a 2-ft diameter culvert beneath Sandlake Road. The East Marsh watershed is approximately 202 acres or 0.32 square miles. Water level monitoring indicates that water levels in the immediate vicinity of Sandlake Road are controlled by the backwater created by the beaver dam in Beltz Marsh. Ground levels and surface water levels in the East Marsh continue to rise to the south, likely impounded by beaver dams and/or emergent vegetation. Observations by ORPD in spring of 2018 indicated water ponded approximately 1 ft above the ground surface throughout this marsh, with deeper water closer to the Sandlake Road culvert (PGG, 2019).

2.2.3 - Hydrologic Characterization Summary

- The Beltz dike tide gate no longer functions properly and allows water to flow into the marsh when water levels are higher in Sand Lake. Still, water levels at high tide (during non-storm conditions) are lower on the inside of the dike under existing conditions due to limited flux allowed in and out of the existing box culvert.

- The existing box culvert/tide gate is undersized to efficiently drain accumulated freshwater within the marsh following intense rainfall events and thus it currently takes multiple low tide cycles for water levels in Beltz Marsh to drain out and equilibrate with Sand Lake.

- The existing beaver dam at the south end of Beltz Marsh impounds water and creates a backwater area that causes water to pond along both sides of Sandlake Road and within the East Marsh.

- Reneke Creek's northern culvert is completely blocked by gravel, and the southern culvert is undersized for passing peak stream flows. This constriction moderates Reneke Creek flows through the culvert at moderate flows and causes overtopping of the roadway during peak streamflow events.

- The East Marsh (No Name Creek upstream of Sandlake Road) is affected by the backwater of the beaver dam in Beltz Marsh and is subject to tidal fluctuations during extreme high tides and storm events. Ponded water has been observed in the East Marsh well above the influence of the backwater from Beltz Marsh.

- Groundwater hydrology characterization and alternatives modeling are covered in detail in the ***Sitka Sedge State Natural Area Hydrologic Analysis Final Report*** (PGG, 2019) Groundwater modeling of a dike breach scenario without a setback dike predicted insignificant changes in shallow groundwater elevations beneath TDM during average conditions and slightly reduced groundwater elevations during large storm events.

2.2 - Proposed Restoration Measures

The Oregon Parks and Recreation Department (OPRD) and Tillamook Estuaries Partnership (TEP) propose to restore estuarine habitat conditions in the Beltz Marsh area of SSSNA and improve fish passage conditions from Sand Lake into Beltz Marsh and the tributary streams by restoring hydrologic connectivity through the dike and

upstream across the road. Tillamook County Public Works (TCPW) is another project partner that will oversee improvements within the Sandlake Road right-of-way. The project team proposes to improve hydrologic connectivity by removing the undersized box culvert within Beltz Dike and the undersized culverts beneath Sandlake Road. Pedestrian and vehicular connectivity will be maintained and improved through the installation of appropriately sized bridges over the restored hydrologic channels. A setback dike will also be constructed at the south end of Beltz Marsh to provide flood protection for residents of Tierra Del Mar, which will include a new tide gate structure to provide drainage and fish passage through the setback dike while preventing peak tidal water levels from reaching TDM.

The project elements are depicted in the 90% Plans and Overview Figure 1, including the following:

- Constructing an earthen setback dike embankment near the south end of Beltz Marsh located approximately 50 feet north/downstream of the existing primary beaver dam to:
  - Prevent high tide water levels from reaching Tierra Del Mar,
  - Provide sufficient storage volume upstream/south of the setback dike to contain stormwater runoff without increasing water levels in Tierra Del Mar, and
  - Maintain the existing beaver dam/lodge complex and freshwater habitat at the south end of Beltz Marsh to minimize potential beaver conflicts at the setback dike and associated tide gate structure.
- Constructing a new tide gate structure within the proposed setback dike designed hydraulic drainage capacity and fish passage featuring:
  - a 12-foot-wide concrete box culvert, concrete headwalls and wingwalls
  - an 8' tall x 10' wide side-hinged aluminum tide gate, with an invert elevation of 5.0'
  - a Muted Tidal Regulator (MTR) control system to operate the tide gate to stay open during the majority of tidal conditions up to an MTR Water Level setting of 8.0'
- Constructing a pair of 36" diameter backup culvert pipes with top-hinged flap gates within the setback dike, set at higher invert elevations of 9.5' to 10' to provide relief drainage in the event that the primary tide gate structure is blocked or during extreme storm events,
- Installing beaver exclusion and management system at the setback dike drainage structures including:
  - 6" x 6" mesh fencing across the upstream and downstream sides of the tide gate structure
  - A Beaver Deceiver flow device including a square filter made of 6"x6" mesh fence panels and a 24" diameter bypass pipe to pass water through the upstream receiver fence
  - Chain link fencing along the tide gate structure's headwalls and wingwalls with gates to provide access for maintenance
  - Trash racks on the inlets of the backup culverts with minimum mesh/bar spacing of 6"
- Filling linear drainage ditches and excavating new tidal channels in the marsh,
- Installing large wood structures in channels and on the marsh plain for habitat complexity and bank stability in select locations,
- Removing the existing 4'x4' box culvert and deteriorated tide gate and reconstructing the dike embankment and overlying trail,

- Excavating a 140-foot-wide breach opening and channel through the existing Beltz Dike to reconnect Beltz Marsh to tidal processes in Sand Lake, located to connect historical tidal channel alignments and minimize tree removal,
- Constructing a 140-foot-span pedestrian bridge over the dike breach to maintain recreational access along the Beltz Dike Trail to the rest of the SSSNA trail network,
- Replacing the undersized culvert at Reneke Creek/Sandlake Road to improve fish passage including:
  - Raising approximately 800 feet of roadway to reduce roadway overtopping and constructing a 49-foot span vehicular bridge in place of the existing undersized culvert
  - Realigning the Reneke Creek channel to move the creek out of the roadside ditch and restore a historical channel flow path and alluvial fan/forested floodplain area.
- A future phase of this project will include the Beltz Creek and No Name Creek culvert replacements, which will primarily impact existing county right-of-way.

3 - WATER LEVEL DATA SUMMARY AND INUNDATION ANALYSIS

3.1 - Existing Water Level Monitoring Data Summary

Water levels in the project area have been monitored at different times during the course of project planning and design including a period in 2017 during the original hydrologic study and again in 2022-2023 during the detailed alternatives analysis leading to the 30% Design. Water levels were monitored continuously inside and outside the existing dike and tide gate, as well as locations upstream at the main beaver dam, south end of Beltz marsh, and upstream of Sandlake Road at Reneke Creek and No Name Creek. Water levels in the project area are summarized as the following:

- Water levels in the Pacific Ocean near the mouth of Sand Lake exhibit full tidal fluctuation from -3.9' to 11.7' (lowest and highest measured tide levels interpolated from NOAA tide stations in Newport and Garibaldi, Oregon)
- Water levels in the Sand Lake rise to the full height of ocean tides and possibly slightly higher based on water level monitoring data. However, the low tides in Sand Lake are muted due to bed features in Sand Lake that prevent full drain out at low tide.
  - At a location within Sand Lake approximately 1 mile upstream of the mouth, the water level falls to a minimum elevation of approximately 3.0'.
  - At the existing Beltz Dike location (approximately 2 miles upstream of the mouth), the water level never falls below 5.5'.
- Water levels in Beltz Marsh, inside the existing dike, are influenced by tidal water levels in Sand Lake and freshwater inputs from Reneke, Beltz, and No Name Creeks, stormwater runoff from the Tierra Del Mar and Sandlake Road ditches, and direct precipitation.
  - During typical tidal conditions, the deteriorated tide gate allows tide water to enter the marsh, but the constriction of the 4'x4' box culvert limits peak tides approximately 1.5' lower than the high tide in Sand Lake.
  - During storm conditions, runoff and streamflow entering Beltz Marsh build up behind the dike and the 4'x4' box culvert has limited capacity to drain the water. Water levels rise in the marsh up to an elevation of nearly 12' and can take several days to drain out.
- Water levels at the south end of Beltz marsh (south of the main beaver dam) are always impounded at approximately 8' (8.5' in winter and as low as 7.5' during the summer, or due to minor changes in the height of the beaver dam).
  - This beaver dam impoundment backwaters the ditch network and No Name Creek channel at Sandlake Road

  - The No Name Creek/Sandlake Road culvert is always submerged, and this backwater effect extends upstream of Sandlake Road as well.
- The highest water levels in Beltz Marsh occur during periods of heavy rainfall due to accumulation of runoff in the marsh and slow drain out through the 4'x4' box culvert through Beltz Dike.

3.2 - Proposed Conditions Modeled Water Levels and Inundation Extents

As detailed in ESA's ***Sitka Sedge Tidal Wetland – Hydraulic Analysis Memorandum*** (ESA, 2024), we performed extensive 2D HEC-RAS modeling of existing conditions and proposed alternative conditions. The 2D models have been updated at each step of the design and to reflect the locations and dimensions of site features included in the 90% Design Plans. Figure 9 shows the 2D model extents and hydrologic boundary conditions.

The HEC-RAS modeling included a series of hydrologic scenario simulations including:

- a calibration event modeled after an actual storm event in February of 2017, which included a period of typical winter tide conditions and moderate stream flows followed by a storm event that generated 1-year stream flows and stormwater runoff
- a long-duration, 14-day hydrologic scenario that includes 3 days of typical wintertime conditions, 5 days of a 'first storm' using 2-year peak stream flows with 10' high tides, and 6 days of a 'second storm' that includes 100-year peak stream flows and stormwater runoff rates combined with 12' King Tides

Figure 10 shows the streamflow inputs for the February 2019 Calibration event. Figure 11 shows the streamflow inputs for the 100-year storm scenario. Figure 12 shows the downstream tidal boundary condition water level in Sand Lake used in the 100-year storm scenario, scaling up tides to reach 12' King Tide levels. Figure 13 shows the sample point locations within the 2D model domain where data was sampled for display on the water level and velocity plots.

Following the proposed dike breach, water levels in Beltz Marsh north of the proposed setback dike are expected to match the high and low water levels in Sand Lake, ranging between the low water level of 5.5' up to approximately 12' (HMT based on NOAA tide station analysis is 11.7', while a maximum water level of 11.97 was measured onsite in Sand Lake). This will increase the peak tidal water levels by approximately 1.5' compared to existing conditions, increasing the inundation extent around the perimeter of the marsh. All areas subject to increased inundation extent are within the SSSNA, mostly in areas of existing wetlands. The Reneke Creek crossing of Sandlake Road will also experience the increased peak tide elevations, as the existing culvert and proposed channel invert elevation is 9.5'. The existing roadway elevation at this crossing is approximately 16' and currently overtops during flood events due to high flows on Reneke Creek, not due to backwater from Beltz Marsh. Under proposed future conditions along with replacing this culvert with a new 49-foot-span bridge, Sand Lake will also be raised by 2 feet to an elevation of 18' to provide at least 2 feet of freeboard beneath the bridge during 100-year flows on Reneke Creek.

Water levels upstream/south of the setback dike will be controlled by the operations of the proposed tide gate structure and the continued presence of the main beaver dam.

  - Tidal water levels between the setback dike and the beaver dam (within the excavated tide gate drainage channels) are expected to rise and fall between 5.5' (minimum water level) and 8' (allowed by the proposed MTR).
  - Water levels south of the beaver dam will not experience much change under proposed conditions because they are already impounded between 7.5' (summer) and 8.5' (winter) due to the existing beaver dam. Operating the MTR to allow 8.0' water levels upstream of the tide gate will match the existing elevation of the beaver dam. Inundation extents will generally be limited to within the existing channels and shallow inundation within the vegetated marsh (maintaining the existing conditions).

- During storm events, runoff from No Name Creek and Tierra Del Mar will accumulate behind the setback dike during high tides above 8.0' when the tide gate is closed. However, hydraulic modeling of typical tides, 2-year storms, 50-year storms, and a 100-year storm event coupled with King Tides demonstrates that peak water levels will be reduced under proposed conditions and water levels will recede much faster due to increased conveyance capacity of the proposed tide gate system and the free-draining breach channel through the existing dike. The 100-year storm modeling results comparing proposed conditions to the existing conditions are displayed in Figure 14.
- These modeling results support ESA's No Rise Certification submitted to Tillamook County Planning Department as part of the project's Floodplain Development Permit application. The proposed setback dike location, tide gate structure characteristics, and water level management plan are designed to maintain water current levels under typical (non-storm) conditions and reduce peak water levels and inundation duration during storm and peak tide conditions.

4 – DIKE BREACH CHANNEL AND PEDESTRIAN BRIDGE

At the north/downstream end of the project site, all flows from the three tributaries of Reneke, Beltz, and No Name Creek ultimately drain to Sand Lake through the existing Beltz Dike via a 4'x4' concrete box culvert with a deteriorated tide gate. The SSTW project proposes to restore tidal connectivity and fish passage from Sand Lake into Beltz Marsh by excavating a large breach channel through the dike. The dike breach was located to connect to the apparent historical primary tidal channel, which is still visible on the inboard and outboard side of the dike, as shown in Figure 15.

In order to maintain recreational access to the SSSNA trail network via the Beltz Dike Trail which runs along the top of the existing dike, the dike breach would need to be spanned by a pedestrian bridge. Other alternatives including full dike removal, a boardwalk system with multiple bridge crossings, and replacing the existing tide gate with a larger modern tide gate system were evaluated prior to selecting the preferred alternative of a dike breach with a pedestrian bridge. Figure 16 shows a conceptual plan for breaching Beltz Dike, constructing a 140-foot pedestrian bridge over the breach channel, and removing the existing box culvert and reconstructing the dike embankment.

The breach channel was sized to exceed the minimum expected tidal inlet size based on tidal channel design guidelines (a channel with a 32-foot bottom width and 80-foot top width at 12' elevation). This channel size is similar to the width of the remnant historical channel outboard of the dike breach location, visible in aerial photos or during low tide when the broader inundation of Sand Lake narrows to reveal the channel form approximately 35 feet wide. Active Channel Width (ACW) measurements of the existing channels upstream and downstream of the dike breach are not applicable because the dike breach channel will connect to open water areas in Sand Lake and Beltz Marsh, where daily high tides occupy wetted widths of nearly 1000 feet downstream in Sand Lake and 500 feet in Beltz Marsh. For reference, the cumulative ACW for the three creeks that drain into Beltz Marsh add up to approximately 25 feet, but the dike breach channel will also convey daily tidal flows of water filling and draining from Beltz Marsh.

Subsequent hydraulic modeling and alternatives analysis led to the selection of a 140-foot-wide dike breach channel spanned by a 140-foot span pedestrian bridge. These dimensions were optimized by using the 2D HEC-RAS model to evaluate the peak velocities and shear stresses that would occur during the peak drain out of the 100-year storm event with 12' King Tides. Figure 17 shows a comparison of modeled peak velocity and shear stress for 100-, 140-, and 200-foot-span alternatives.

For the selected 140-foot-span dike breach and bridge alternative, peak velocities through the dike breach remain below 2 feet per second (fps) at all times, even during the moment of peak drain out/outflow during the

100-year storm + King Tides scenario (1.8 fps). During typical tidal conditions, velocities within the breach remain below 1 fps. During 2-year flows, peak velocities reach 1.5 fps. Figure 18 shows modeled velocities through a 100-foot dike breach, which only resulted in peak velocities exceeding 2 fps during the height of the 100-year storm drain out.

Peak shear stresses also remain relatively low, at 0.08 lb/ft2 (psf). These shear stress values are expected to be capable of eroding fine sediments but not gravels or riprap. Given that the existing substrate in the Sand Lake consists of sand and fine sediment, it is possible for the native sediment to be eroded. However, with the size of the designed breach channel and alignment with historical primary channel path, we would expect the channel to be relatively stable in the proposed location. We would prefer to maintain natural sediment within the beach channel bed and banks, while reasonably protecting the substantial investment of the new bridge. We propose to utilize buried riprap armoring around the base of the bridge abutment pile caps, covered in a minimum of 2 feet of native sediment and topsoil. The areas above 5.5' and capable of supporting vegetation will be planted with native herbs and shrubs above the 5.5' low-water elevation. A full revegetation plan is included in the 90% Plans.

The proposed dike breach and pedestrian bridge will restore full fish passage from Sand Lake into Beltz Marsh. Figure 19 shows the dike breach channel grading plan from the 90% Plans. Figure 20 shows the dike breach channel section view and elevation view of the 140-foot pedestrian bridge. The 90% Plans show more detail of the grading plan, channel alignments and dimensions, structural details, and construction logistics for building the pedestrian bridge and breaching the dike.

5 – TIDE GATE SYSTEM DESIGN

5.1 - Tide Gate System Design Requirements

The new tide gate will require approval by Oregon Department of Fish and Wildlife (ODFW) and comply with requirements in the Tidal Areas Restoration Program (TARP), a programmatic biological opinion prepared by the National Marine Fisheries Service (NMFS). The final design and operation of the new tide gate system will also seek to avoid impacts on the private properties in Tierra Del Mar, south of the proposed setback dike.

Relevant excerpts from OAR 635-412-0035 state:

(2) "Requirements for fish passage at dams and other artificial obstructions which create a discontinuity between upstream and downstream water surface or streambed elevations are:" … "Fishway water velocities shall: (A) Range between 1 and 2 feet per second in transport channels;" … "and (C) Not exceed 8 feet per second in discrete fishway transitions between the fishway entrance, pools, and exit through which fish must swim to move upstream, including but not limited to slots, orifices, or weir crests." … "(e)In fishway locations through which fish must swim, water depths shall be a minimum of 6 inches where only juveniles require passage and 12 inches where adults require passage…"

- (4) "Requirements for fish passage at artificial obstructions in estuaries, and above which a stream is present, are:" … "Tide gates and associated fish passage structures shall be a minimum of 4 feet wide and shall meet the requirements of OAR 635-412-0035(2) within the design streamflow range and for an average of at least 51% of tidal cycles, excluding periods when the channel is not passable under natural conditions.
- (5) "Requirements for fish passage at artificial obstructions in estuaries, floodplains, and wetlands, and above which no stream is present, are:"… "(b) Upstream Fish Passage: a fishway or road-stream crossing structure with or without a tide gate shall be provided during the period determined by the Department if there is current or historic native migratory fish spawning or rearing habitat within the estuary, floodplain, or wetland area impounded by the artificial obstruction."

The design team has consulted with local ODFW biologists and state fish passage staff, including email coordination, a virtual meeting on February 5, 2025, and subsequent site visit to review the No Name Creek area on February 10, 2025. In an email from Assistant District Fish Biologist, Mike Sinnott, on January 22, 2026, ODFW confirmed that, consistent with ODFW fish distribution layers, ***No Name Creek is a cutthroat-only stream that would not support adult salmon use. There is a possibility that juvenile coho could utilize the lower portion of No Name Creek for foraging once passage is improved, but the habitat is not suitable for adult coho or steelhead. The proposed tide gate and associated beaver exclusion system do not need to be sized for adult salmon. (ODFW, 2026)***

While the No Name Creek and Beltz Marsh area upstream of the setback dike are likely not suitable for adult salmon and there may limited use by juvenile coho or steelhead, this voluntary restoration project's goals are to provide the maximum fish passage improvements feasible within the constraints of the site context.

5.2 - Tide Gate Modeling Methodology

ESA utilized a combination of statistical analysis of tidal water level data and extensive 2D HEC-RAS modeling to:

- determine the appropriate sizing of the proposed tide gate system,
- demonstrate that water velocities inside the tide gate structure (fishway) and tide gate threshold (transition) meet ODFW fish passage velocity criteria, and
- estimate the % of time that the tide gate structure would be open and meeting fish passage criteria.

Following guidance in the TARP Section 30b *Tide/Flood Gate Removal, Replacement, or Retrofit,* (Corps of Engineers, Federal Emergency Management Agency, and Federal Highways Administration, 2018), ESA:

- obtained 10-years of tide data from the nearest NOAA tide stations,
- correlated the 10-year time series of tide data to the project site location, and
- performed statistical analysis on 10-years of available tide data to develop monthly water level exceedance probability plots for each month of the year.

The SSTW project site and Sand Lake are located between the NOAA tide gages at South Beach, OR (9435380) and Garibaldi, OR (9437540). Tidal levels, reported relative to NAVD88, were downloaded at a 6-minute interval for both NOAA gauge sites. The two tidal water level time series were interpolated based on the relative distance from each of the NOAA tide stations to develop a tidal time series near the mouth of the Sand Lake. The time series was then shifted to best fit the peaks and troughs of the water levels observed outside the existing Beltz Dike tide gate in Sand Lake. A linear regression relationship was developed from the composite NOAA tidal time series and the monitored water levels in Sand Lake. However, the regression relationship was not found to have a good fit due to the significant muting of the low tide in Sand Lake due to bed features that prevent full drain out and prevent water levels from falling below 5.5' at the location of the existing dike.

The tidal datums reported for the South Beach and Garibaldi NOAA tide stations are summarized in Table 1, along with the interpolated tidal datums and the mouth of Sand Lake and the effective tidal datums at the existing dike due to the muted low tide.

Table 1. Tidal datum elevations at the project site.

Elevation (feet, NAVD88)	Garibaldi	South Beach	Sand Lake (mouth)	Sand Lake at Dike
Highest Measured Tide (HMT)	11.75	11.66	11.7	11.7
Mean Higher High Water (MHHW)	7.99	7.57	7.9	7.9
Mean High Water (MHW)	7.28	6.87	7.2	7.2
Mean Tide Level (MTL)	4.16	3.74	4.0	5.5*

Mean Low Water (MLW)	1.04	0.61	0.9	5.5*
Mean Lower Low Water (MLLW)	-0.33	-0.77	-0.5	5.5*
Lowest Measured Water Level	-3.80	-4.11	-3.9	5.5*

Note: *The water level in Sand Lake at the location of the existing Beltz Dike never falls below 5.5' due to controlling sand bed features in the lake that impound water and prevent full drain out.

The 10-year time series of tidal water level data does not account for periods where water levels in the system are elevated due to high stream flows or drainage limitations during the wet months. Under high stream flow scenarios, water levels at the site may remain higher than modeled tidal levels, which could result in the tide gate remaining closed for longer periods of time due to elevated water levels. However, we expect that the proposed dike breach channel and central drainage channel will provide adequate drainage to allow water levels to fall to the minimum 5.5' elevation during all low tides in Sand Lake.

ESA modeled culvert hydraulics, water levels, and velocities through the tide gate structure and dike breach channel using a 2D HEC-RAS model of the project's proposed conditions. Extensive modeling of a range of project alternatives, including a range of dike breach widths, setback dike locations, and tide gate operations (MTR water level settings) is documented in ESA's ***Sitka Sedge Tidal Wetland – Hydraulic Analysis Memorandum*** (ESA, 2024). The HEC-RAS modeling included a series of hydrologic scenario simulations including:

- a calibration event modeled after an actual storm event in February of 2017, which included a period of typical winter tide conditions and moderate stream flows followed by a storm event that generated 1-year stream flows and stormwater runoff
- a long-duration, 14-day hydrologic scenario that includes 3 days of typical wintertime conditions, 5 days of a 'first storm' using 2-year peak stream flows with 10' high tides, and 6 days of a 'second storm' that includes 100-year peak stream flows and stormwater runoff rates combined with 12' King Tides.

5.3 - Tide Gate Modeling Results

MTR Water Level Setting

In order to meet ODFW's criteria for the tide gate to be open 51% of the time, upstream/inboard water levels would need to be allowed to reach approximately 4.5 feet NAVD88, based on mean tide level calculated for Sand Lake mouth. Because of the elevated/muted low tide level in Sand Lake at Beltz Dike, with water levels never falling below 5.5', the tide gate would need to stay open any time water levels were at this minimum water level of 5.5'.

At the proposed location of the setback dike and tide gate structure, water levels of up to 8.0' could be allowed to occur without substantially changing inundation extents or causing any negative impacts to adjacent properties or infrastructure. A proposed MTR water level setting of 8.0' was deemed acceptable by OPRD in earlier stages of alternatives analysis, and the 8.0' MTR setting has been incorporated in hydraulic modeling of the project since the 30% Design stage.

This approach of essentially maintaining the typical water level conditions at the south end of Beltz Marsh is supported by OPRD and project stakeholders, and an 8.0' MTR setting would easily meet ODFW's requirement for keeping the tide gate open for more than 51% of the time. Using 10 years of available NOAA tide station data, analyzed by each month, ESA used a MATLAB model to predict the percentage of time the tide gate would be open during each month, and the full year, under a range of MTR water level settings from 7.0' to 10.0'. These results are presented in Table 2. Using a 9' or 10' MTR setting would essentially cause the tide gate to stay open nearly 98-100% of the time, but these baseline water levels would increase daily inundation extents compared to existing conditions and could lead to expanded inundation extents on adjacent private properties during flood events.

Tidal water level frequency analysis and model results predict that a tide gate with the proposed MTR water level setting of 8.0' NAVD88 would be open for 90.7% of the time (for the full year), with monthly values ranging from 83.5% in January, up to 93.6% of the time in June and July.

Table 2. Percent of time meeting fish passage criteria by month (% of time tide gates will be open under each MTR water level setting between 7' and 10' NAVD88

MTR Water Level Setting (ft, NAVD88)	FULL YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
7	79.7	71.8	76.8	76.9	82.6	84.8	84.5	83.7	81.8	81.3	80.9	77.2	74.1
8	90.7	83.5	88.3	88.8	93.1	94.2	93.6	93.6	92.8	92.8	92.2	89.1	86.5
9	97.2	93.2	96.4	97	98.1	98.3	98.4	98.6	98.8	98.7	97.9	96.2	94.4
10	99.5	98.3	99.7	99.6	99.8	99.9	99.8	100	100	100	99.7	99	98.6

*Assuming velocity and water depth fish passage criteria are met, which will be met with a single 8' tall x 10' wide tide gate with Invert Elevation = 5.0'.

Velocities Through Tide Gate Structure

For the proposed conditions as depicted in the 90% Plans (a tide gate structure with a 12-foot-span box culvert and a single 8' tall x 10' wide side-hinged tide gate, with a 5.0' invert elevation, controlled by a Muted Tidal Regulator system operated with an MTR water level setting of 8.0'), the modeling results show that velocities through the tide gate structure will remain below 2 feet per second during all typical tidal conditions. Short duration peak velocities exceed 2 feet per second (fps) only when passing 1-year and 2-year stream flows and stormwater runoff, with peak velocities reaching as high as 2.3 fps through the fishway and up to 2.8 fps through the gate threshold transition. During extreme flood conditions (100-year flows and stormwater runoff coupled with 12' King Tides), modeled peak velocities through box culvert and tide gate threshold reached 3.7 fps and 4.5 fps, respectively.

Figures 21 and 22 show water levels immediately inside and outside of the tide gate structure, as well as modeled velocities inside the box culvert/fishway and at the tide gate threshold/transition during the February 2017 model calibration event that included typical winter tide conditions followed by a 1-year storm event. Figure 23 shows water levels and velocities at the tide gate structure during the extreme flood model scenario including a 2-year storm followed by a 100-year storm event coupled with King Tides.

This modeling was conducted with some assumptions of typical conditions (e.g., frictional losses at culvert entrances, optimal MTR performance, no leakage through tide gates). Actual performance of the tide gates will vary somewhat from these conditions when site-specific factors influence inboard water levels and flow velocities. Minor adjustments to the tide gates to avoid inundation on the neighboring properties may be necessary. These adjustments could slightly modify actual time of fish passability from the predicted passability percentages.

5.4 - Tide Gate System Design Plans

The proposed setback dike and tide gate system are depicted in detail in the 90% Design Plans. Figure 24 shows an excerpt of the setback dike plan view. Figure 25 shows an enlargement of the tide gate structure grading plan. Temporary culverts will be installed in the existing central ditch channel to provide stream bypass while the setback dike and tide gate structure are under construction. Once the new tide gate structure is complete, the drainage channel will be excavated to realign the channel through the new tide gate structure. Figure 26 shows profile views of the tide gate structure drainage channel and the beaver exclusion system.

6 – WATER MANAGEMENT PLAN FOR TIDE GATE STRUCTURE

Operating Protocol:

We propose a single, year-round MTR water level setting of 8.0' NAVD88. No seasonal adjustment is intended.

This water level setting is intended to maintain daily peak water levels upstream/behind the tide gate at 8.0' NAVD88, in the absence of stormwater runoff inflow or flood flows. This water level setting is intended to result in maximum water levels of 11' NAVD88 during periods of heavy rainfall and stormwater runoff, based on hydraulic modeling of extreme storm conditions (100-year flows coupled with King Tide water levels).

Responsible Party:

Oregon Parks and Recreation Department (OPRD) is the property owner of Sitka Sedge State Natural Area and will be the primary party responsible for determining if tide gate water level setting adjustments are needed. If adjustments are required, they will be performed by the tide gate system manufacturer, authorized contractor, or OPRD staff trained by the tide gate manufacturer. If a seasonal adjustment routine were ever established, OPRD staff will be trained by the tide gate system manufacturer to make seasonal adjustments.

Initial water level settings and fine tuning:

Upon completion of construction, the tide gate manufacturer/contractor will be required to tune the tide gate system's water level controls to meet the designed upstream water level of 8.0' NAVD88 during dry, summer-time conditions (no stormwater runoff inflow). OPRD, or an authorized representative, and the project engineer will observe inundation levels upstream of the tide gate structure and confirm no unintended inundation is occurring on the adjacent private property. To aide in visual water level observations, a staff gage will be installed on either side of the tide gate (inside and outside), mounted on the concrete wing walls, and surveyed in to reflect accurate elevations in NAVD88 vertical datum.

Monitoring protocol:

In addition to periodic on-the-ground observation of water levels and inundation extents, water level monitoring instrumentation will be installed inside and outside of the tide gate structure to record water levels occurring upstream and downstream of the setback dike/tide gate. OPRD, or TEP monitoring staff, will retrieve water level data every six months, at a minimum, perform barometric compensation, and plot water level data for review. Water level data will be reviewed and analyzed to document daily peak water levels inside the tide gate and peak water levels during storm events. These water levels will be utilized along with LiDAR and/or as-built terrain surfaces to estimate inundation extents. The data will be analyzed to estimate the % of time the tide gates are open during each month throughout the year.

Reporting Protocol:

During the first three years following construction, OPRD, or an authorized contractor, will prepare annual reports on the tide gate system's performance. The reports will include short narratives, photos taken from upstream and downstream of the tide gate structure, plots of water level data, and tables summarizing the performance including estimated % of opening time.

Adaptive Management Plan:

Monitoring data and observations described above will be analyzed by OPRD, or an authorized contractor, and will be utilized to make recommendations for maintaining the current operating protocol or adapting the operating protocol to improve fish passage or manage water levels/inundation extents to protect neighboring property.

If the tide gate system's fish passage performance is not meeting the % passability goals outlined in the approved fish passage plan, OPRD will reassess the MTR water level setting to improve fish passage, as long as this adjustment does not cause adverse effects on neighboring property.

If water levels on adjacent properties, or subsequent permission from adjacent landowners, allow for increasing the MTR water level setting, the water level setting may be increased in 0.25-foot increments while observing the effect over a series of high tides. If water levels (daily tidal levels or water levels during storm events) result in inundation, or salinity intrusion, on neighboring property, the water level setting may be decreased. In this event, ODFW will be consulted for approval of any required adjustments.

7 – BEAVER EXCLUSION MEASURES AT TIDE GATE STRUCTURE

The proposed setback dike and tide gate structure are located in an area of existing beaver activity. The main beaver dam located approximately 50 feet south of the proposed tide gate structure has been present and persistent for the last 10 years or more, since the early stages of project planning and hydraulic analysis. Other smaller beaver dams have been observed throughout the project area, including beaver dams in the central ditch north of the proposed setback dike and beaver dams in the Reneke Creek, Beltz Creek, and No Name Creek flow paths. We expect that the main beaver dam and other beaver activity in the project area will persist after the dike breach and other restoration actions are completed.

In order to protect the proposed setback dike tide gate structure and MTR control system, and to prevent the tide gate structure from being blocked by beaver activity or debris from a beaver dam blow out, we propose to construct a beaver exclusion system at the tide gate structure consisting of:

- a 6"x6" mesh fencing across the channel downstream of the tide gate structure (downstream exclusion fence), with 6"x6" mesh fencing on the channel bottom to prevent beaver from digging under the fence,
- a 6"x6" mesh fencing across the channel upstream of the tide gate structure (upstream receiver fence)
- a 6"x6" mesh box with internal 6"x6" mesh baffles (square fence filter) to allow water to pass through and into a 24-inch diameter bypass pipe that penetrates the upstream receiver fence, and
- chain link fencing along the top of the tide gate structures headwalls and wingwalls (at the top and side slopes of the setback dike embankment), with gates to provide human access into the fenced area for maintenance.

The proposed beaver exclusion system at the tide gate structure is depicted in Figures 27 and 28. Additional details and notes describing the beaver exclusion system are shown on Sheet D4.1 of the 90% Design Plans.

OAR 635-412-0035, states:

(C) Beaver exclusion culvert protection devices shall:

(i) Allow for easy maintenance and debris removal;

(ii) Be maintained, monitored, and cleaned as necessary to provide fish passage;

(iii) Have a minimum clear space between vertical and horizontal members of 6 inches when only resident trout, Entosphenus and Lampetra species (lamprey) species are present;

(iv) Be approved on a case by case basis in areas with salmon, steelhead, bull trout, or other large bodied species.

As previously discussed, ODFW has communicated that No Name Creek is a cutthroat-only stream. Based on fish use data and on-the-ground observations, No Name Creek and the beaver-impounded wetland area south of the setback dike and upstream of Sandlake Road are not suitable for adult salmon use. Therefore, 6-inch spacing of vertical and horizontal members of beaver exclusion elements/fencing may be used at this site.

The beaver exclusion system is designed to be easily maintained with access to both sides of the exclusion fencing. The setback dike will have a gravel access road on top, providing vehicular access to the tide gate structure for monitoring and maintenance, including access by an excavator or vacuum truck in the event that such equipment is needed to perform maintenance or removal of mud or debris.

OPRD staff are present at the SSSNA most of the time, with the Park Manager's residence located within the park. OPRD staff will frequently monitor and maintain the tide gate and beaver exclusion system (as further described in Section 9) to maintain hydraulic function and fish passage.

8 – BACKUP OVERFLOW CULVERTS SEEKING FISH PASSAGE EXEMPTION

In addition to the main tide gate structure through the proposed setback dike, the project design includes two 36" diameter culvert pipes passing through the setback dike to provide emergency relief in the event that the main tide gate structure was blocked by beaver or other reasons.

The main tide gate structure is designed to have adequate hydraulic capacity to provide drainage of peak flows from No Name Creek and stormwater runoff from Tierra Del Mar during a 100-year storm event. However, stakeholders have expressed concern that even with a beaver exclusion system described in Section 6, they would feel more comfortable if a backup drainage system were in place in case the tide gate structure became blocked by beaver damming or associated debris.

The main tide gate structure has an invert elevation of 5.0'. Modeling of peak flood events indicate that water levels south of the setback dike will rarely exceed 10.0' except during extreme events such as a 100-year storm/streamflow event, which would lead to short-duration peaks up to 11.0'. The backup culverts are designed with invert elevations of 10.5' (upstream end) and 9.5' (downstream end), such that they will only convey flow during the largest or storm events or under emergency conditions if water levels in the upstream marsh exceed 10' elevation.

The backup culverts are designed in accordance with US Army Corps of Engineer's Engineering Manual EM1110-2-1913 Design and Construction of Levees, including appropriate bedding and backfill. The upstream and downstream ends of the culvert pipes will include concrete headwalls with riprap armoring around the headwalls and at the pipe outfalls.

The upstream headwall/pipe inlet will be protected with a metal trash rack with 9-inch bar spacing. Because the pipes will be elevated above typical water levels, we do not anticipate significant pressure from beavers to block the backup culverts or build dams near them, as they will only convey flow during extreme storm events or if the main tide gate structure were blocked for some reason.

The downstream ends of the backup culverts will include traditional top-hinged flap gates to allow for one-way drainage of water, from upstream to downstream. The top-hinged flap gates would reliably prevent tide water from flowing upstream through the backup culverts into the area upstream of the setback dike. Figure 29 shows the plan view locations of the two backup culverts and a profile view of one of the backup culverts.

Due to the infrequent conditions when water will pass through the backup culverts and their intended function as emergency backup drainage features, we are seeking an exemption or waiver for meeting fish passage criteria at these crossings. Rationale for providing an exemption or waiver is outlined below.

OAR 635-412-0025 - Fish Passage Waivers and Exemptions states:

(1) The Commission (or Department as applicable) may grant exemptions from fish passage requirements at an artificial obstruction if it is determined that:

(a) A lack of fish passage has been effectively mitigated;

(b) The owner or operator has received a legal waiver for the artificial obstruction from the Commission or the Department; or

(c) There is no appreciable benefit to providing fish passage.

The lack of fish passage through the two backup culverts through the setback dike is mitigated by the establishment of an effective fish passage route through the setback dike via the main tide gate structure described in detail in Section 5. As documented herein, the main tide gate structure will provide fish passage approximately 91% of the time throughout the year by allowing fish passage at all times when tidal water levels are at or below 8' NAVD88 (just below MHHW) and only closing during peak tides exceeding this elevation.

The backup culverts are designed with invert elevations of 9.5' (downstream end) and 10.5' (upstream end), such that they will only convey flow during heavy storm events or under emergency conditions if water levels in the upstream marsh exceed 10.5' elevation. Furthermore, the area upstream of the setback dike, including No Name Creek which is characterized by ODFW as a cutthroat-only stream, is not suitable for adult salmon use. We do not believe providing fish passage through these elevated backup culverts would provide any appreciable benefit to fish passage through the setback dike.

9 – MAINTENANCE PLAN

This preliminary maintenance plan is intended to provide the information required in the TARP guidance, as well as for maintenance plan documentation required for Tillamook County Land Use permitting and a general understanding of the anticipated maintenance that will be required for ensuring operation and hydraulic capacity at the proposed stream crossing structures.

The tide gate manufacturer and installation contractor will be required to provide the final Operation and Maintenance (O&M) Plan with additional detail for inspecting, maintaining, and replacing specific components of the tide gate structure and Muted Tidal Regulator (MTR) mechanical system.

Responsible Parties:

Oregon Parks and Recreation Department (OPRD) is the property owner of Sitka Sedge State Natural Area (SSSNA) and will be the primary party responsible for maintaining the tide gate system and pedestrian bridge crossing over the Beltz Dike breach channel. Tillamook Estuaries Partnership (TEP) will assist with water level monitoring and periodic observations as agreed to with OPRD. Oregon Department of Transportation (ODOT) bridge inspectors inspect OPRD's pedestrian bridges every three years and provides inspection reports to OPRD.

OPRD will retain the tide gate manufacturer/installer, or authorized contractor, to periodically inspect the tide gate system components and repair or replace components as needed to maintain the intended function. The tide gate manufacturer, or authorized contractor, with specific knowledge and understanding of the MTR mechanical system will be retained for inspections at one year, five years, and 10 years after construction.

Monitoring Protocol, including frequency:

- **Setback Dike, Tide Gate Structure, and Pedestrian Bridge by OPRD**
  - ***During first year after construction:***
    - Monthly visual inspection of tide gate opening and closing: the tide gate should stay open when tidal water levels are below 8' NAVD88 and should close when tidal water levels upstream of the structure rise above 8' NAVD88. Tide gate visual inspections to

observe the tide gate's closing function should occur when tidal water levels are predicted to exceed 8'.

- Record water level observed on upstream staff gauge when gates close and note water mark of previous high-water levels, if present.
- Monitor extent of inundation at the south end of Beltz Marsh during high tides
- Conduct continuous water level monitoring inside (upstream) and outside (downstream) of the tide gate at 15-minute intervals.

- Monthly visual inspection of the beaver exclusion measures, including the exclusion fencing, flow device, and bypass pipe
  - Check for debris racking or beaver damming activity
  - Observe the location and orientation of the 'square filter' flow device and bypass pipe for stability, proper anchoring, displacement, debris, or damage
  - Observe the point where the bypass pipe enters the receiver fence for proper flow, blockage, or displacement

- ***Annual Inspection:***
  - Visual inspection of tide gate structure area:
    - tide gates and adjacent channel areas for debris, logs, sedimentation, or other potential blockages
    - MTR mechanical components for apparent damage, debris racking, or functional problems,
    - side slopes and wing walls for scour, rock displacement, or damage,
  - Visual inspection/monitoring of setback dike side slopes for erosion, vegetation establishment, signs of settlement
  - Visual inspection/monitoring of channels and interior of concrete structure for sedimentation or scour
  - Visual inspection of pedestrian bridge abutments, breach channel banks, and dike embankment side slopes in vicinity of pedestrian bridge
  - Visual inspection of beaver exclusion systems, including fencing, flow device, and bypass pipe
  - Inspections should occur during low tides, but high tide gate operations/closing should also be verified
- ***Following flood event*** (tidal water levels exceeding 11' NAVD88, or local rainfall exceeding 3 inches over a 48-hour period)
  - Visual inspection of setback dike side slopes
  - Visual inspection of tide gate area for scour, damage, or debris
  - Visual inspection of beaver exclusion system for damage, debris, or damming activity
- ***Remedies/Actions:***
  - Remove debris (logs, small woody debris racking, rocks mobilized to within 10 feet of tide gate doors). Hire excavator operator to remove larger debris or sediment if necessary.
  - Remove debris from beaver exclusion fencing and flow device square filter. Fix any damage or displacement of flow device square filter and bypass pipe.
  - Remove deposited sediment if affecting opening and closing of gates by hand excavation or mechanical dredging. Hire vacuum truck operator if necessary to remove sediment from the tide gate structure or immediate vicinity and conduct work in accordance with permit conditions including but not limited to work area isolation and fish salvage.
  - Report any apparent damage or functional issues of the MTR system to tide gate manufacturer for follow-up inspection

- By specialized tide gate contractor
  - *1 year, 5 years, 10 years post construction (update maintenance plan after 10 years)*
    - Full inspection and adjustment of MTR mechanical system,
      - Inspect Door Actuator
      - Inspect Linkage (telescoping arm, middle stationary linkage, and inner adjustable linkage arm)
      - Inspect Modulator (hinged counter weighted float tank)
      - Inspect tide gates, hinges, seals
      - Adjust MTR system as needed for optimum operation or adaptive management in coordination with OPRD.
        - Adjust stroke to optimize torque and gate opening width
        - Adjust MTR interior water level setting if needed
  - ***Remedies/Action:***
    - Adjust MTR system components as needed
    - Repair or replace damaged or worn parts
    - Remove debris in immediate vicinity of tide gate

Reporting Protocol

During the first three years following construction, OPRD, or an authorized contractor, will prepare annual reports on the tide gate system's performance. The reports will include short narratives, photos taken from upstream and downstream of the tide gate structure, plots of water level data, and tables summarizing the performance including estimated % of opening time. Annual tide gate performance reports will be provided to ODFW for review and comment.

Modification process if passage or habitat conditions fail to meet project goals

OPRD will coordinate with ODFW to evaluate whether the tide gate system's performance meets the project goals or if adjustments are necessary.

The tide gate structure has been sized using hydraulic modeling to meet fish passage velocity criteria and hydraulic conveyance capacity to provide adequate drainage of the basin upstream of the setback dike, resulting in the proposed interior structure span and tide gate widths. Modifying these characteristics after construction will not be feasible. The primary means of modifying the fish passage performance of the tide gate system after construction will be adjusting the MTR water level setting to increase the amount of time the tide gates stay open. As described under Adaptive Management, OPRD will fine tune the MTR water level setting (8.0 NAVD88 baseline) to maximize tide gate opening time while preventing inundation on the neighboring private property.



Photo 1 – Existing tide gate/culvert at the drainage connection from Beltz Marsh to Sand Lake during low tide



Photo 2 – Sand Lake looking north, outboard of the existing dike and tide gate structure



Photo 3 – Beltz Marsh looking south of existing dike and tide gate structure, low tide in summer



Photo 4 – Beltz Marsh looking south/inboard of existing tide gate, high incoming tide during winter

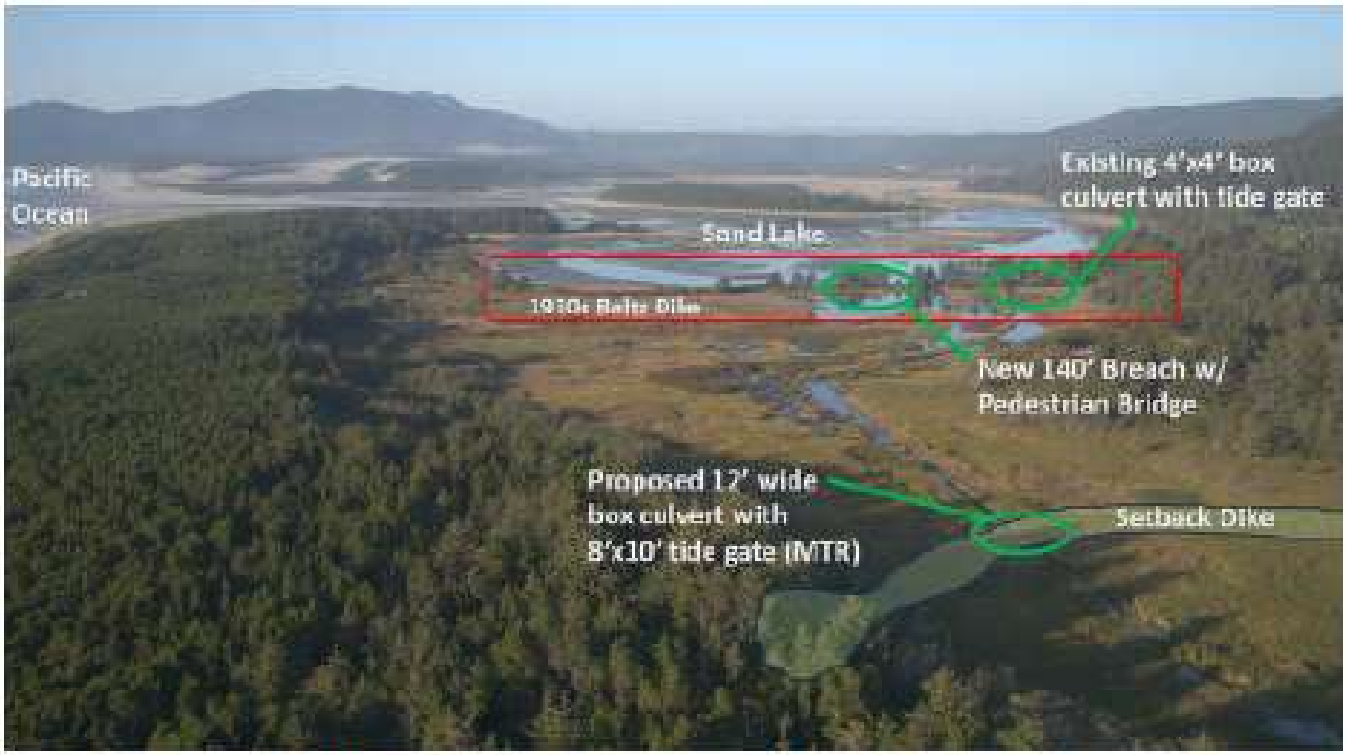


Photo 5 – Annotated aerial photo, facing north, showing existing dike and tide gate, proposed setback dike and tide gate structure in relation to Sand Lake and Pacific Ocean



Photo 6 – Annotated aerial photo showing existing Beltz Dike and location of existing culvert/tide gate and proposed dike breach location to be spanned by 140-foot-span pedestrian bridge



Photo 7 – Annotated photo showing approximate location of proposed setback dike across Beltz Marsh and existing main beaver dam upstream that backwaters south end of marsh



Photo 8 – Beltz Marsh south/upstream of proposed setback dike (facing north) impounded by existing main beaver dam





Photo 9 – Reneke Creek culvert crossing at Sandlake Road, location overtops causing embankment damage requiring repairs by TCPW

Beltz Creek at Sandlake Road

- 36-inch diameter culvert pipe
- 2.5-foot perched outfall
- 6-foot active channel width
- Downstream of culvert flows through old quarry pond area



Culvert inlet



Perched culvert outlet

Photo 10 – Beltz Creek culvert inlet and perched outlet at Sandlake Road crossing



Photo 11 – No Name Creek culvert inlet at Sandlake Road crossing, beaver exclusion cage on pipe inlet and permanently impounded wetland area upstream (private property within Tierra Del Mar)



Photo 12 – No Name Creek culvert inlet at Sandlake Road crossing, beaver exclusion cage on pipe inlet and permanently impounded wetland area upstream

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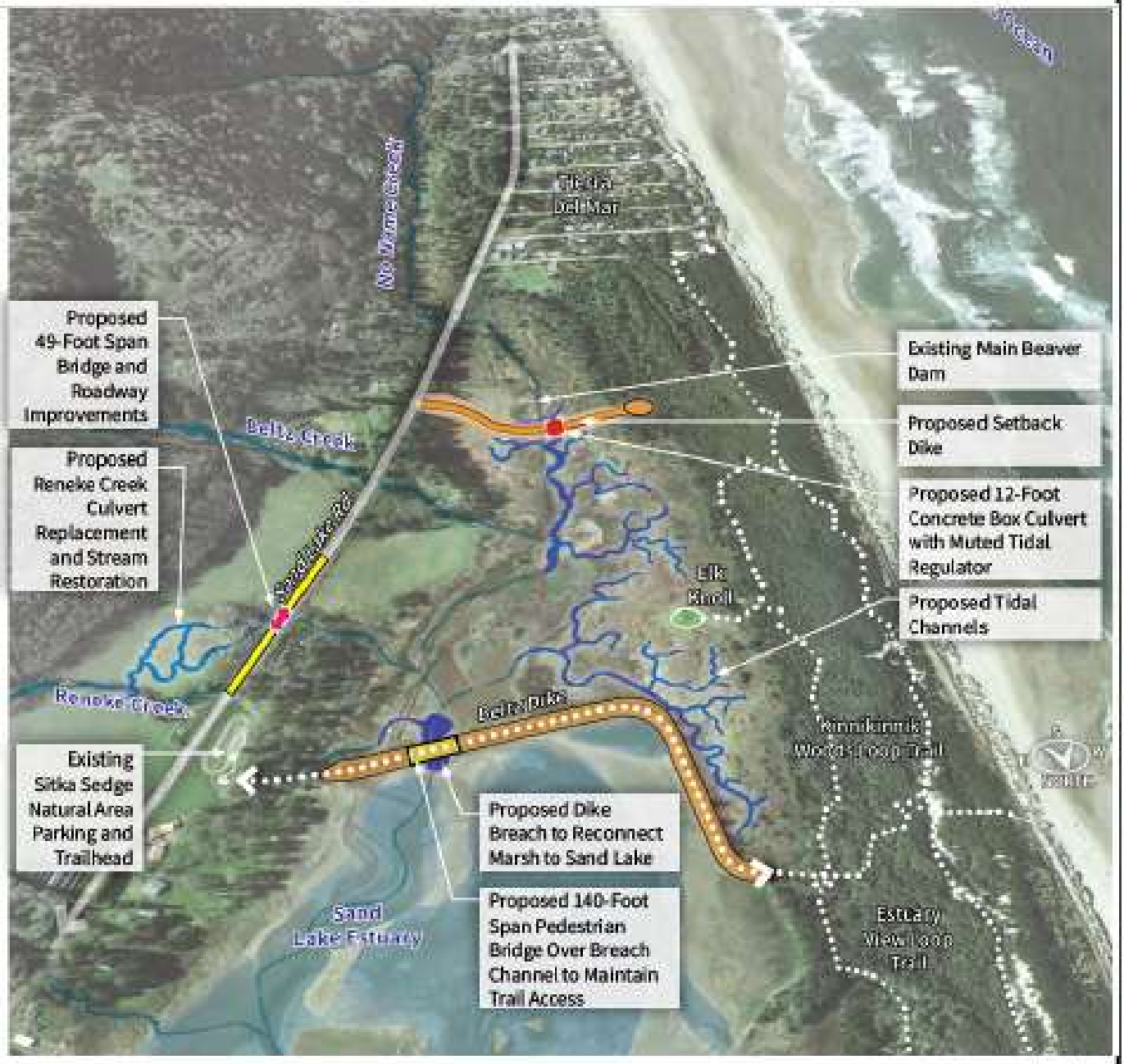
FEMA, 2013. Procedures for "No-Rise" Certification for Proposed Developments in the Regulatory Floodway. October 2013.

FEMA, 2020a. FEMA Region 10 FAQ: Rescinded Policy on Fish Enhancement Structures in the Floodway. August 17, 2020.

Leo Kuntz, 2024. Personal Communication. Nehalem Marine Manufacturing (MTR and tide gate manufacturer).

ODFW, 2026. Personal Communication. Various dates including email coordination, virtual meeting, site visit, and email from Mike Sinnott on January 22, 2026 confirming No Name Creek is a cutthroat-only stream that would not support adult salmon use.

APPENDIX A – FIGURES



Note: This figure shows an overview the proposed elements of the Sitka Sedge Tidal Wetland Restoration Project.

Figure 1
Sitka Sedge Tidal Wetland Restoration Concept Plan Overview

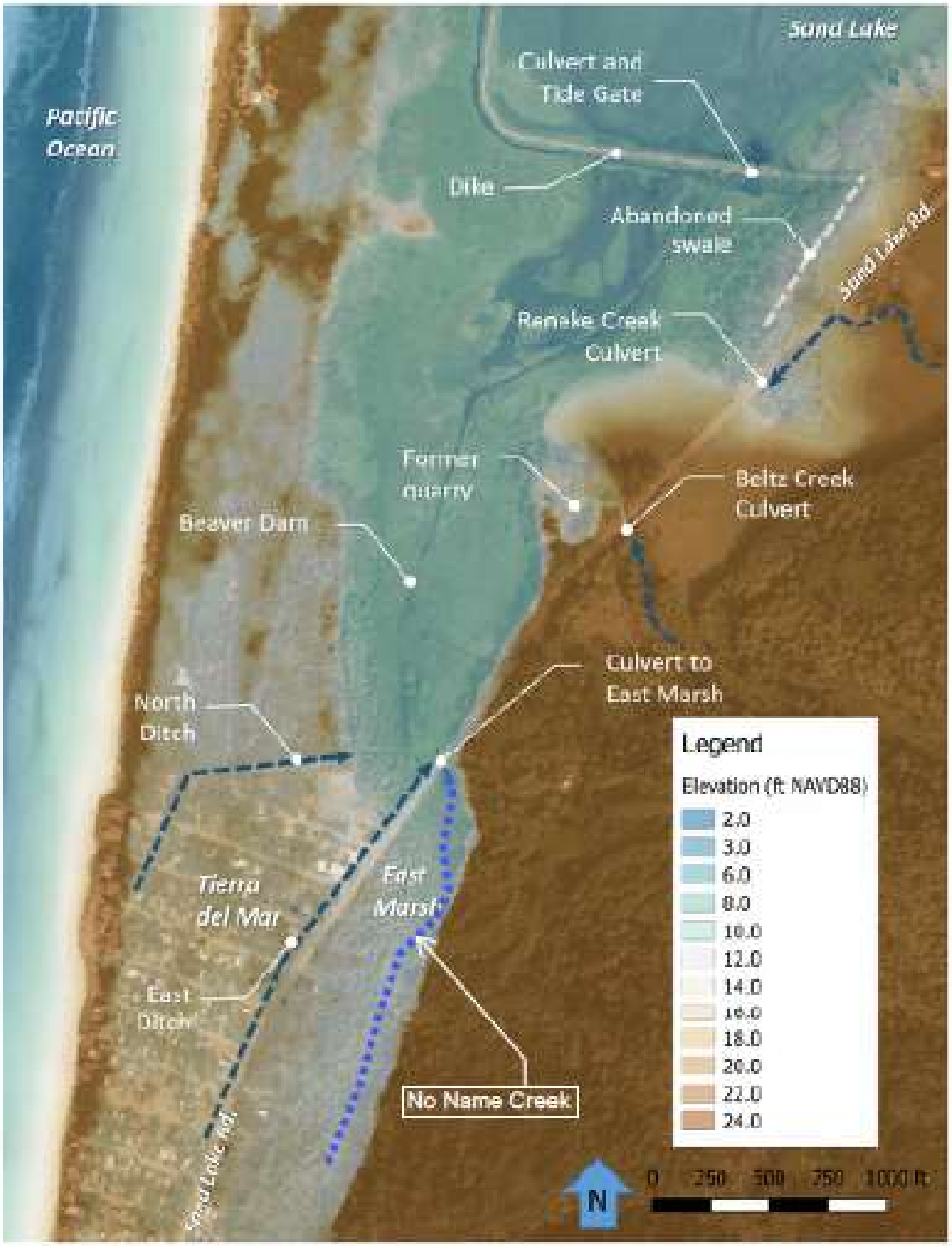


Figure 2
Project Location Map



Note: This figure is an aerial photo identifying key site features of the project area.

Figure 3
Annotated Site Map and Aerial Photo



Note: This figure is an aerial photo with a LiDAR elevation data overlay. Key site features are labeled for reference.

Figure 4
Annotated Site Topographic and Hydrologic Map

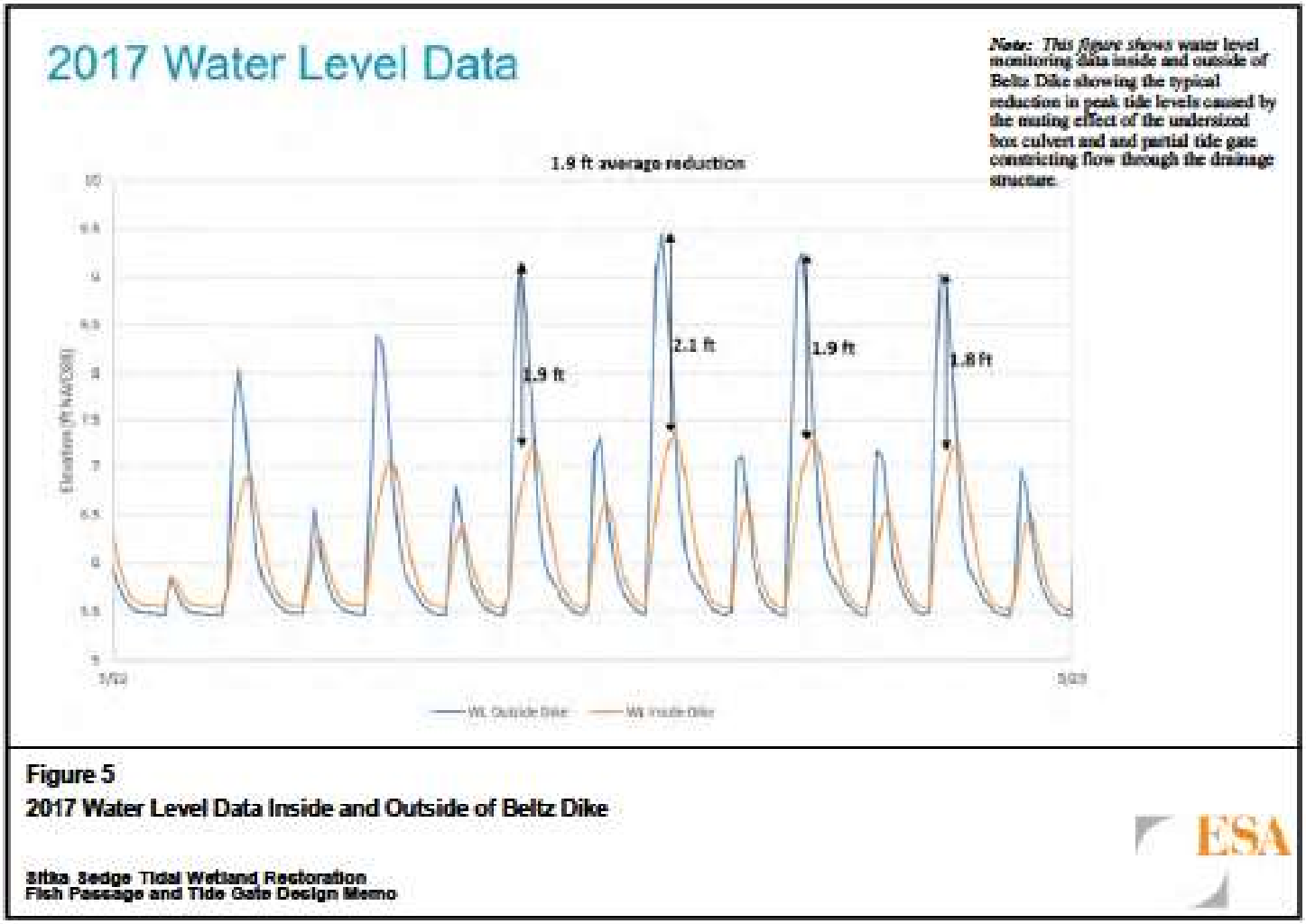


Figure 5
2017 Water Level Data Inside and Outside of Beltz Dike

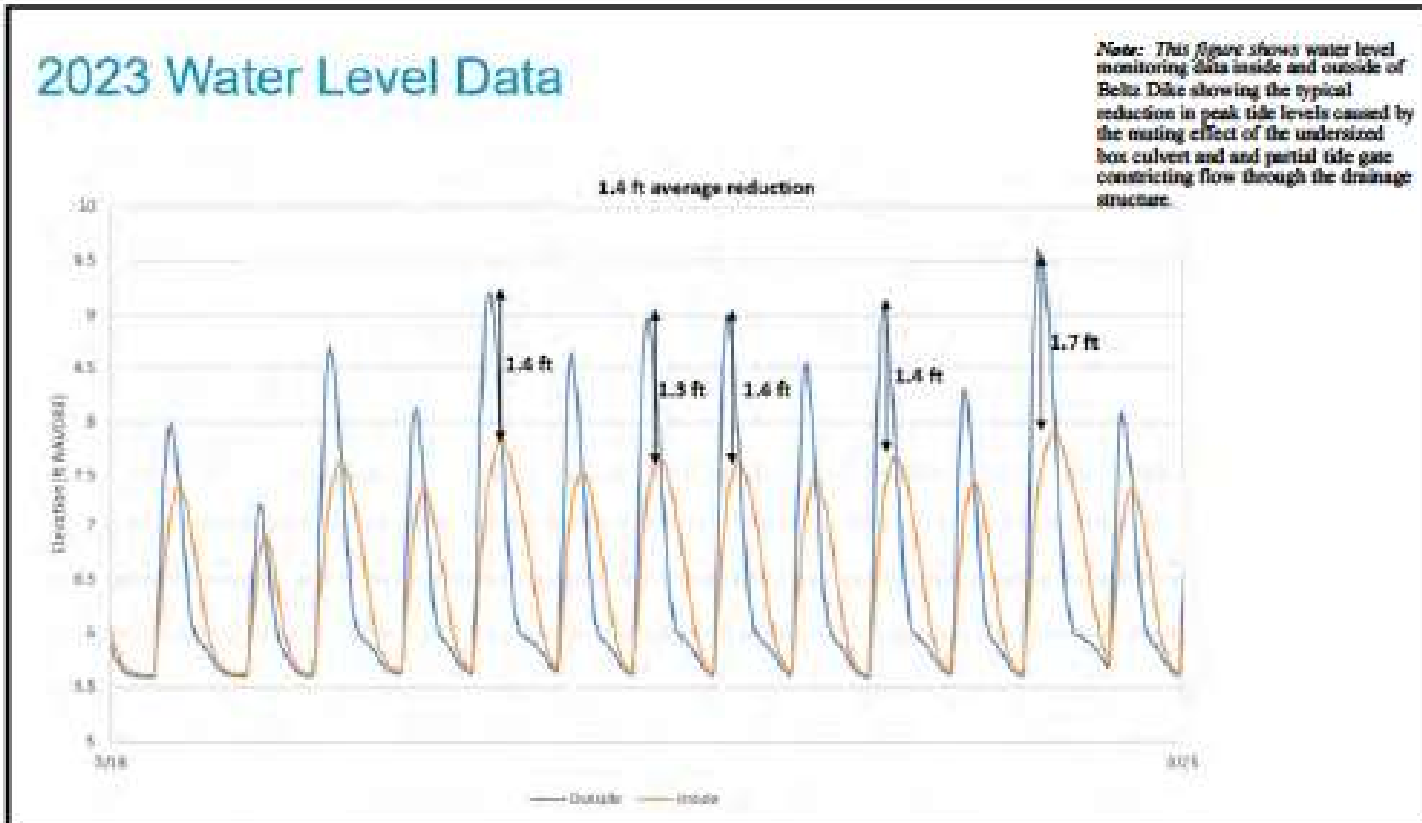
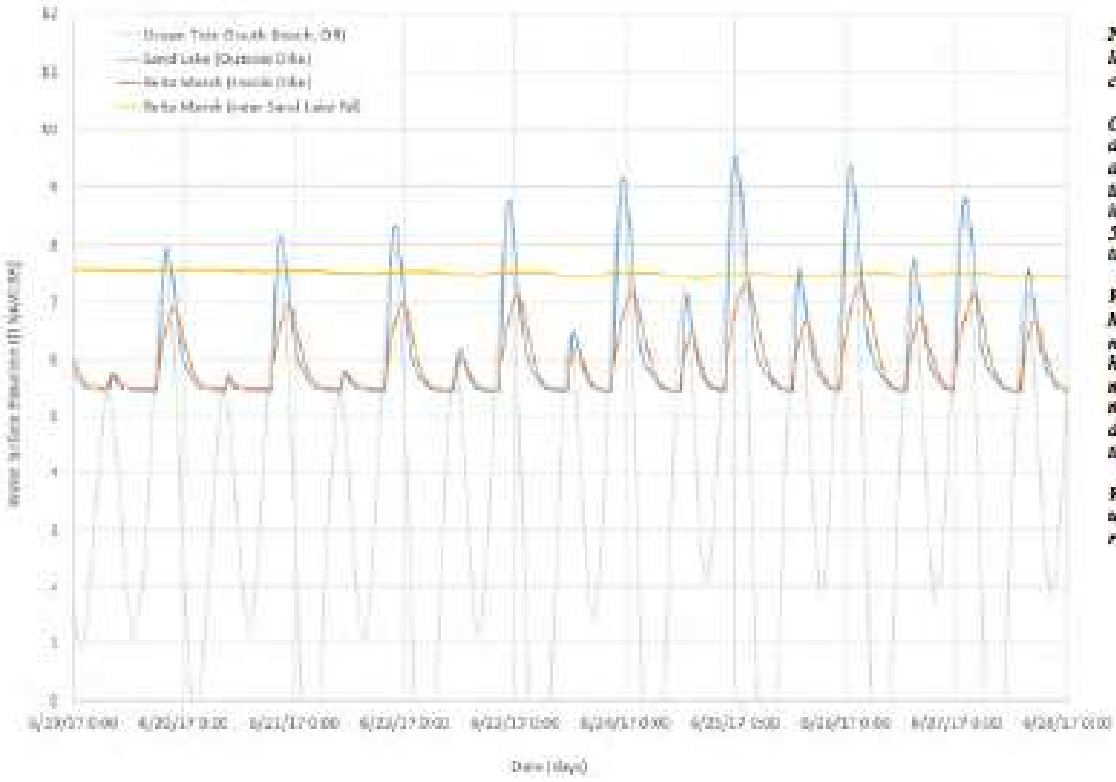


Figure 6
2023 Water Level Data Inside and Outside of Beltz Dike



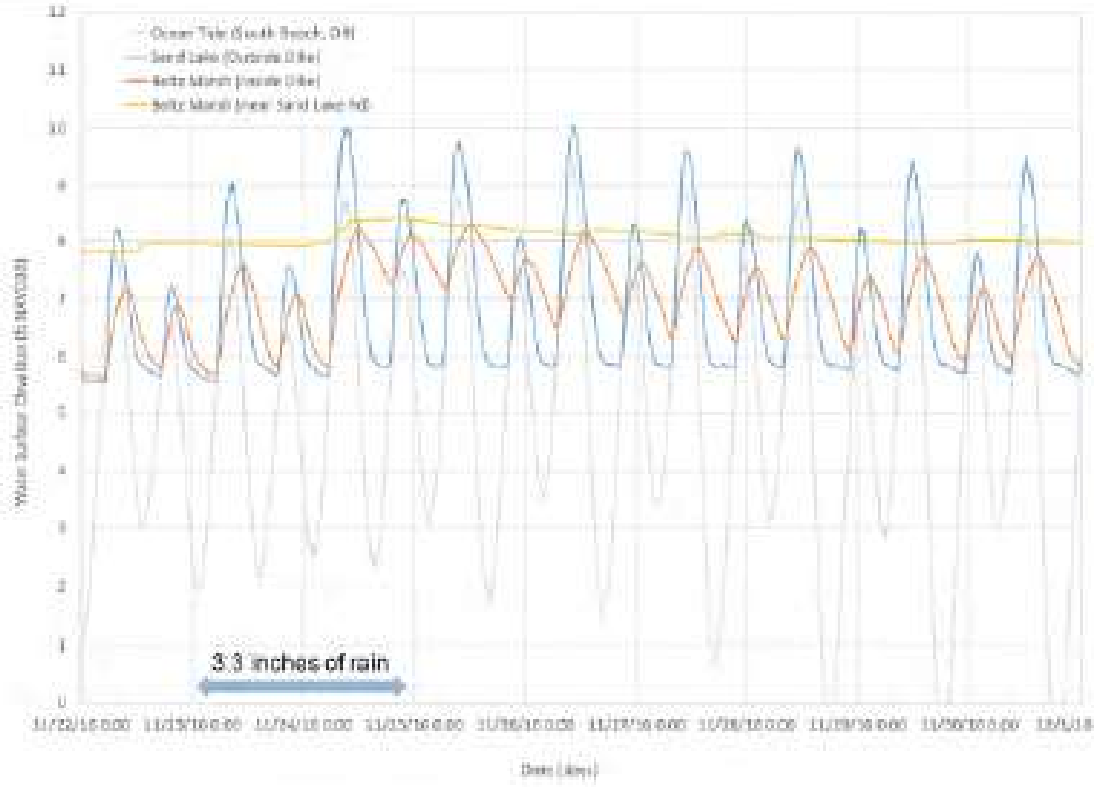
***Note:** This figure shows water level data collected during a typical period during the dry season.*

Ocean tides fluctuate between 0' and 9.5'. Sand Lake water levels are typically slightly higher than the ocean at high tide. Low tides in Sand Lake never fall below 5.5' due to controlling bed elevations.

Water levels inside the dike/ Marsh show a muted tidal connection. The tide gate prevents high tides from entering the marsh. Water drains out of the marsh as tides recede, slightly delayed due to limited capacity of the culvert.

Water levels at Sand Lake Road upstream of the beaver dam are relatively constant at 7.5'.

Figure 7
Typical Water Level Fluctuations in Beltz Marsh During the Dry Season



Note: *This figure shows water level data collected during a typical period during the wet season.*

Ocean tides fluctuate between 0' and 9.5'. Sand Lake water levels are typically slightly higher than the ocean at high tide. Low tides in Sand Lake never fall below 5.5' due to controlling bed elevations.

Water levels inside the dike/ Marsh show a muted tidal connection and low capacity for drainage. The tide gate prevents high tides from entering the marsh. After a rainfall event, water accumulates within the marsh, and the undersized culvert prevents efficient drainage. Accumulated water takes several days, and several low tides, to drain.

Water levels at Sand Lake Road upstream of the beaver dam are relatively constant at 8', with a slight increase following a rainfall event.

Figure 8
Typical Water Level Fluctuations in Beltz Marsh During the Wet Season



Note: This figure shows the extent of the 2D HEC-RAS Model, hydrologic inputs, and downstream boundary.

Figure 9
2D Model Extents, Downstream Boundary, and Hydrologic Inputs

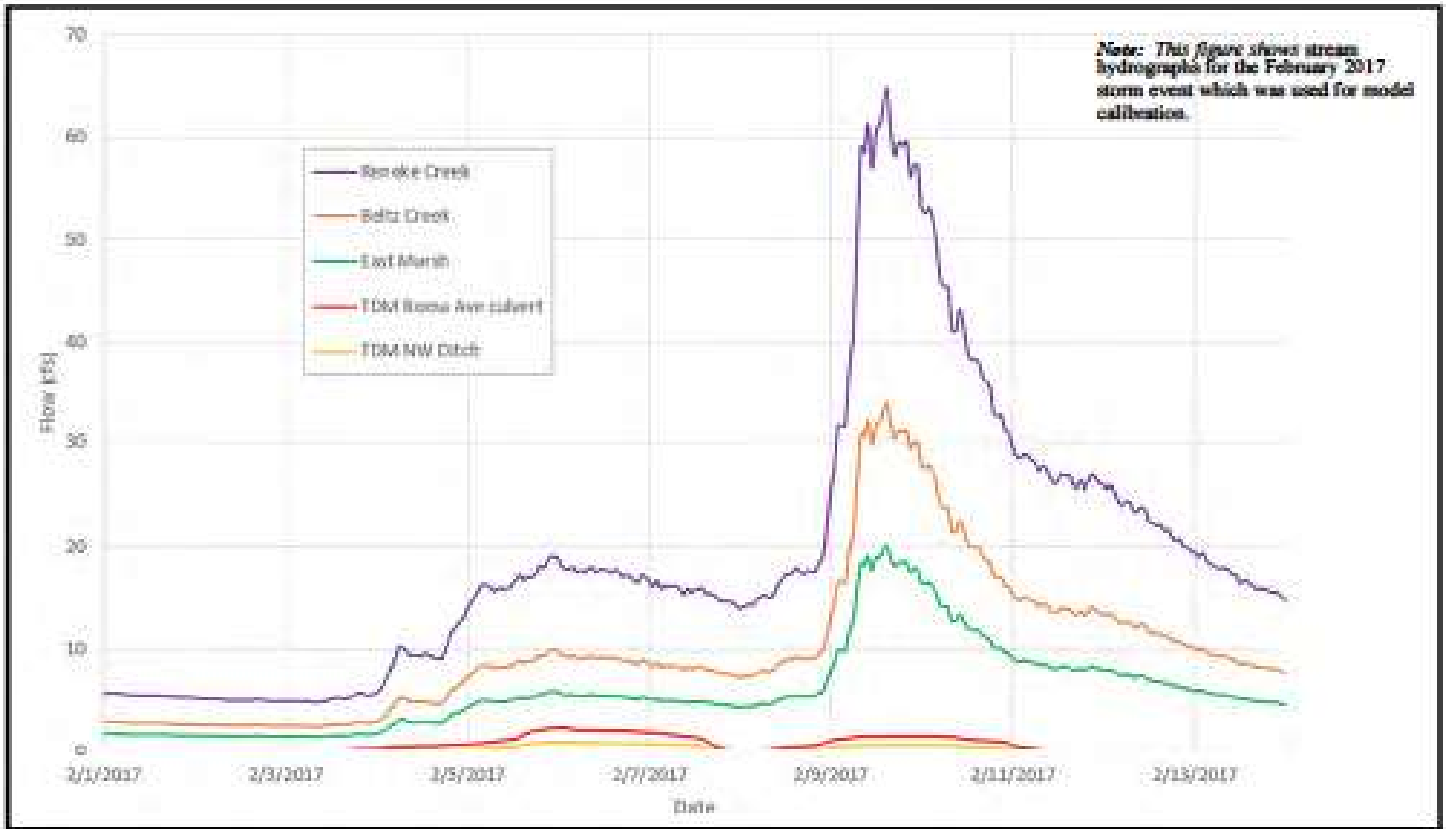


Figure 10
2D Model Inflow Hydrographs, February 2017 Calibration Event

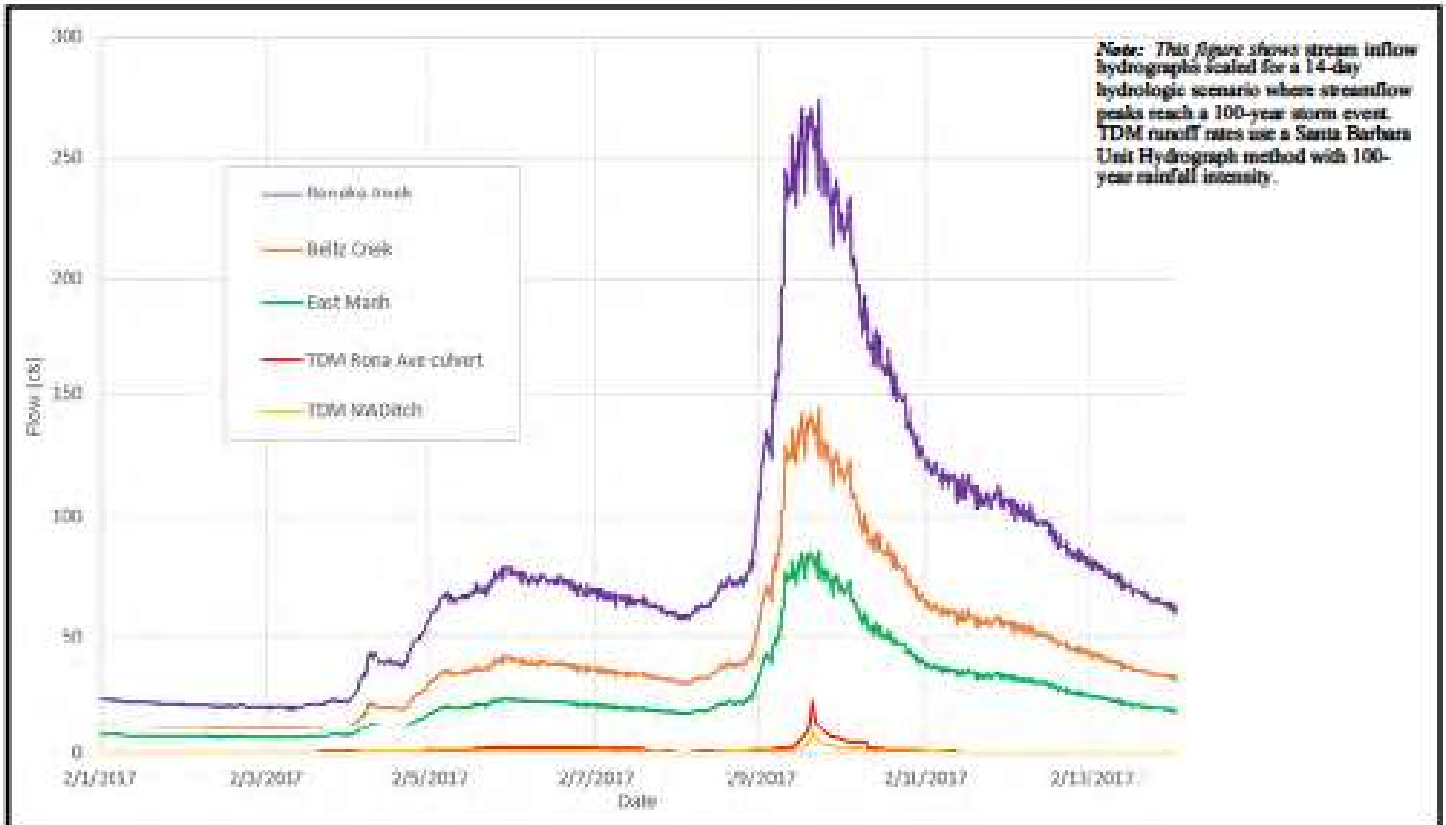


Figure 11
2D Model Inflow Hydrographs for the 100-year storm even

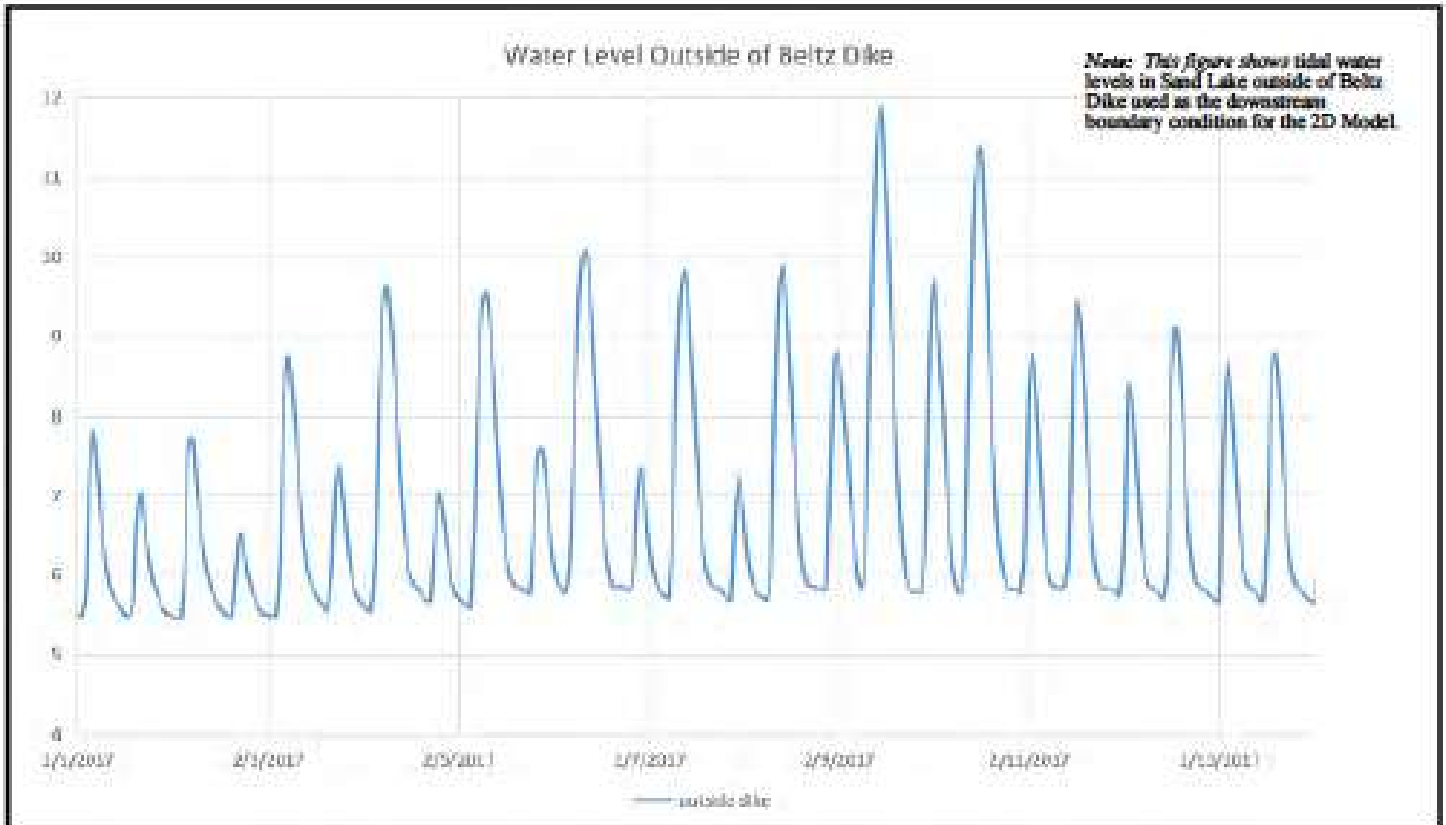
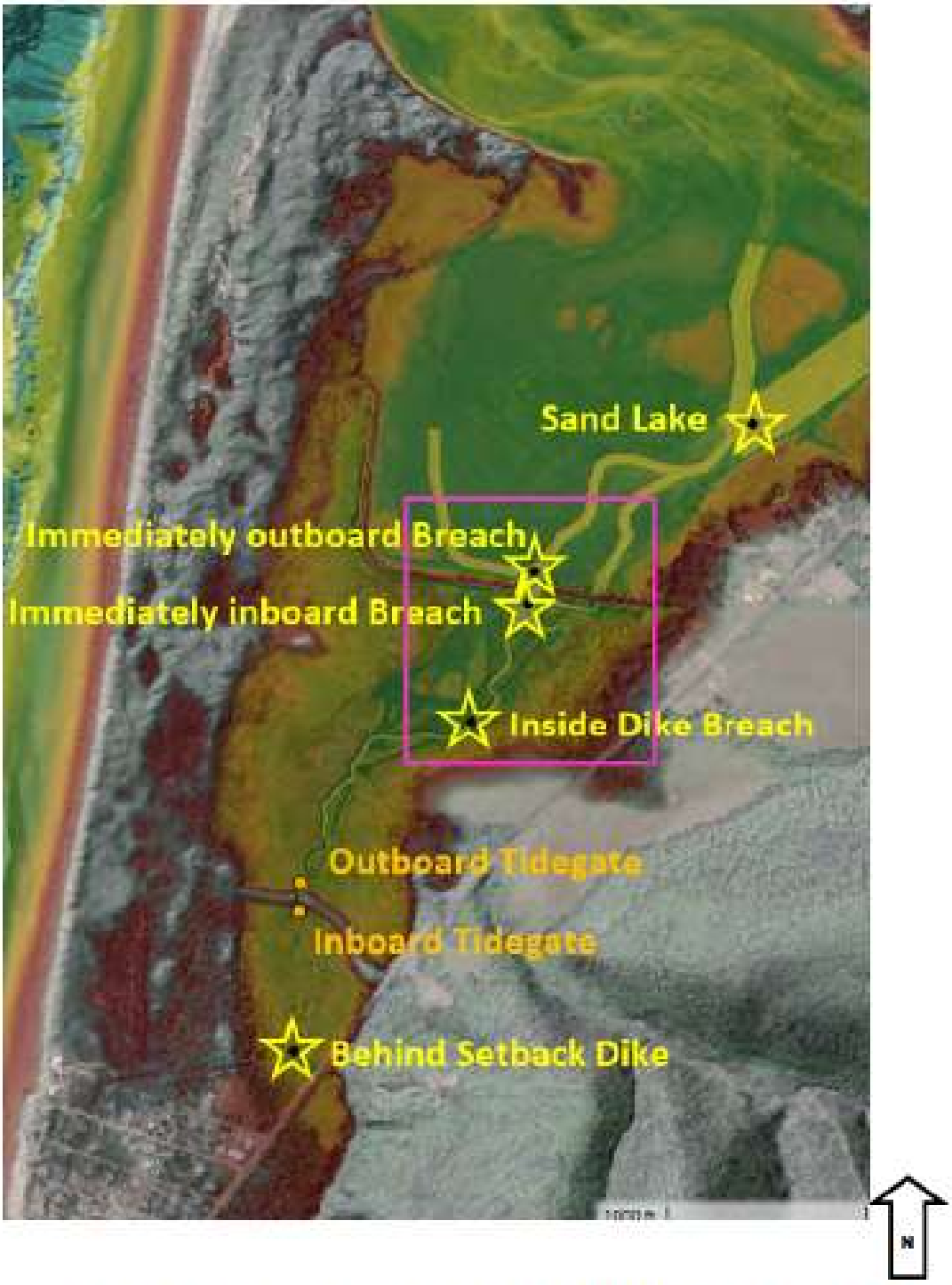


Figure 12
2D Model Downstream Boundary Condition



Note: This figure shows the sample location points within the HEC-RAS model for water level and velocity plots.

Figure 13
Sample Point Locations in the 2D Model for Water Level and Velocity Plots

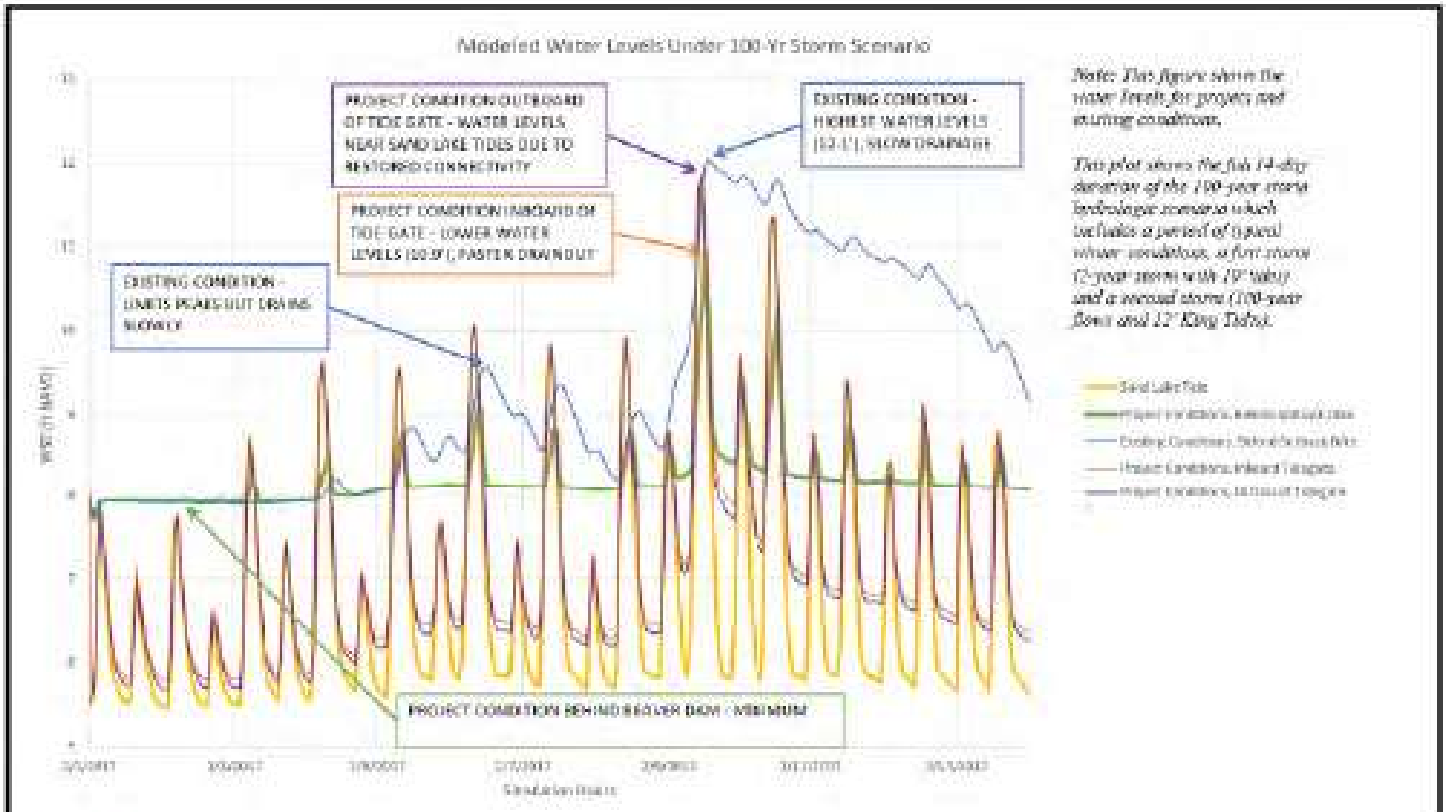
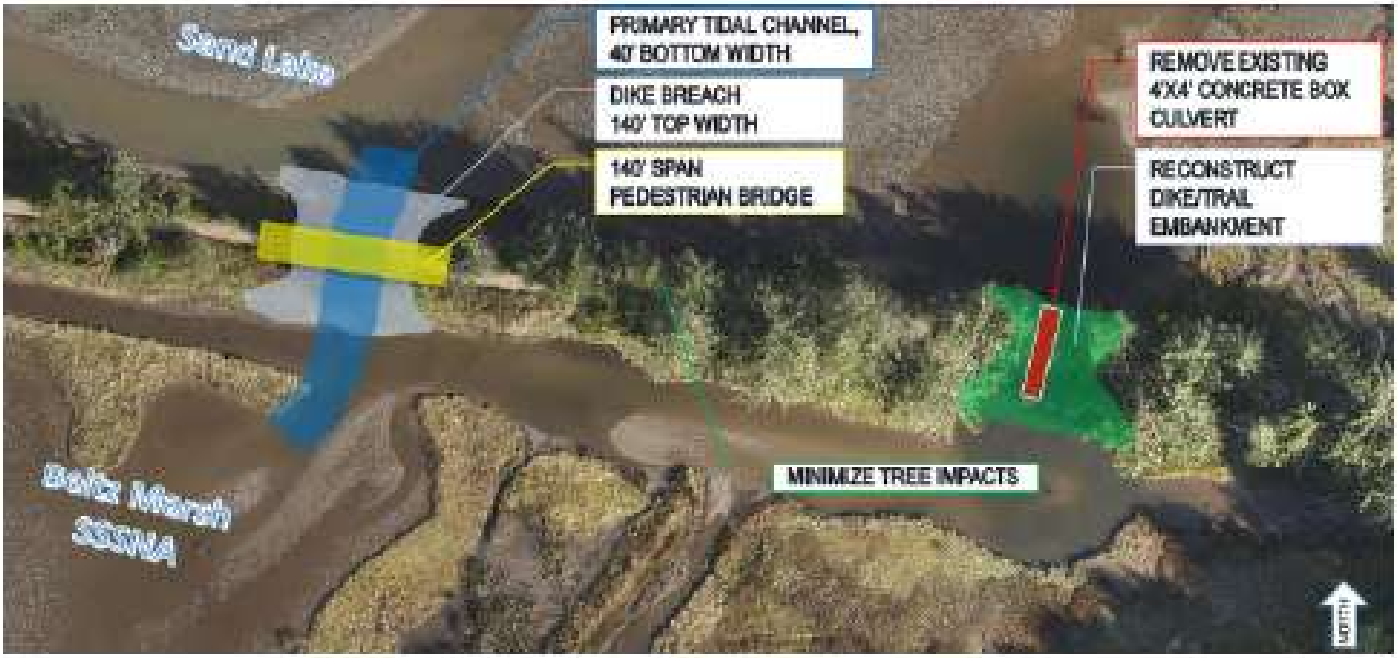


Figure 14
Modeled Water Levels at South End of Beltz Marsh During 100-Year Storm Scenario (Full Duration)



Note: This figure shows a current aerial photo (NS Consulting, 2022), showing the central historical flow path connecting channels inboard and outboard of Beltz Dike, as well as the existing box culvert/tide gate location which is not aligned with historical flow paths.

Figure 15
2022 Aerial Photo of Beltz Dike, Existing Culvert, and Historical Flow Path



Note: This figure shows a conceptual plan for maintaining recreational access by constructing a pedestrian bridge over a dike breach, as well as removing the existing failing box culvert and reconstructing the eroding embankment to provide long-term trail stability.

Figure 16
Concept Plan for Constructing a Pedestrian Bridge Over a Dike Breach

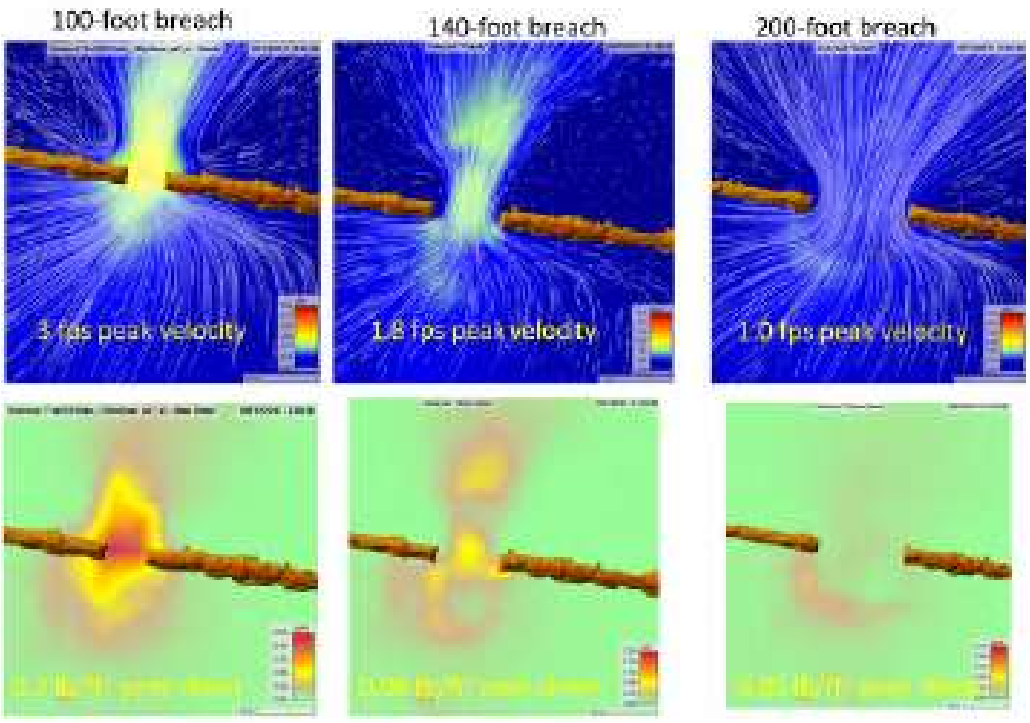


Figure 17
Comparison of Peak Velocity and Shear Stress Through 100-, 140-, and 200-Foot Dike Breaches

PEAK VELOCITY DURING 100-YR STORM RUN (MAX TIDE SWING)

Peak velocity ~ 3ft/sec (short duration peak, majority of time is less than 2 ft/sec: fish passage velocity threshold)

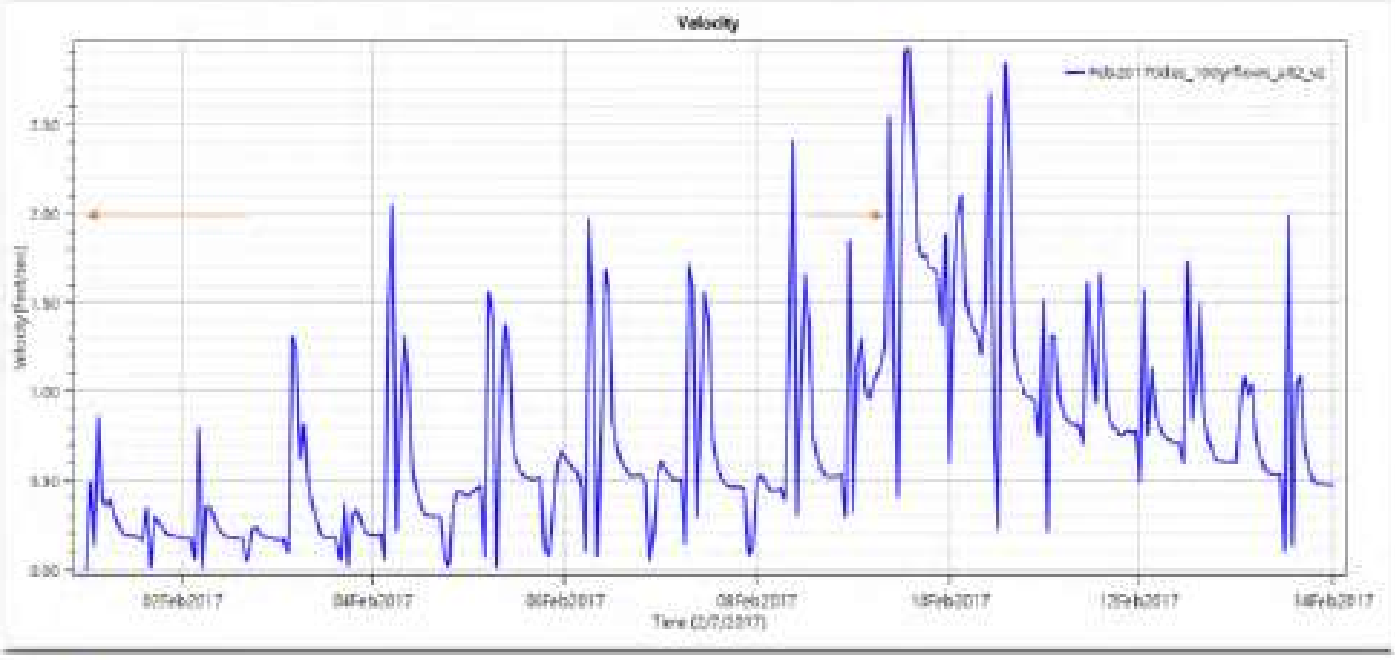


Figure 18
Water Velocity Through a 100-Foot Dike Breach During the 100-Year Hydrologic Scenario

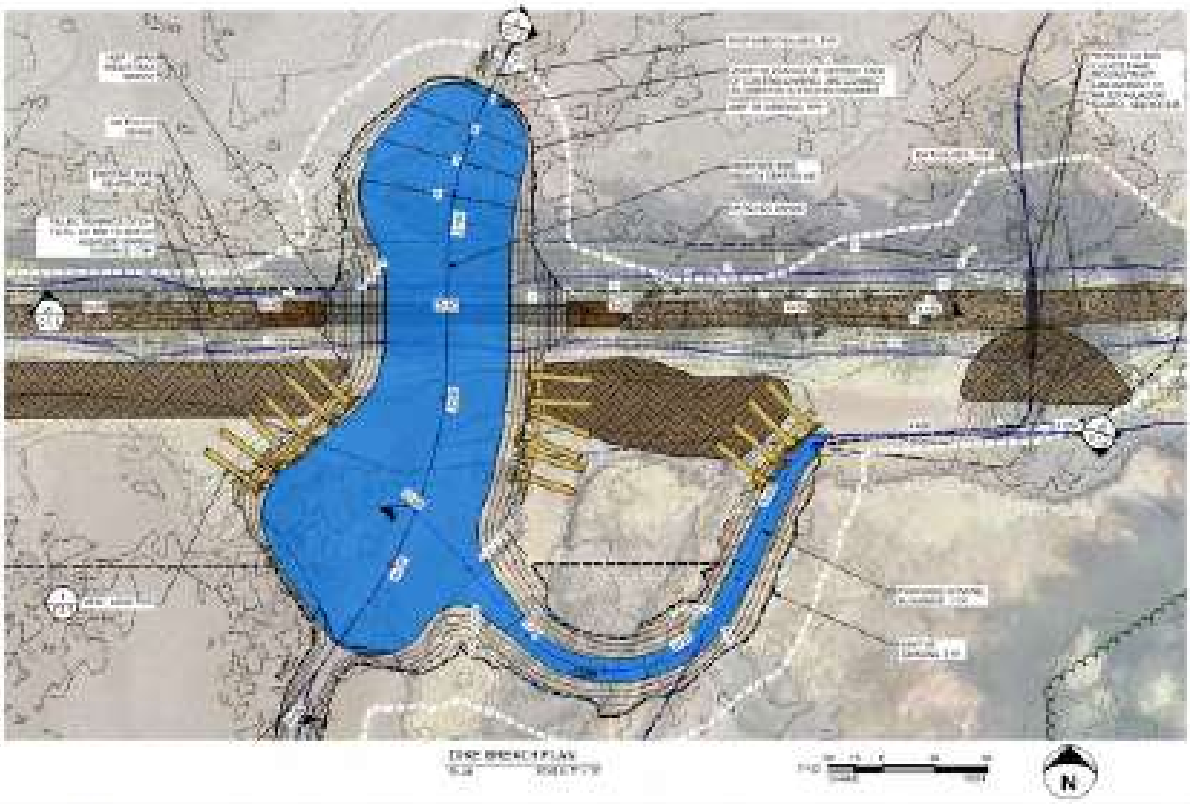


Figure 19
Dike Breach Channel and Pedestrian Bridge - 90% Design Plan View

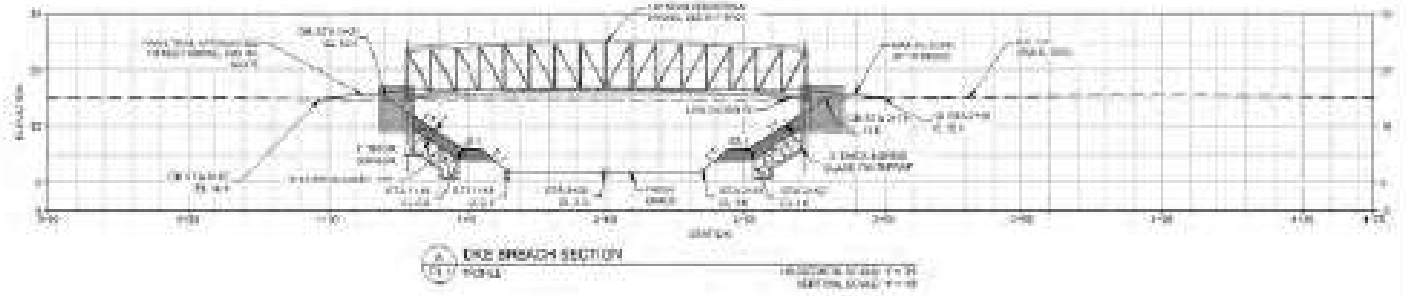


Figure 20
Dike Breach Channel and Pedestrian Bridge - 90% Design Breach Section View

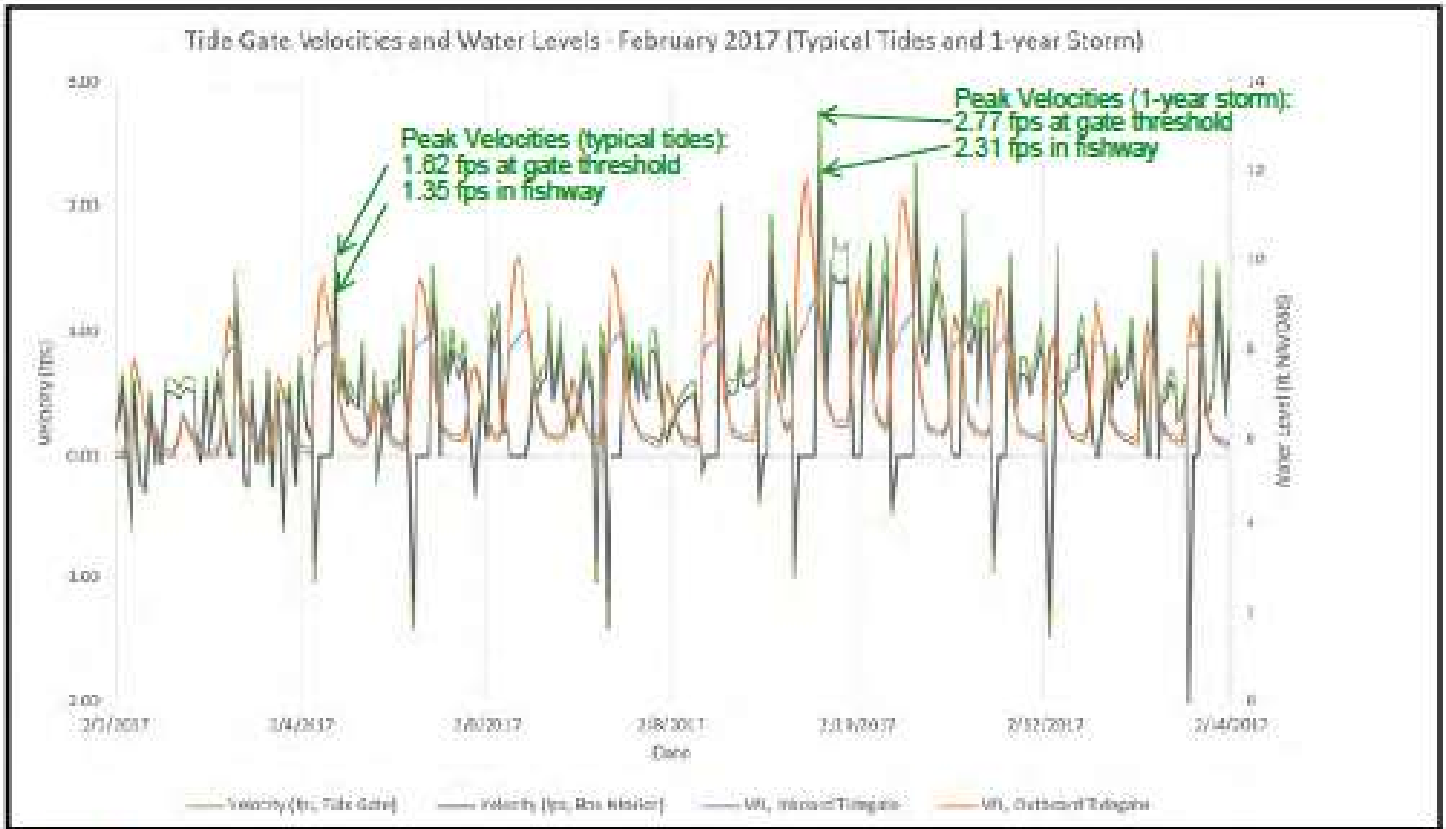


Figure 21
Modeled Tide Gate Velocities and Water Levels - February 2017 Typical Winter Conditions and 1-year Storm

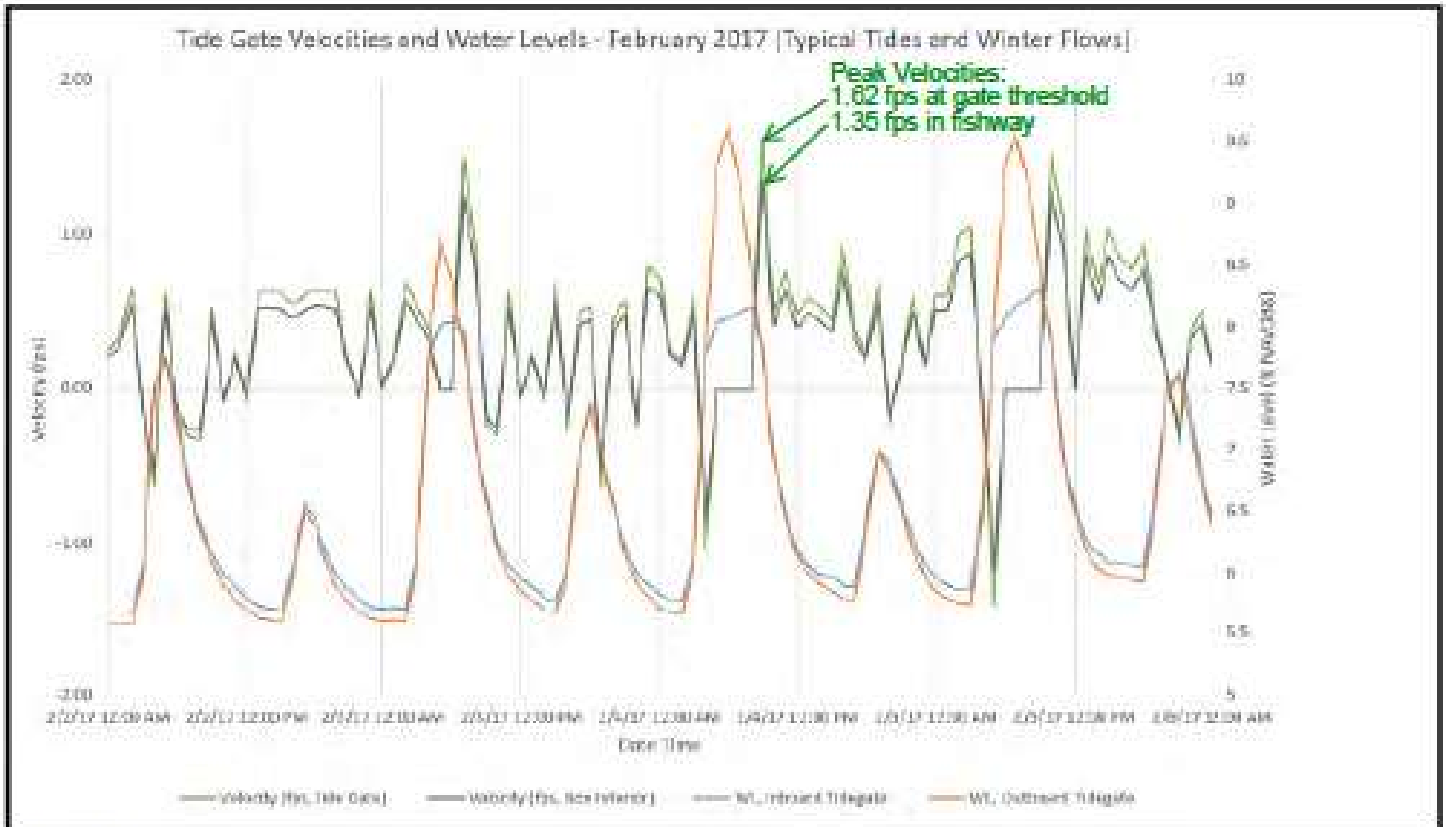


Figure 22
Modeled Tide Gate Velocities and Water Levels - February 2017 Typical Tidal Conditions

ESA

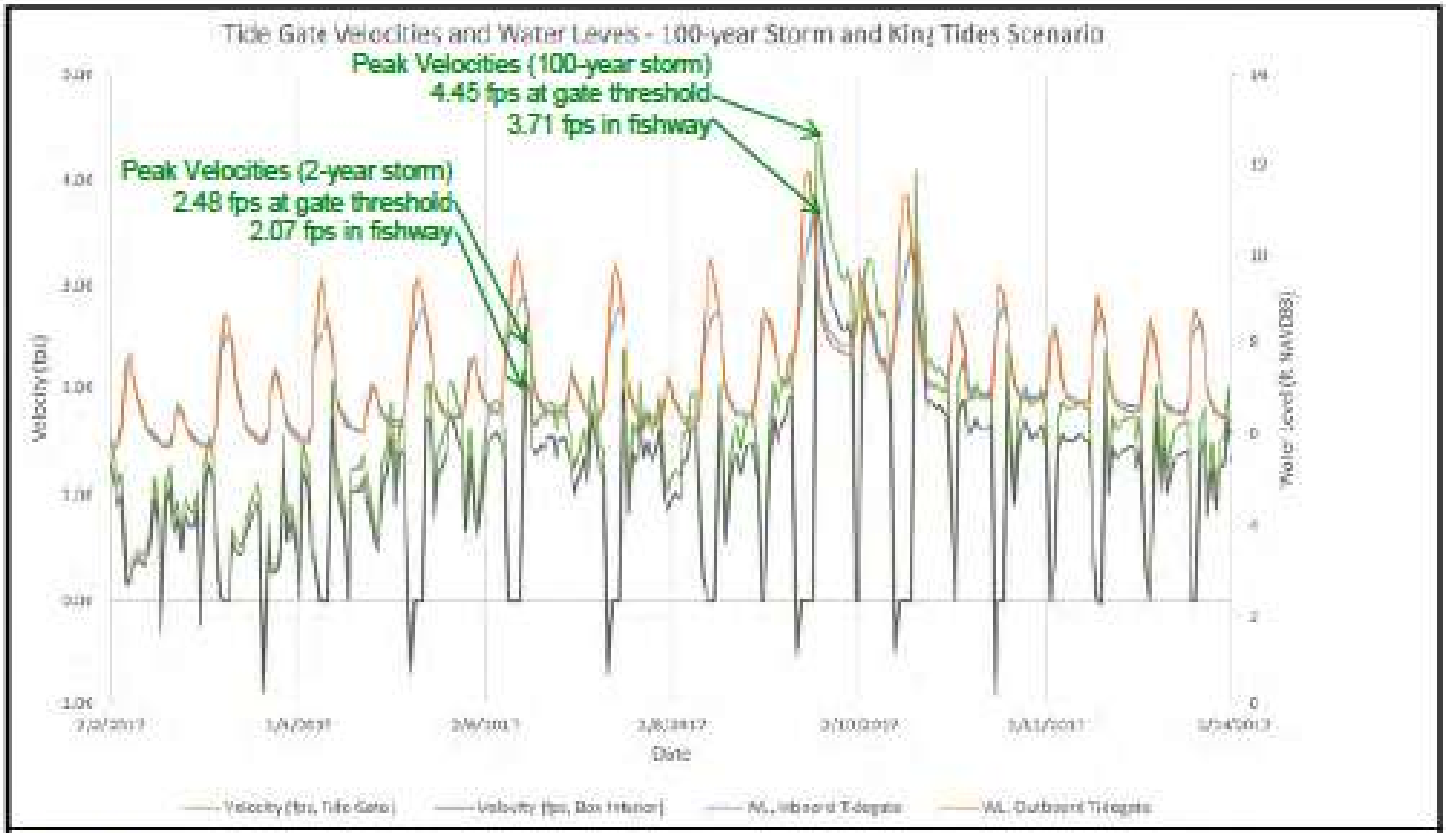


Figure 23
Modeled Tide Gate Velocities and Water Levels - 100-Year Storm Event and King Tides

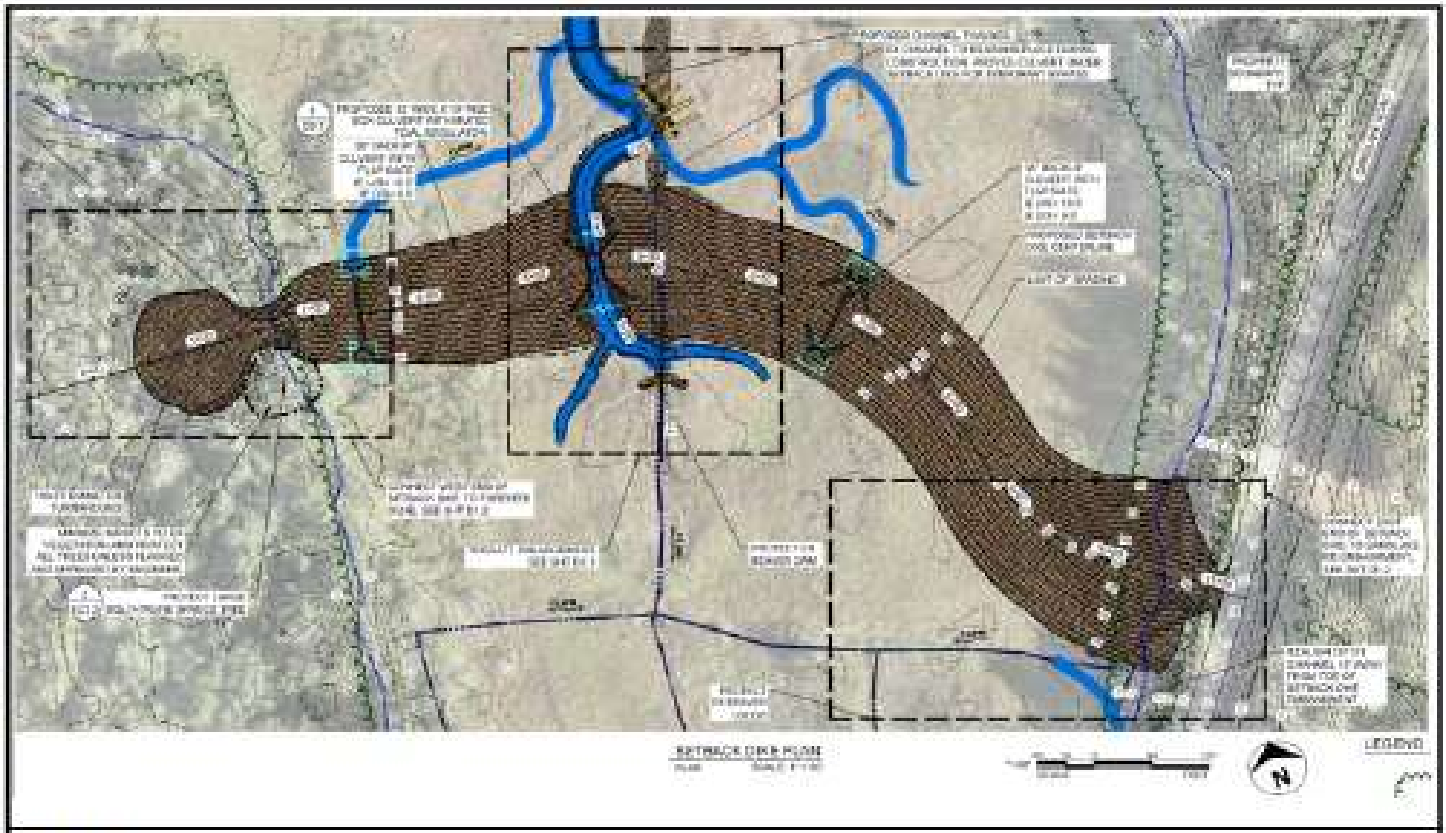


Figure 24
Setback Dike Plan - 90% Design

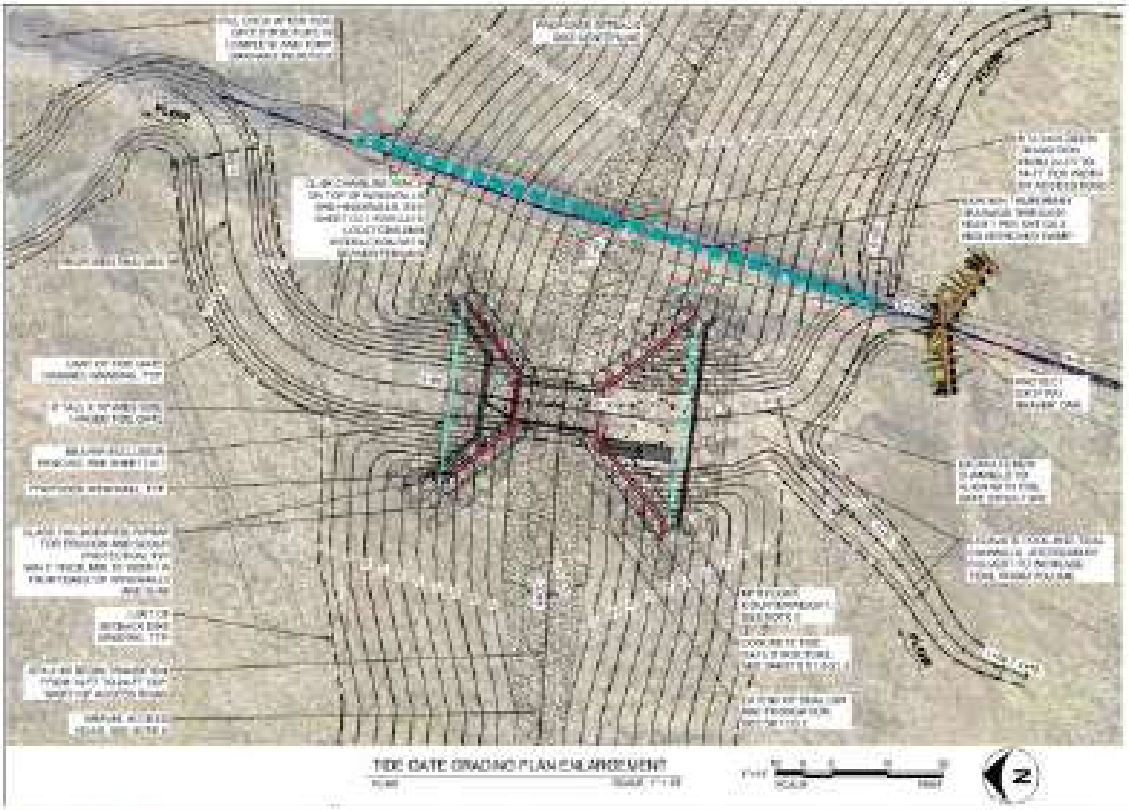


Figure 25
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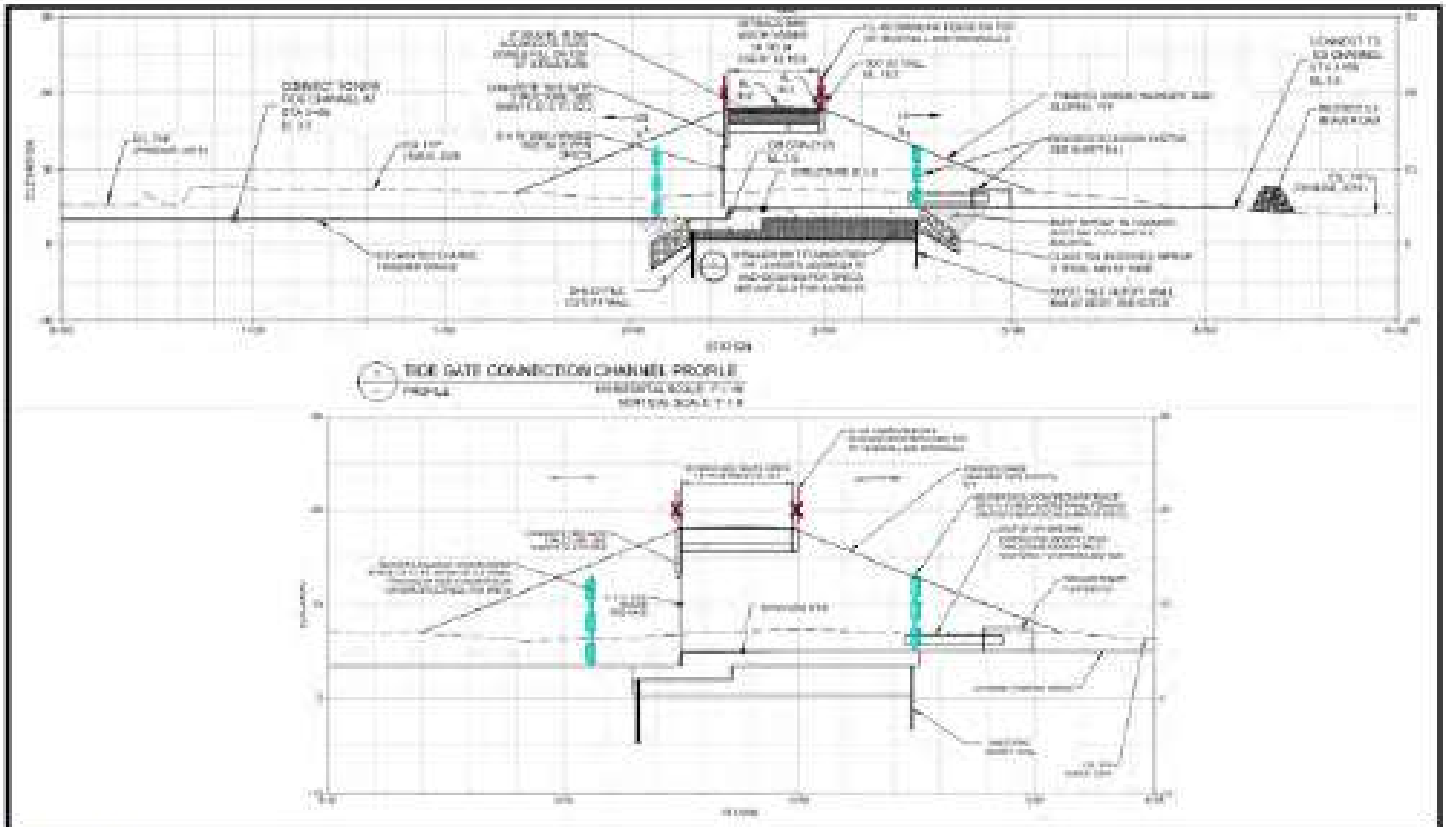


Figure 26
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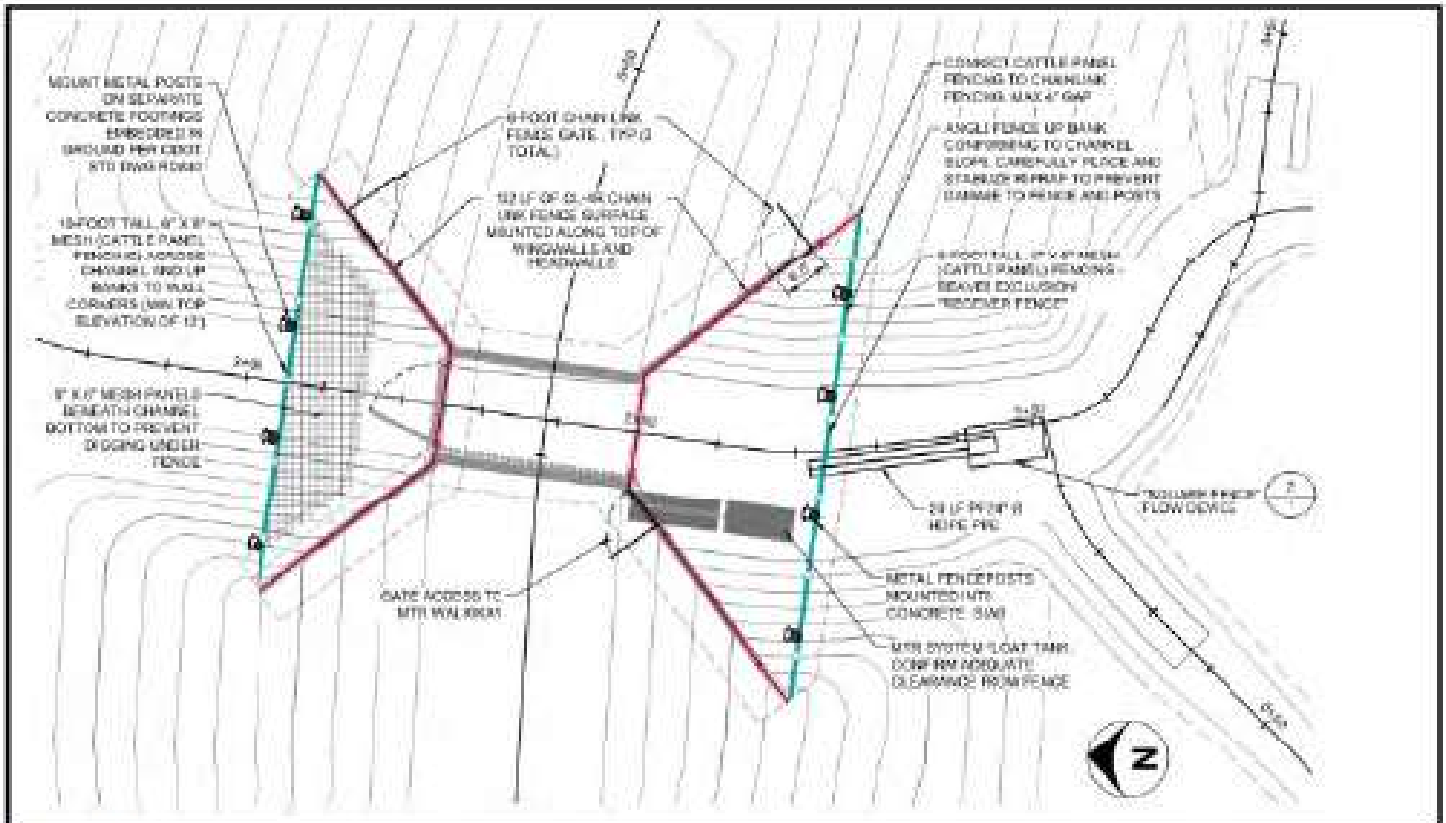


Figure 27
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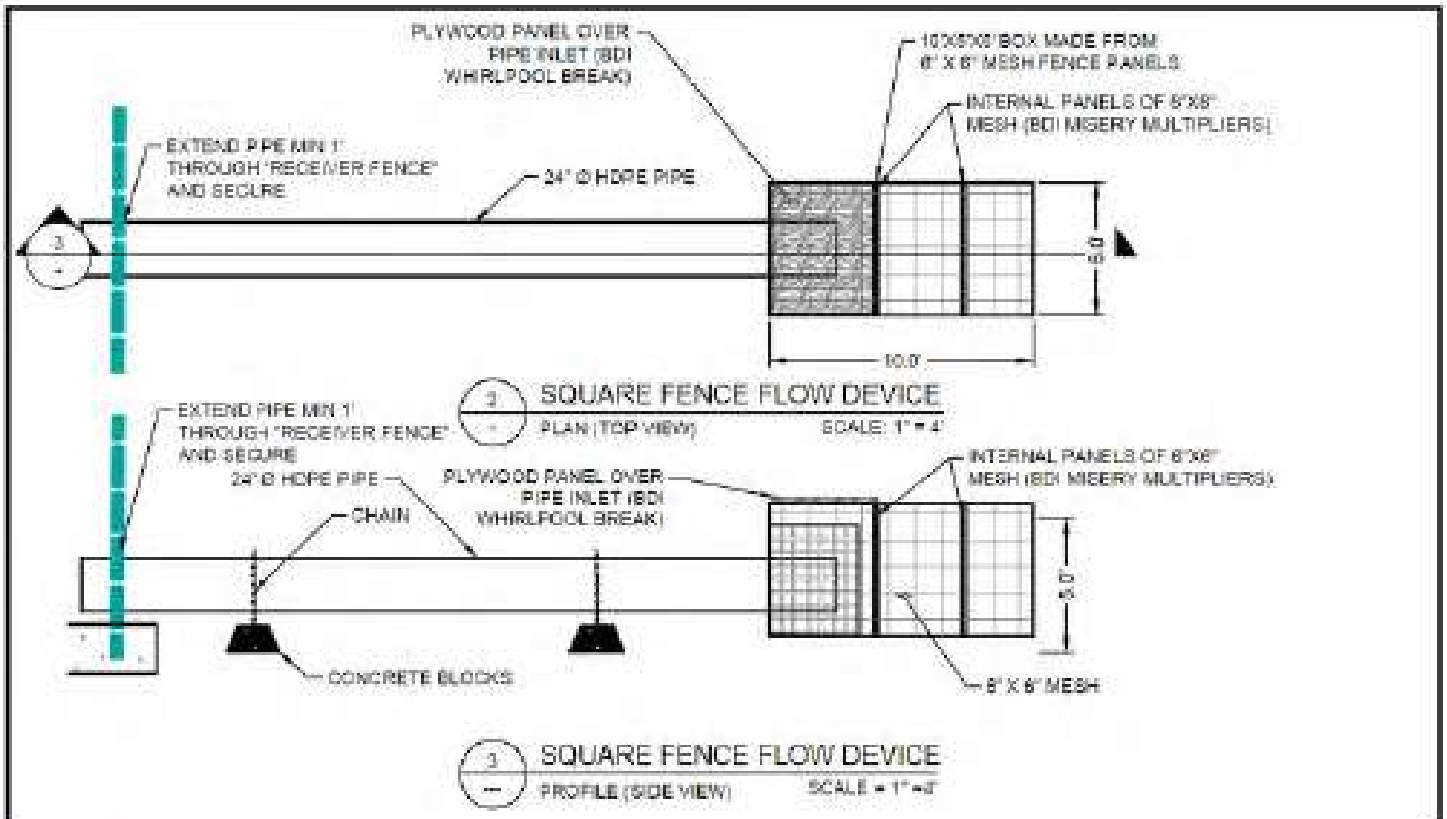


Figure 28
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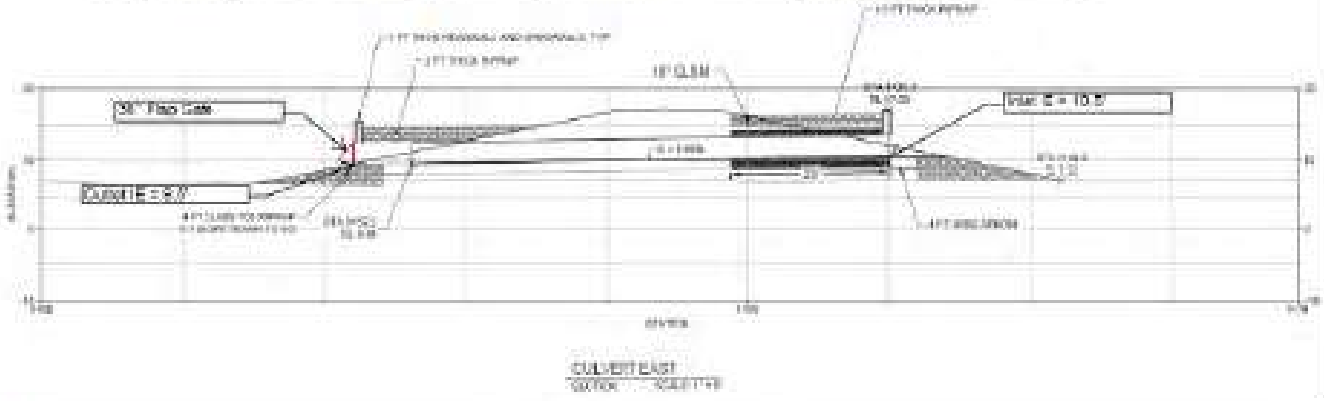


Figure 29
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Appendix F.
Sitka Sedge Tidal Wetland Restoration (Bridges and Culvert) Preliminary Geotechnical Engineering Report



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PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

Sitka Sedge Tidal Wetland Restoration (Bridges and Culvert)

TILLAMOOK COUNTY, OREGON





SHANNON & WILSON

November 2025

Shannon & Wilson No: 112880

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Submitted To: Environmental Science Associates
819 SE Morrison Street,
Suite 310
Portland, OR 97214
Attn: Hunter White

Subject: DRAFT PRELIMINARY GEOTECHNICAL ENGINEERING REPORT, SITKA SEDGE TIDAL WETLAND RESTORATION (BRIDGES AND CULVERT), TILLAMOOK COUNTY, OREGON

Shannon & Wilson prepared this Preliminary Geotechnical Engineering Report (Bridges and Culvert) and participated in this project as a subconsultant to Environmental Science Associates (ESA). Our scope of services was specified in the Subcontractor Agreement with ESA dated September 23, 2024. This report presents the findings of our geotechnical investigation, the results of our geotechnical analysis, and our recommendations for the geotechnical elements of the project. This report was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or if we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON

Travis Nguyen, PE, GE
Senior Associate

Eric Paslack, PE
Senior Associate

SCF:SCS:TTN:ECP:RPP/mmb

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Important Information

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ACRONYMS

ASTM International	American Society for Testing and Materials International
AASHTO	American Association of State Highway and Transportation Officials
BDDM	Bridge Design and Drafting Manual
bgs	below ground surface
CSZ	Cascadia Subduction Zone
FHWA	Federal Highway Administration
GDM	Geotechnical Design Manual
LRFD	Load and Resistance Factor Design
MSE Wall	Mechanically Stabilized Earth Wall
ODOT	Oregon Department of Transportation
SPT	Standard Penetration Test
USCS	Unified Soil Classification System

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1 INTRODUCTION

1.1 Project Overview

This preliminary report presents the results of our preliminary geotechnical design recommendations for a proposed Pedestrian Bridge along the trailhead and the proposed culvert replacements with bridges and a box culvert along Sandlake Road as part of the overall Sitka Sedge Tidal Wetland Restoration Project (the Project) in Tillamook County, Oregon. As a consultant to Environmental Science Associates (ESA), Shannon & Wilson is providing geotechnical engineering services, including field explorations, laboratory testing, and preliminary engineering analysis and recommendations for the proposed bridges and culvert on Sandlake Road and Sitka Sedge trailhead included in the project. The location of the project site is shown on the Vicinity Map, Figure 1.

The final report will be prepared for these bridges and culvert during final design phase. Geotechnical design recommendations for levee setback design elements will be provided in a separate report titled "Sitka Sedge Tidal Wetland Restoration (Levee Setback)".

Elevations are referenced to North American Vertical Datum 1988 (NAVD88).

1.2 Scope of Services

Shannon & Wilson's services were conducted in accordance with the Subcontractor Agreement with ESA dated September 23, 2024. The completed geotechnical design services for the project consisted of the following tasks:

- Performing a site reconnaissance to determine site conditions, access, potential drilling locations, and safety concerns;
- Performing the Field Exploration Program, including drilling seven borings;
- Performing laboratory testing to determine soil index properties;
- Evaluating subsurface conditions;
- Reviewing available existing geotechnical data, including geology, soil maps, seismicity, and previous borings;
- Evaluating site-specific seismic hazards, including liquefaction potential, lateral spreading, and slope instability;
- Evaluating foundation types for bridges;
- Providing deep foundation design recommendations for bridges;
- Providing shallow foundation design recommendations for culvert;

- Providing recommendations for bridge abutment walls, including lateral earth pressures and global stability;
- Evaluating the overall stability of the existing trailhead embankment in the vicinity of the proposed pedestrian bridge; and
- Preparing this Preliminary Geotechnical Engineering Report summarizing all tasks listed above.

2 PROJECT UNDERSTANDING

2.1 Site Description

The Project site is located along the Sitka Sedge trailhead near Sitka Sedge Parking and Trailhead Park and along Sandlake Road between Sitka Sedge Parking and Trailhead Park and Roma Avenue in Tillamook County, Oregon.

The trailhead consists of fill embankment through the Sitka Sedge wetland. The trail is approximately 8 to 15 feet wide with an approximate side slope gradient of 1H:1V to 2H:1V. Brushes and trees are growing along the trailhead as shown on Exhibits 2-1 and 2-2.

Sandlake Road runs northwest to southeast from Sitka Sedge Parking and Trailhead to Roma Avenue. The pavement of Sandlake Road consists of asphaltic concrete pavement without shoulders on both sides. Southbound from the Sitka Sedge Parking and Trailhead, Sandlake Road slopes gently downhill to a local low point at the Reneke Creek Culvert, by borings B-3 and B-4. From this crossing, the road slopes uphill past the Beltz Creek culvert, where B-5 is located, to a local crest slightly south of a residential gravel driveway on the east side of the road. Sandlake Road then slopes downhill to the South, passing boring B-6 and crossing an unnamed creek at the location of boring B-7 before leveling out at the intersection with Roma Avenue. The east side of the road generally consists of a wooded hillside. The Sitka Sedge wetland is located to the west of the road. Vegetation consisting of grass, brushes, and trees are growing on both sides of the shoulder. Exhibit 2-3 through Exhibit 2-6 present site photographs showing several views of the site and existing features.



Exhibit 2-1: Looking West Along Trailhead from Boring B-1



Exhibit 2-2: Looking East Along Trailhead from Boring B-2



Exhibit 2-3: Looking Northeast Along Sandlake Road from Borings B-3 and B-4



Exhibit 2-4: Looking Southeast Along Sandlake Road from Boring B-5



Exhibit 2-5: Looking Northwest Along Sandlake Road from Boring B-5



Exhibit 2-6: Looking Northwest Along Sandlake Road from Boring B-7

2.2 Project Description

The Project consists of a proposed pedestrian bridge along the trailhead and culvert replacements with bridges and a box culvert along Sandlake Road. The culvert replacements consist of proposed bridges at Reneke Creek and Beltz Creek and a box culvert at No Name Creek. The structure locations are shown on the Site and Exploration Plan, Figure 2.

Sixty percent (60%) design documents for the proposed Pedestrian Bridge and Reneke Creek Bridge are included in Appendix A. Thirty percent (30%) design documents for the proposed Beltz Creek Bridge and proposed No Name Creek culvert are included in Appendix A. Levee setback is included in the 60% design documents. Based on our discussions with a structural engineer at David Evans and Associates (DEA), we understand foundation type, size, and depth for these proposed bridges may be changed during final phase design.

The existing culvert on the Sitka Sedge trailhead is a 24-inch diameter concrete pipe culvert with a missing gate. The existing culvert will be removed and backfilled with on-site materials to the same geometry as the existing trailhead. Based on 60% design documents, the Pedestrian Bridge is a 140-foot pre-manufactured truss single-span bridge. Based on 60 design documents, each bent of the proposed single-span Pedestrian Bridge will be supported on three (3) 20-inch-diameter closed-end steel pipe piles. The factored axial compressive load for each pile was provided by DEA and is summarized in Exhibit 2-7. A dike breach will be required for the Pedestrian Bridge to provide a fish-friendly and wetland connection. The Pedestrian Bridge will have concrete abutment walls up to 8 feet tall and wing walls. The bottom of the pile cap is proposed at approximate elevation of 8.5 feet. We also understand that the wingwalls will be cantilevered from the bridge abutments and pile caps. The bridge approach may be raised up to 2 feet to accommodate the finish grade of the bridge. The existing trailhead embankment will not be widened.

Exhibit 2-7: Factored Axial Compressive Load for each Pile at Pedestrian Bridge

Structure	Factored Axial Compressive Load for Each Pile(kips)		
	Strength Limit (kips)	Service Limit (kips)	Extreme Event Limit (kips)
Pedestrian Bridge	Not Provided	133	Not Provided

Based on 60% design documents, the existing culvert at Reneke Creek undercrossing Sandlake Road is a 24-inch diameter concrete pipe culvert. The existing culvert will be replaced with a 49-foot single-span bridge. Based on 60% design documents, each bent of the proposed single-span Reneke Creek Bridge will be supported on eight (8) steel H-piles, HP 16x141. The factored axial compressive load for each pile was provided by DEA and is

summarized in Exhibit 2-8. The bridge will have concrete abutment walls and wing walls up to 6 feet tall. The bottom of the pile cap is proposed at approximate elevation of 12.5 feet. We also understand that the wingwalls will be cantilevered from the bridge abutments and pile caps.

Exhibit 2-8: Factored Axial Compressive Load for each Pile at Reneke Bridge

Structure	Factored Axial Compressive Load for Each Pile (kips)		
	Strength Limit (kips)	Service Limit (kips)	Extreme Event Limit (kips)
Reneke Creek Bridge	151	96	166

The existing culvert at Beltz Creek undercrossing Sandlake Road at a skew is a 36-inch diameter concrete pipe culvert. Based on the 30% design document, the existing culvert will be replaced with a 45-foot single-span bridge. The factored axial compressive load for each pile was provided by DEA and is summarized in Exhibit 2-9. The bridge will have concrete abutment walls and wing walls up to 6 feet tall. The bottom of the pile cap is proposed at approximate elevation of 26 feet. We understand that the proposed Beltz Creek Bridge structure may be similar to the Reneke Bridge.

Exhibit 2-9: Factored Axial Compressive Load for each Pile at Beltz Bridge

Structure	Factored Axial Compressive Load for Each Pile (kips)		
	Strength Limit (kips)	Service Limit (kips)	Extreme Event Limit (kips)
Beltz Creek Bridge	180	118	168

The existing culvert at No Name Creek undercrossing of Sandlake Road is a 24-inch-diameter metal pipe culvert. Based on the 30% design document, the existing culvert will be replaced with an 8-foot x 12-foot precast box culvert. The top of the proposed culvert is located approximately 2 feet below the pavement surface. The bottom of the culvert will be located at an approximate depth of 10 feet bgs (approximate elevation 1.5 feet).

3 GEOLOGIC AND SEISMIC SETTING

3.1 Regional Geology

The project site lies within the Coast Range physiographic province near the western edge of the mountains of the Coast Range and in the estuary and wetland areas east of the shoreline of the Pacific Ocean. Basement rocks within the Coast Range consist of volcanic and sedimentary rocks of the Siletz Terrane. The Siletz Terrane is an exotic crustal block made up of rocks from an ocean island arc within the Pacific, which was accreted to the western edge of the North American continent approximately 51 to 49 million years ago (Wells and Others, 1994). Overlying the rocks of the Siletz Terrane is a series of sedimentary

rocks deposited in shallow seas within the western continental shelf of North America throughout the late Eocene (55.8 to 33.9 million years ago), Oligocene (55.8 to 23 million years ago), and into the Miocene (23 to 5.2 million years ago) (Wells and others, 1994).

Tectonic uplift of the Coast Range from the Miocene into the Pleistocene (2.3 million to 11,700 years ago), combined with erosion, created the rugged coastline and bay inlets seen today as the rivers and streams flowing out of the Coast Range mountains carved large valleys and canyons, which were inundated as sea levels rose after the last ice age. Sediments transported from inland, as well as from ocean currents and wind, accumulated within these inundated bay areas creating wetlands and estuaries.

3.2 Site Geology

The coastal area around Sand Lake is located within an estuary located between Pacific City to the south and Cape Lookout to the north. The estuary of Sand Lake area was created within a wide ancient river channel where multiple creek valleys converge within an area of low elevation before draining into the Pacific Ocean. The estuary area is bounded and protected due to long-shore transport of beach sediment which developed sand bars along the shoreline that partially shielded Sand Lake from the ocean (Wilsey and Ham, Inc. 1974). Large amounts of sediment have been deposited within this estuary through a combination of sources, including stream alluvium deposited from creeks which drain from the Coast Range highlands, oceanic and tidal sediments deposited from long-shore currents and tidal activity, and eolian sediments deposited by the wind (Wilsey and Ham, Inc. 1974; Wells and others, 1994). The surface geology of the greater Sand Lake area, including the project site, is generally mapped by Wells and others (1994) as quaternary sediment, including fluvial and estuarine deposits of unconsolidated clay, silt, sand, and gravel deposited by rivers and streams, as well as tidal flat mud, sand, and peat. Wells and others (1994) mapped the uplands to the east as Alsea Formation, consisting of Oligocene and Miocene marine sedimentary rocks, including tuffaceous siltstone and sandstone. The Alsea formation is locally intruded by irregular bodies of 6 to 17 million-year-old Grande Ronde Basalt (Wells and others, 1994).

Landslides and debris flows are also common within the slopes of the mountains and foothills of the Coast Range (DOGAMI SLIDO, 2024). According to the Statewide Landslide Information Database for Oregon (SLIDO), alluvial fan deposits are mapped within the alignment and near borings B-3, B-4, and B-5. A small landslide is mapped between B-6 and B-7. The landslide deposits near B-7 are designated as historic (<150 years old), and the other alluvial fan and landslide deposits are designated as prehistoric (>150 years old) (SLIDO, 2025). Exhibit 3-1 presents an image from SLIDO of the mapped landslide located upslope from Sandlake Road.

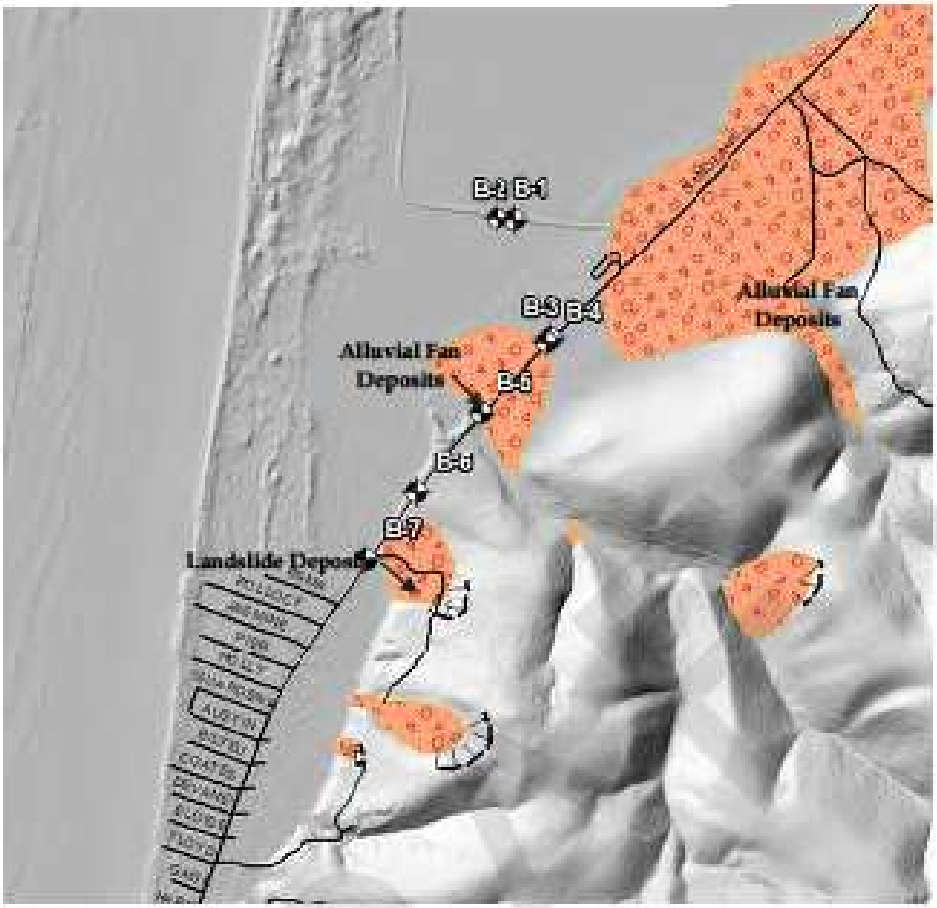


Exhibit 3-1: SLIDO data showing mapped landslides and alluvial fan deposits within the project area (SLIDO, 2025).

In recent history, the landscape of the Sand Lake Estuary has been further altered by human development, including changes to waterways, grading, and placement of fill material. Three dike structures have altered the tidal flow of the estuary. These dike structures include part of the county access road to Whalen Island, the Beltz Creek Dike, and a flood control dike located on the north shore of the Sandlake between Galloway Road and Sand Creek east of the recreation area. These dikes have constricted tidal flows and created significant increases in marsh areas within the bay (Wilsey and Ham, Inc., 1974).

3.3 Seismic Setting

Earthquakes in the Pacific Northwest occur largely as a result of the region's proximity to an active convergent plate boundary, the Cascadia Subduction Zone (CSZ). In the CSZ, dense oceanic crust of the Explorer, Juan de Fuca, and Gorda Oceanic Plates are subducting beneath the overriding, less dense, westward-moving continental crust of the North

American Plate. Oblique convergence of these plates not only results in east-west compressive strain, but also in dextral shear, clockwise rotation, and north-south compression of accreted crustal blocks that form the leading edge of the North American Plate (Wells and others, 1998). The CSZ runs approximately parallel to the coastline for about 750 miles from northern California to southern British Columbia. Within the present understanding of the regional tectonic framework and historical seismicity, three broad seismogenic sources have been identified:

- A mega-thrust source at an interface between the North American and Juan de Fuca plates in the CSZ;
- A deep subcrustal zone (intraslab) in the subducted Juan de Fuca Plate in the CSZ; and
- A shallow crustal zone within the North American Plate.

The following sections briefly describe the location, characteristics, and seismicity of each of the sources.

3.3.1 Cascadia Subduction Zone: Mega-Thrust Interface Source

CSZ mega-thrust earthquakes originate along the interface between the subducting oceanic plates and the North American plate. Because of the significant uncertainty of the landward extent of a potential rupture surface, estimates of the closest distance between the project and the potential rupture surface range from about 40 to 80 miles. Focal depths for mega-thrust earthquakes are commonly on the order of about 15 to 25 miles. Rupture of the interface could result in earthquakes with moment magnitudes on the order of 8.5 to over 9.0, with strong shaking that lasts for several minutes. No large earthquakes have occurred in this zone during historic times (the last 170 years). However, geologic evidence suggests that coastal estuaries have experienced rapid subsidence at various times within the last 2,000 years (e.g., Atwater, 1987; Atwater and Hemphill-Haley, 1997). It appears that ruptures of this zone have occurred at irregular intervals that span from about 100 to more than 1,200 years, with an average recurrence interval of about 300 to 500 years (Atwater and Hemphill-Haley, 1997). Based on historical tsunami records in Japan (Satake and others, 1996), the most recent interplate event on the CSZ was a moment magnitude (Mw) 9 event on January 26, 1700.

3.3.2 Cascadia Subduction Zone: Intraslab Source

CSZ intraslab earthquakes originate from within the subducting oceanic plates because of down-dip tensional forces and bending caused by mineralogical and density changes in the plates at depth. These earthquakes typically occur 28 to 37 miles beneath the surface. Because intraslab events involve high-angle normal faulting, the area of the rupture surface and magnitude is strongly dependent on the thickness of the subducting slab. Young

subduction zones, such as the CSZ, generally have relatively thin subducting slabs. Thermal modeling of the CSZ (Hyndman and Wang, 1993) and the observed geometry of the Wadati-Benioff Zone confirm the likelihood that the subducting slab is relatively thin.

Worldwide observations indicate that the largest intraslab earthquakes are on the order of magnitude (M) 8, with the 12 largest of these occurring in older subducting slabs. The largest recorded intraslab earthquake beneath the Puget Lowland, the 1949 Olympia earthquake, was a surface wave magnitude M7.1 event. Ludwin and others (1991) estimate that the maximum magnitude from this source zone would be about 7.5.

At the project site, ground shaking produced by intraslab earthquakes would generally be less intense and less prolonged than ground motions generated by large subduction zone interface earthquake events. Historic seismicity from this source zone includes the 1949 magnitude 7.1 Olympia earthquake; the 1965 magnitude 6.5 earthquake between Tacoma and Seattle; and the 2001 magnitude 6.8 Nisqually earthquake. While intraslab events have occurred frequently in the Puget Sound area, they are historically rare in Oregon.

3.3.3 Shallow Crustal Source

Shallow crustal earthquakes within the North American Plate have historically occurred in a diffuse pattern within western Oregon, typically within the upper 4 to 19 miles of the continental crust. The largest known crustal earthquake in the Pacific Northwest is the 1872 North Cascades earthquake of magnitude 6.8. Other examples include the 1993 magnitude 5.6 Scotts Mill earthquake and the 1993 magnitude 6.0 Klamath Falls earthquake.

Shallow crustal faults and folds throughout Oregon and Washington have been located and characterized by the United States Geological Survey (USGS). The USGS provides approximate fault locations and a detailed summary of available fault information in the USGS Quaternary Fault and Fold Database. The database defines four categories of faults, Class A through D, based on evidence of tectonic movement known or presumed to be associated with large earthquakes during Quaternary time (within the last 2.6 million years). For Class A faults, geologic evidence demonstrates that a tectonic fault exists and that it has likely been active within the Quaternary period. For Class B faults, there is equivocal geologic evidence of Quaternary tectonic deformation, or the fault may not extend deep enough to be considered a source of significant earthquakes. Class C and D faults lack convincing geologic evidence of Quaternary tectonic deformation or have been studied carefully enough to determine that they are not likely to generate significant earthquakes.

According to the USGS Quaternary Fault and Fold database, there are six (6) Class A features within approximately 30 miles of the project alignment. Their names, general locations relative to the site, and the time since their most recent deformation are

summarized in Exhibit 3-2. Multiple faults associated with the Cascadian Fold and Fault Belt are located as near as eight (8) miles west of the project site, while the main megathrust fault of the CSZ lies approximately 65 to 75 miles west of the project site.

Exhibit 3-2: USGS Class A Faults Within an Approximate 30-Mile Radius of the Project Site

Fault Name	USGS Fault Number	Approximate Length	Approximate Distance and Direction from Project Site[1]	Slip Rate Category[2]	Time Since Last Deformation[3]
Unnamed Offshore Faults	785	174.0 miles	10.2 miles NW	1.0 to 5.0 mm/yr	< 15 ka
Stonewall Anticline	786	43.5 miles	24.7 miles SW	1.0 to 5.0 mm/yr	< 15 ka
Nehalem Bank Fault	789	62.8 miles	12.9 miles NW	1.0 to 5.0 mm/yr	< 15 ka
Tillamook Bay Fault Zone	881	19.8 miles	16.2 miles NE	< 0.2 mm/yr	< 1.6 Ma
Happy Camp Fault	882	1.9 miles	12.1 miles N	< 0.2 mm/yr	< 1.6 Ma
Siletz Bay Faults	883	6.2 miles	23.8 miles S	< 0.2 mm/yr	< 130 ka

NOTES:

1 Approximate distance between project site and nearest extent of fault mapped at the ground surface.

2 mm = millimeters; yr = year.

3 Ma = "Mega-annum" or million years ago; ka = "Kilo-annum" or one thousand years ago.

4 FIELD EXPLORATIONS

Shannon & Wilson explored subsurface conditions at the project site with seven (7) geotechnical borings designated B-1 through B-7. Boring explored depths and locations are summarized in Exhibit 4-1. Geotechnical borings B-3 through B-7 were drilled within the roadway of Sandlake Rd between May 24, 2025, through May 28, 2025. Geotechnical borings B-1 and B-2 were drilled along the trailhead embankment between March 31, 2025, through April 1, 2025. The borings were drilled using a CME-55LCX track rig provided and operated by Western States Soil Conservation Inc. (Western States) of Hubbard, Oregon. The boring locations were measured by handheld GPS in the field, and the approximate boring locations are shown on the Site and Exploration Plan, Figure 2.

Exhibit 4-1: Summary of Field Explorations

Location	Geotechnical Boring Designation	Approximate Explored Depth (feet)
Pedestrian Bridge Bent 1	B-1	76.5
Pedestrian Bridge Bent 1	B-2	86.5
Reneke Creek Bridge Bent 1	B-3	70.1
Reneke Creek Bridge Bent 1	B-4	70.1
Beltz Creek Bridge South Abutment	B-5	74.1
Levee Setback Tie to Sandlake Road	B-6	90.1
No Name Creek Culvert	B-7	41.5

A Shannon & Wilson geologist was on-site throughout the exploration program to locate the borings, observe the drilling, collect samples, and log the materials encountered. Details of the exploration program, including descriptions of the techniques used to advance and sample the borings, logs of the materials encountered, and backfill details, are presented in Appendix B, Field Explorations.

5 LABORATORY TESTING

Laboratory tests were performed on selected samples from the explorations to determine the basic index and engineering properties of the soils encountered. The samples we obtained during our field explorations were transported to our laboratory for further examination. The soil testing program included moisture content tests, Atterberg limit tests, oven-dried liquid limit testing, particle-size analyses, and a Corrosivity suite. Testing was performed by Shannon & Wilson at our in-house laboratory in Lake Oswego, Oregon, and by Cooper Testing, Inc. of Palo Alto, California. All tests were performed in accordance with applicable American Society for Testing and Materials (ASTM) International standards. The results of the laboratory tests and brief descriptions of the test procedures are presented in Appendix C, Laboratory Test Results.

6 SUMMARY OF SUBSURFACE CONDITIONS

6.1 General

The explorations and laboratory testing were performed to evaluate geotechnical soil and groundwater conditions for the proposed structures. This section describes the general soil units encountered in our subsurface exploration program and includes an overview of our interpreted geologic conditions at the proposed structures.

Our observations presented in this report are specific to the locations, depths, and times noted on the logs and may not be applicable to all areas of the site. No amount of explorations or testing can precisely predict the characteristics, quality, or distribution of subsurface and site conditions. Potential variation includes, but is not limited to, the following:

- The conditions between and below explorations may be different.
- The passage of time or intervening causes (natural and manmade) may result in changes to site and subsurface conditions.
- Groundwater levels and flow directions may fluctuate due to seasonal, irrigation-related, and recharge source variations.

If conditions different from those described herein are encountered during construction, we should review our description of the subsurface conditions and reconsider our conclusions and recommendations.

6.2 Geotechnical Soil Units

Shannon & Wilson grouped the materials encountered in our field explorations into six (6) geotechnical units, as described below. This interpretation of the subsurface conditions is based on the explorations and regional geologic information from published sources. The geotechnical units are as follows:

- **Roadway Fill**: includes pavement and base aggregate sections, loose to medium dense, Silty Gravel (GM); very soft to soft, Elastic Silt with Sand (MH);
- **Embankment Fill**: very loose to loose, Poorly Graded Sand with Silt (SP-SM);
- **Marsh Deposits**: very soft to soft, Organic Soil (OH);
- **Alluvial Fan Deposits**: loose to very dense, Silty Gravel to Silty Gravel with Sand (GM); soft to stiff, Elastic Silt to Elastic Silt with Sand (MH); medium dense, Silty Sand to Silty Sand with Gravel (SM);
- **Fluvial and Estuarian Deposits**: very loose to very dense, Poorly Graded Sand with Silt (SP-SP); very loose to very dense, Silty Sand to Silty Sand with Gravel (SM); very loose, Silty Gravel (GM); very soft to soft, Elastic Silt (MH); and
- **Alsea Formation**: extremely weak to weak (R0-R2), slightly to moderately weathered, Siltstone/Sandstone; very weak to weak (R1-R2), slightly to moderately weathered, Siltstone.

The geotechnical units listed above were grouped based on their engineering properties, geologic origins, and their distribution in the subsurface. Our interpretation of their distribution in the subsurface is shown on Figure 3, Interpretive Subsurface Cross Section A-

A', and Figure 4, Interpretive Subsurface Cross Section B-B'. The locations of the cross sections are shown on Figure 2. The cross sections are interpretive, and variations in subsurface conditions may exist between the borings. Contacts between the units may be more gradational than shown in the cross sections and in the Drill Logs in Appendix B. The Standard Penetration Test (SPT) N-values shown on the Drill Logs, cross sections, and discussed below are in blows per foot (bpf) as counted in the field (uncorrected). SPT blow counts in gravelly soils may not be representative of actual soil density due to the influence of gravel and cobble clasts on the sampler. The following sections describe the geotechnical unit characteristics in greater detail.

6.2.1 Roadway Fill

Roadway Fill was encountered in borings B-5 and B-7 below base aggregate and extends to depths ranging between 6 to 8.5 feet. The Roadway Fill generally consisted of loose to medium dense, Silty Gravel with Sand (GM), and very soft to soft, Elastic Silt with Sand (MH). Asphalt pavement and base aggregate are included within the Roadway Fill. The asphalt pavement and base aggregate thickness varied between 22 to 32 inches. SPT N-values ranged between 2 and 10 bpf and averaged 6 bpf.

6.2.2 Embankment Fill

Embankment Fill was characterized as material used to construct the levee in the locations of borings B-1 and B-2. The Embankment Fill was encountered from the ground surface to depths of 12.5 and 9.5 feet, respectively, and generally consisted of very loose to loose, Poorly Graded Sand with Silt (SP-SM). Pockets of Silty Sand (SM) were observed in some samples. One selected sample natural moisture content of 37 percent and a fines content of 18 percent. SPT N-values in the unit ranged from 0 to 4 bpf and averaged 2 bpf.

6.2.3 Marsh Deposits

Marsh Deposits were encountered in borings B-3, B-4, B-5, and B-7, and extended to depths ranging between 7.5 and 12.5 feet. The Marsh Deposits generally consisted of high plasticity Organic Soil (OH). Few to little organic wood and debris material was observed within the samples, including what appeared to be interbeds of peat in boring B-7. Natural moisture contents ranged from 83 to 112 percent. Plasticity indices ranged between 31 and 93, with oven-dried liquid limits ranging between 47 and 56 percent. A United Soil Classification System (USCS) designation of OH was assigned after assessment of the natural moisture content and Atterberg limit testing data. SPT N-values ranged from 0 to 2 bpf and averaged 1 bpf.

6.2.4 Alluvial Fan Deposits

Alluvial Fan Deposits were encountered in borings B-1, B-2, B-5, and B-6 between depths of 2.5 to 35.0 feet bgs and ranged in thickness between approximately 4.5 and 16.0 feet. The unit generally consisted of loose to very dense, Silty Gravel and Silty Gravel with Sand (GM); soft to stiff, Elastic Silt to Elastic Silt with Sand (MH); and medium dense, Silty Sand and Silty Sand with Gravel (SM). The Alluvial Fan Deposits are interbedded within the Fluvial and Estuarian Deposits and interpreted to be ancient and modern alluvial fans which were created as fine- and coarse-grained sediment was transported and deposited at the mouth of the stream channels into the bay as they exited the mountains of the Coast Range. Some of the alluvial fans are interpreted to have formed before the Holocene during the ice age, when sea levels were lower, and then as sea levels rose, these alluvial fans were overlain and buried under younger Fluvial and Estuarian Deposits.

Within the unit, the fines ranged from low to medium plasticity, and an Atterberg limit test was performed on a single selected sample which indicated a plasticity index of 16 and a United Soil Classification System (USCS) designation of MH. Disturbed textures were observed with moderate to heavy iron oxide staining. The unit generally ranged in color from red-brown to gray and brown. Natural moisture contents ranged from 30 to 42 percent and averaged 37 percent. Particle size analysis indicated fines ranging between 35 and 39 percent. SPT N-values ranged from 6 to 32 bpf and averaged 19 bpf, with one SPT attempted meeting refusal, where over 50 blows were required to advance the sampler 6 inches.

6.2.5 Fluvial and Estuarian Deposits

Fluvial and Estuarian Deposits were encountered in all borings underlying Fill and Marsh Deposits, and interbedded with Alluvial Fan Deposits. The unit generally consists of very loose to very dense, Poorly Graded Sand with Silt (SP-SP); very loose to very dense, Silty Sand and Silty Sand with Gravel (SM); very loose, Silty Gravel (GM); and very soft to soft, Elastic Silt (MH). Fines in the unit generally ranged from nonplastic to high plasticity. Slight to moderate iron oxide staining was observed in some samples, and the material was typically micaceous. The sand constituent was generally fine to coarse-grained. Natural moisture contents ranged from 23 to 50 percent and averaged 30 percent. Particle size analysis indicated fines ranging between 7 and 46 percent. One Atterberg Limits test was performed on a single selected sample which indicated a plasticity index of 17 and a USCS designation of MH. SPT N-values ranged from 2 to 91 bpf and averaged 28 bpf.

6.2.6 Alsea Formation

The Alsea Formation was encountered in borings B-3, B-4, B-5, and B-6 at depths ranging between 60 and 67 feet, and the borings were terminated within the unit. The Alsea Formation generally consisted of extremely weak to weak (R0-R2), slightly to moderately weathered, Siltstone/Sandstone; and very weak to weak (R1-R2), slightly to moderately weathered, Siltstone.

6.3 Groundwater

The borings were drilled using mud rotary drilling techniques, which introduce fluid into the boring during drilling, making it difficult to discern groundwater levels. ESA provided groundwater conditions for the four sites. Groundwater conditions are summarized in Exhibit 6-1.

Exhibit 6-1: Summary of Groundwater Conditions

Location	Tidally Influenced? (yes or no)	Elevation of Channel Bottom (feet)[1]	Elevation of Mean High Water (feet)	Elevation of Static Groundwater[1]
Pedestrian Bridge	Yes	2.0	7.7	N/A
Reneke Creek Bridge	Yes (partially)	9.4	7.7	N/A
Beltz Creek Bridge	No	20.0	N/A[2]	22.0
No Name Creek Culvert	Yes	4.0	7.7	N/A

NOTES:
1 Estimated values.
2 N/A - Not Applicable.

Based on Exhibit 6-1, we assume groundwater for design is at the Mean High Water elevation of 7.7 feet for Pedestrian Bridge, Reneke Creek Bridge and No Name Creek Culvert. For Beltz Creek Bridge, we assume groundwater for design is at an elevation of 22.0 feet.

6.4 Soil Corrosivity

As mentioned in Section 6.3, the Pedestrian Bridge, Reneke Creek Bridge and No Name Creek Culvert are subjected to marine environment. The Beltz Creek Bridge is not subjected to marine environment.

Analytical testing was performed to evaluate the corrosivity potential of the soil at the project site. The corrosivity testing suite included soil resistivity, pH, chloride content, sulfate content, sulfide content, and oxidation-reduction potential. Analytical testing was

performed by Cooper Testing Laboratory, Inc., Palo Alto, California. Corrosivity test results are present in Appendix C.

Soil corrosivity potential was evaluated based on the guidelines in Section 10.7.5 of the AASHTO LRFD (AASHTO, 2024). Soil corrosivity tests were performed on a soil sample at a depth of 10 feet from boring B-1 for the Pedestrian Bridge and a soil sample at a depth of 2.5 feet from boring B-3 for Reneke Creek Bridge. Soil sample at a depth of 10 feet from boring B-1 for the Pedestrian Bridge is considered to be corrosive based on test results. Soil sample at a depth of 2.5 feet from boring B-3 for Reneke Creek Bridge is considered not to be corrosive based on test results. Corrosion design requirements for steel piles in marine environments are provided in Section 8.3.

7 SEISMIC GROUND MOTIONS AND HAZARD EVALUATIONS

7.1 Site Class

The Seismic Site Class for the bridge and retaining wall seismic design criteria was developed based on the recommended procedure from the recently released AASHTO LRFD 10 (AASHTO, 2024). Prior to LRFD 10, bridge structures and retaining walls were typically designed for ground motions corresponding to a 1,000-year return period under "No Collapse" criteria. LRFD 10 now adopts a design approach based on a "limit state of incipient collapse," targeting a risk of approximately 1.5 percent over a 75-year period for ordinary structures. Since direct shear wave velocity measurements were not available at the project site, shear wave velocities were estimated based on SPT data. Our evaluation indicates that the most critical site class derived from Vs, Vs/1.33, and 1.3Vs corresponds to Site Class E. Site Class E is defined by soft soil having an average shear wave velocity less than 500 feet per second (AASHTO, 2024).

7.2 Seismic Design Ground Motions

The ground motion parameters were obtained from the AASHTO-USGS Seismic Design Ground Motion Database (AASHTO, 2023), provided in Exhibit 7-1.

Exhibit 7-2: Design Response Spectrum for Site Class E

Period (s)	Spectral Acceleration (g)
0	0.755
0.01	0.755
0.02	0.787
0.03	0.818
0.05	0.889
0.075	1.02
0.1	1.19
0.15	1.33
0.2	1.41
0.25	1.5
0.3	1.62
0.4	1.67
0.5	1.64
0.75	1.43
1	1.19
1.5	0.92
2	0.722
3	0.468
4	0.342
5	0.265
7.5	0.153
10	0.116

NOTES:
g = acceleration due to gravity; s = seconds.

7.3 Seismic Hazard Evaluation

7.3.1 General

The expected seismic hazards at the project site are primarily ground shaking, tsunamis, liquefaction, and liquefaction-related phenomena such as settlement and post-seismic soil strength reduction. Other seismic hazards, such as fault rupture, and seiche, are not considered hazards for this site. The following sections include a discussion of the relevant seismic hazards present at the project site.

7.3.2 Tsunami

The Project area within the Oregon Coast lies adjacent to the Cascadia megathrust subduction zone, capable of producing tsunamis during an earthquake event. Please see he geologic hazard report titled "Geologic Hazard Report, Sitka Sedge Tidal Wetland Restoration, Tillamook, Oregon," dated October 2025, prepared by Shannon & Wilson, fof further discussions.

7.3.3 Liquefaction

Soil liquefaction is a phenomenon in which excess pore water pressure of loose to medium dense, saturated, granular soils increases during ground shaking to a level near the initial effective stress. The increased excess pore pressure results in a reduction of soil shear strength. The effects of liquefaction typically include lateral spreading, slope instability, and ground settlement.

Important factors in evaluating a soil's susceptibility to liquefaction include relative density, the fines content (percent of soil by weight smaller than 0.075 millimeters, passing the No. 200 sieve), and the plasticity characteristics of the fines. Relative density can be estimated from SPT N-values that were performed for this project. We performed laboratory Atterberg Limits testing to evaluate the plasticity of the soils encountered at the project site.

7.3.3.1 Screening

We evaluated the geotechnical borings B-1 to B-5 at the project site for potentially liquefiable soil layers. We evaluated the liquefaction potential of the soils considered susceptible by our screening analysis using the SPT-based liquefaction evaluation procedures of Boulanger and Idriss (2014) method. The SPT-based procedures evaluate the earthquake loading based on the estimated peak ground acceleration (F_{PGA} x PGA), A_s, and Mw of the design event. In accordance with 2024 AASHTO LRFD 10, we used the following ground motion parameters (PGA and M) in our liquefaction analyses:

- Site-adjusted peak ground acceleration (A_S) of 0.755g and moment magnitude of 9 for a 1,000-year return period..

Based on the results of our analysis, we estimate that loose to medium dense Poorly Graded Sand with Silt (SP-SM) and Silty Sand (SM) from Fluvial and Estuarian Deposits, and loose to medium dense Silty Gravel with Sand (GM) from Alluvial Fan Deposit will liquefy during the design event. A summary of the results of the liquefaction analysis is presented in Exhibit 7-2.

Exhibit 7-1: Summary of Results from Liquefaction Analysis

Exploration ID	Approximate Ground Surface Elevation (feet)	Approximate Groundwater Depth (feet)	Approximate Groundwater Elevation (feet)	Approximate Liquefiable Layer Elevation (feet)	Approximate Liquefiable Layer Depth (feet)	Boring Bottom Depth (feet)
B-1	14.5	6.8	7.7	7.7 to -8.5, -44.0 to -52.0	6.8 to 23, 58.5 to 66.5	76
B-2	14.8	7.1	7.7	7.7 to -5.2, -13.2 to -20.2, -24.2 to -27.2, -37.2 to -43.2	7.1 to 20, 28 to 35, 39 to 42, 52 to 58	86
B-3	15.0	7.3	7.7	7.7 to 2, -3 to -5, -28 to -33, -38 to -43	7.3 to 13, 18 to 23, 43 to 48, 53 to 58	70
B-4	15.0	7.3	7.7	3 to -5, -27 to -43	12 to 20, 42 to 58	70
B-5	32.5	10.5	22.0	4.5 to 0	28 to 32.5	74

7.3.3.2 Liquefaction-Induced Settlement

We evaluated the potential liquefaction-induced settlement of the soil layers considered susceptible by our screening analysis using the Boulanger and Idriss (2014) method. We anticipate that Fluvial and Estuarian Deposits and Alluvial Fan Deposits are susceptible to seismically-induced settlement during the 1,000-year return period seismic event. Susceptible liquefaction zone begins below groundwater level and extends to a depth of 66.5 feet (elevation -52 feet). Settlement may occur in cohesionless soil that undergoes liquefaction and pore pressure development during ground shaking. The settlement is related to densification and rearrangement of particles during ground shaking, as well as volume change, as the excess pore pressure dissipates after ground shaking. Seismic ground settlement may not occur uniformly over an area, and differential settlement may impact existing or proposed structures supported by liquefied soil. Liquefaction-induced settlement magnitudes were estimated using Ishihara and Yoshimine (1992). Results of our seismically induced settlement analyses are summarized in Exhibit 7-3.

Exhibit 7-2: Summary of Results of Liquefaction-Induced Settlement Analysis

Exploration ID	Estimated Liquefaction-Induced Settlement at the Existing Ground Surface (inches)
B-1	11
B-2	10
B-3	6
B-4	6
B-5	1

Differential seismic settlement may range between 50 and 100 percent of total settlement. This degree of settlement may cause damage to the proposed structures if not mitigated.

If deep foundations are used for foundation support, liquefaction-induced settlement will result in downdrag forces on the deep foundation elements.

7.3.4 Lateral Spreading

We estimated the shear strength of the liquefied soil using methods recommended in the ODOT GDM (ODOT, 2024) and other standard methods. These methods include Idriss and Boulanger (2007) and Kramer (2008). In general, these methods base the liquefied (residual) soil shear strength on adjusted SPT N-values $(N_1)_{60}$ or $(N_1)_{60\text{-}cs}$. For our analysis, we estimated the residual shear strength by taking the average of the residual shear strengths determined using the recommended methods. Our analysis indicates that the shear strength of the liquefied Sand (Fluvial and estuarine deposits) should be reduced by 60% of the static shear strength.

Based on our liquefaction evaluation, we predict that liquefaction-induced permanent ground deformation (lateral spreading and flow failure) may occur at the bridge abutments without mitigation. Lateral spreading occurs when the shear strength of liquefied soil is incrementally exceeded by the inertial forces induced during an earthquake, which produces permanent soil strain. Flow failures are driven by large static stresses that exceed the residual shear strength of the soil following liquefaction triggering and lead to large deformations, or flow. The ODOT GDM (ODOT, 2024) notes that flow failure should be considered likely when the post-seismic FS is less than 1.05. Flow failure typically occurs near the end of, or shortly after, strong ground shaking, whereas lateral spreading occurs during shaking.

Our global stability analysis of the proposed abutments for each bridge is detailed in Sections 8.3.2, 8.4.2, and 8.5.2. For abutments at the Pedestrian Bridge and Beltz Creek Bridge, additional abutment support or scour protection is needed to prevent lateral

spreading. Based on discussion with DEA, we understand the bridge replacement bent foundations will be designed to tolerate the anticipated lateral loads due to lateral spreading and flow failure.

8 PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

8.1 General

Design recommendations are based on the preliminary plans and additional information provided by DEA, and our review of the available existing information. Geotechnical design recommendations are provided for the proposed bridges in the following sections. Also, key construction considerations were developed associated with the geotechnical design recommendations. If foundation types or configurations change after this report, Shannon & Wilson should be contacted to provide updated recommendations.

We understand that the bridge foundations and wing walls will be designed considering the following manuals and specifications:

- AASHTO LRFD Bridge Design Specifications (AASHTO LRFD), 10th Edition, 2024;
- ODOT Geotechnical Design Manual (ODOT GDM), May 2024;
- ODOT Oregon Standard Specifications for Construction (ODOT OSSC), 2024; and
- Applicable FHWA geotechnical guidelines.

8.2 Bridge Foundation Design Alternatives

The selection of an appropriate foundation system for the proposed bridge structure is dependent upon several factors, including foundation capacities, tolerance to total and differential settlement resulting from static loads, and construction considerations. Based on the explored subsurface conditions and the design loads, the sites are susceptible to liquefaction, and therefore, we considered steel-driven pipe piles, steel H piles, and drilled shaft foundations. A comparison of these three types of foundations is presented in Exhibit 8-1.

Exhibit 8-1: Comparison of Bridge Foundation Alternatives

Foundation	Advantages	Disadvantages
Steel Driven Pipe Piles (Closed-end)	Capable of mobilizing higher-end bearing resistance; Feasible for proposed construction sequence; Relatively fast construction; Lower cost than drilled-in foundations;	Risk of premature refusal prior to achieving the minimum required tip elevation in dense or obstructed soil layers may require preboring; Potential significant negative skin friction (downdrag) in liquefiable soils;
Steel Driven Pipe Piles (Open-end)	Enhanced drivability, enabling installation to the target penetration depth with reduced risk of refusal; Feasible for proposed construction sequence; Relatively fast construction; Lower cost than drilled-in foundations;	Potential significant negative skin friction (downdrag) in liquefiable soils; May require greater embedment depth to achieve design capacity;
Steel Driven H-Piles	Enhanced drivability, enabling installation to the target penetration depth with reduced risk of refusal; Feasible for proposed construction sequence; Relatively fast construction; Lower cost than drilled-in foundations;	Potential significant negative skin friction (downdrag) in liquefiable soils; May require greater embedment depth to achieve design capacity;
Small- to Medium Diameter Drilled Shafts (2- to 4-foot diameter)	Higher level of control of construction variability compared to driven piles; Provides higher lateral resistance than driven piles; Feasible for proposed construction sequence; Can be constructed using non-vibratory methods;	Relatively more expensive than driven piles; Require a specialty contractor; Highest construction QA/QC requirements; Relatively longer construction duration compared to driven piles; High mobilization/demobilization costs compared to driven piles; Potential significant negative skin friction (downdrag) in liquefiable soils;

Based on our deep foundation evaluation and discussion with DEA, a summary of bridge foundation selection for each bridge is presented in Exhibit 8-2. Details are discussed in each individual section.

Exhibit 8-2: Bridge Foundation Selection Recommendation

Bridge	Bent	Pile Type
Pedestrian Bridge	Bents 1 and 2	Steel pipe pile (Closed-end)
Reneke Creek Bridge	Bents 1 and 2	Steel H-pile
Beltz Creek Bridge	North and South Abutments	Steel H-pile

8.3 Steel Pile Corrosion

Corrosion of steel piles should be anticipated due to the marine environment and the alternate wetting and drying from tidal action present at the Pedestrian Bridge, Reneke Bridge, and No Name Creek Cuvlert sites. According to the Federal Highway Administration (FHWA) Geotechnical Engineering Circular (GEC) 012 - Volume I, Exhibit 8-3 presents the average and maximum probable marine corrosion rates for steel piles in seawater. The laboratory test results for pH and corrosivity are included in Appendix C.

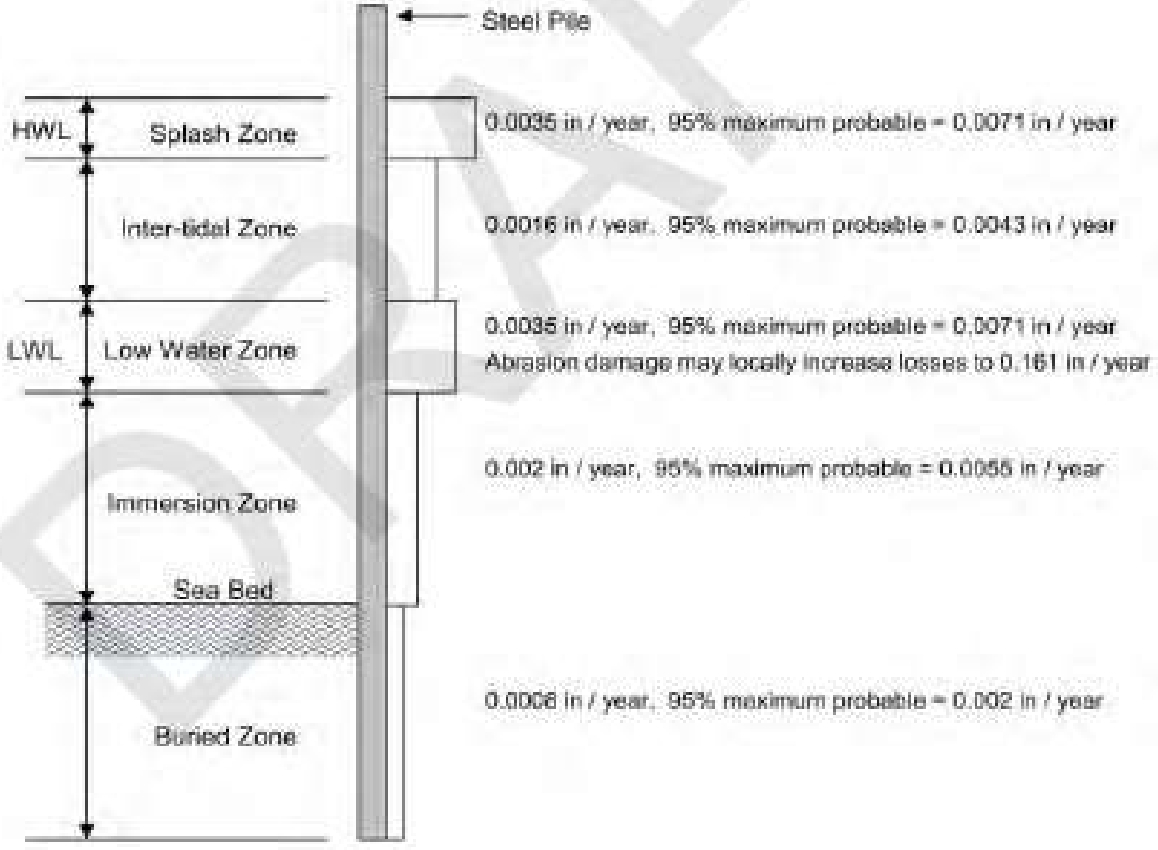


Exhibit 8-3: Loss of Thickness by Corrosion for Steel Piles in Seawater (Figure 6-29; FHWA, 2016)

8.4 Preliminary Foundation Design Recommendations for Pedestrian Bridge

8.4.1 General

Based on 60 design documents included in Appendix A, each bent of the proposed single-span Pedestrian Bridge will be supported on three (3) 20-inch diameter closed-end steel pipe piles. Based on our discussion with DEA, the foundation type, size, and depth will be redesigned during final design phase.

8.4.2 Bridge Abutment Global Stability

We conducted a global stability analysis for the proposed bridge abutments using the computer program SLOPE/W, Version 2024.2.1. This program employs limit equilibrium methods in accordance with AASHTO. The Morgenstern-Price slope stability analysis method was used for a rotational surface failure mechanism. The analysis was performed for static, seismic, and post-seismic conditions. A live load of 250 pounds per square foot (psf) was assumed for the static condition. For seismic global stability analyses, pseudo-static procedures described in the ODOT GDM Chapter 7 were followed. Horizontal acceleration coefficients equal to one-half of the site peak ground accelerations (0.5 x PGA) were used. For our seismic global stability analysis, we used a horizontal seismic coefficient equal to 0.377. An ordinary high-water design elevation of 7.7 feet was assumed in the channel for all cases.

The ODOT GDM requires that slopes supporting bridge foundations be designed with a maximum resistance factor for global stability of 0.65, equivalent to a factor of safety (FS) of 1.5, for the static condition. For seismic and post-seismic conditions, a maximum resistance factor of 0.9, or an FS of 1.1, is required. Per the ODOT GDM, a factor of safety below 1.05 for the post-seismic case indicates that a flow failure is likely.

Global stability analysis results and soil parameters are presented in Exhibit 8-4 and in Figures D1 through D-3 in Appendix D. Based on our analysis, the proposed bridge abutments will satisfy the minimum global stability FS requirements for static conditions. However, the FS for seismic and post-seismic conditions are less than 1.1, and therefore, lateral spreading and flow failure toward the creek will be an issue at the site, and the potential impacts of the embankment slope failures include significant settlement of the approach to the bridge and additional lateral loading on the foundations.

Based on discussion with DEA, we understand the bridge abutments will be designed to tolerate the anticipated additional lateral loads due to seismic and post-seismic effects, e.g.,

lateral spreading and flow failure, but the approach embankments and roadway will be allowed to deform and settle significantly in the post-seismic case.

Exhibit 8-4: Global Stability Analysis Results at Pedestrian Bridge

Type of Analysis	Required Factor of Safety	Factor of Safety
Static Condition	1.5	2.0
Seismic Condition	1.1	0.9
Post-Seismic Condition	1,1	0.4

Based on our global stability analysis, the bridge foundation should be designed to resist lateral spreading loads at each abutment for seismic and post-seismic conditions. The calculated factors of safety account for the lateral resistance that will need to be provided by the bridge foundations due to static and seismic active lateral earth pressures on the abutment wall (see Section 8.4.3.5). This resistance is modeled as a point load on the abutment wall at the location of the resultant force from the triangularly distributed earth pressures.

8.4.3 Driven Pipe Pile Design Recommendations

8.4.3.1 General

Based on observed subsurface conditions encountered in boring B-1 and B-2, the pile axial compressive resistances were evaluated for a 20-inch-diameter closed-end pipe pile with 0.625-inch wall thickness. The recommended pile lengths are estimated as the distance between the bottom of the pile cap elevation and the estimated pile tip elevation required to resist the anticipated loading.

Based on liquefaction analysis, the highly variable potential liquefiable layers extend to an approximate elevation of -52 feet, and therefore, we recommend that the pipe pile should be extended below an elevation of -52 feet. In addition, pre-boring may be required to drive through dense to very dense sand between approximate depths of 22 to 27 bgs (elevations -7 and -12 feet) and 38 to 57 bgs (elevations -23 and -42 feet) at Bent 1 (boring B-1) and 22 to 27 bgs (elevations -7 and -12 feet) at Bent 2 (boring B-2) to reach the minimum required pile tip elevation.

8.4.3.2 Driven Pile Axial Resistance

We recommend that the steel piles conform to the requirements of ASTM A252, Grade 3. Mill certification of the steel should be provided by the supplier. All portions of pile design and construction should meet the requirements of ODOT OSSC, Section 00520, and its project special provisions. Exhibit 8-5 presents the typical pile section design properties.

Exhibit 8-5: Steel Pipe Pile Section Properties

Pile Diameter x Thickness (inches)	Steel Grade (kips/inch²)	Section Area (inch²)	End Condition
PP 20 x 0.625	A252 Grade 3	38.04	Reinforced Flat Plate

The estimated nominal and factored pile compressive resistances for the recommended 20-inch-diameter closed-end pipe piles are presented in Figures E1 and E2 in Appendix E and summarized below in Exhibit 8-6. The estimated nominal compressive resistance is established using a resistance factor of 0.4, assuming the FHWA Gates equation will be used to establish the pile driving criteria (see Section 8.6.1.1). The estimated settlements are less than 1 inch at the maximum loads applied to the piles under the strength limit state loadings; therefore, the nominal resistance may be considered the nominal service limit state resistance as well. We recommend a resistance factor of 1.0 be applied to the nominal resistance for the service limit state and extreme event limit state evaluations.

Exhibit 8-6: Steel Pipe Pile Length and Compressive Resistance for Pedestrian Bridge Foundations

Location	Bottom of Pile Cap Elevation (feet)	Estimated Driven Length[1] (feet)	Estimated Pile Tip Elevation (feet)	Nominal Axial Compressive Resistance (kips)	Pile Driving Criteria	Factored Axial Compressive Resistance (kips)		
						Strength Limit	Service Limit	Extreme Event Limit
Bent 1	8.5	68.5	-60	360	Gates Eq.	145	360	110
Bent 2	8.5	68.5	-60	200	Gates Eq.	90	200	100

NOTES:

1 Estimated driven pile length, taken as the distance between bottom of pile cap and estimated pile tip elevation, is assumed to be +/-2 feet of the table value.

The axial compressive loads for Strength Limit and Extreme Event Limit were not provided. The axial resistances for Service Limit do meet factored structural design load of 133 kips per pile. We discussed the axial loads with DEA. The deep foundation will require redesign during final design. For final design, DEA will consider drilled shaft foundation or hanging the pipe pile on dense to very dense sand layer at approximate depths of 25 feet at Bent 1 and 45 feet at Bent 2, and accept seismic settlement up to 2 inches for each pile.

The geotechnical resistances of an individual pile under strength, service, or extreme event limit state shall not be greater than the pile's structural capacity. The structural engineer should evaluate the pile structural capacity in accordance with AASHTO LRFD Section 6.

The estimated nominal axial resistances assume the piles are spaced at least 2.5 pile diameters apart, measured center-to-center. Based on this assumption, the pile group effects are not considered. If the pile spacing is reduced during final design, the appropriate pile

efficiency factor must be established and applied, as recommended by AASHTO LRFD Section 10.7.3.9.

8.4.3.3 Driven Pile Seismic Downdrag Loads

Based on post-liquefaction settlement analysis, the driven piles at both bents will experience seismic downdrag loads. At Bent 1, the seismic downdrag load will be on the order of 210 kips per pile if the pile is embedded at the recommended minimum tip elevation. At Bent 2, the seismic downdrag load will be on the order of 110 kips per pile if the pile is embedded at the recommended minimum tip elevation. A load factor of 1.0 is recommended to be applied to this seismic downdrag force in the Extreme Event limit state.

In accordance with AASHTO LRFD Section 10.7.1.6.2, the downdrag load does not impact the nominal geotechnical resistance and does not need to be considered in geotechnical limit state evaluations. However, the downdrag load should be considered in structural limit state evaluations of the pile section.

8.4.3.4 Driven Pile Lateral Resistance

The driven pile foundations will be subjected to lateral loads resulting from live, seismic, and abutment global instability loading. We understand that the laterally loaded pile analyses will be performed with the aid of the LPILE computer program. Geotechnical input parameters for LPILE are provided in Table 1, attached at the end of this report, for the static/seismic conditions.

The estimated lateral resistance parameters presented in Table 1 are recommended for driven piles with center-to-center spacing greater than five pile diameters (5D) and in a single row. Based on this assumption, the pile group effects are not considered. If pile spacing is less than 5D or multiple rows of piles are required, the appropriate P-Multiplier must be established and applied, as recommended by AASHTO LRFD Section 10.7.2.4.

8.4.3.5 Additional Lateral Load Due to Global Instability

The global stability analysis results present that seismic and post-seismic FS of less than 1.1, indicating seismic failure and flow failure are likely to occur. In the global stability analysis, additional pile reinforcement was added in order to achieve the required FS. Exhibit 8-7 summarizes the additional lateral pile reinforcement needed for FS of 1.1 at seismic and post-seismic conditions. Global stability analysis results and additional lateral load are presented in Exhibit 8-7 and in Figures D4 and D5 in Appendix D. The distributions of these lateral pressures are shown on Figure 5, Estimated Additional Lateral Earth Pressure on Pedestrian Bridge Abutment.

Exhibit 8-7: Global Stability Analysis with Pile Reinforcement at Pedestrian Bridge

Type of Analysis	Factor of Safety	Additional Lateral Load Needed (lb/feet)
Seismic Condition	1.1	2,400
Post-Seismic Condition	1.1	2,700

8.5 Preliminary Foundation Design Recommendations for Reneke Creek Bridge

8.5.1 General

Based on 60% design documents, each bent of the proposed single span Reneke Creek Bridge will be supported on eight (8) steel H-piles, HP 16x141.

8.5.2 Bridge Abutment Global Stability

Global stability at the Reneke Bridge abutment is evaluated in a similar manner as described in Section 8.3.1.

Global stability analysis results are presented in Exhibit 8-8 and in Figures D6 through D8 in Appendix D. Based on our analysis, the proposed bridge abutments will satisfy the minimum global stability FS requirements for static, seismic, and post-seismic conditions.

Exhibit 8-8: Global Stability Analysis Results at Reneke Creek Bridge

Type of Analysis	Required Factor of Safety	Factor of Safety
Static Condition	1.5	2.6
Seismic Condition	1.1	1.2
Post-Seismic Condition	1.1	1.3

8.5.3 Driven Pile Design Recommendations

8.5.3.1 General

Based on observed subsurface conditions encountered in boring B-3 and B-4, the pile axial compressive resistances were evaluated for HP 16x141. The recommended pile lengths are estimated as the distance between the bottom of the pile cap elevation and the estimated pile tip elevation required to resist the anticipated loading.

Based on liquefaction analysis, the highly variable potential liquefiable layers extend to an approximate elevation of -43 feet, and therefore, we recommend that the H-piles should be extended below the elevation of -43 feet to Siltstone/Sandstone (Alsea Formation at an approximate elevation of -49 feet.

8.5.3.2 Driven Pile Axial Resistance

We recommend that the steel piles conform to the requirements of ASTM A572, Grade 50. Mill certification of the steel should be provided by the supplier. Because driving of the piles on rock is anticipated, the H-pile tips should be reinforced with prefabricated cast steel points. All portions of pile design and construction, including pile tip reinforcement, should meet the requirements of OSSC Section 00520 and its project special provision. Exhibit 8-9 presents the typical pile section design properties.

Exhibit 8-9: Steel H-Pile Section Properties

Pile Diameter x Thickness (inches)	Steel Grade (kipa/inch^2)	Section Area (inch^2)	End Condition
HP 16x141	A572 Grade 50	41.7	Reinforced

The estimated nominal and factored pile compressive resistances for the recommended HP 16x141 are presented in Figures E3 and E4 in Appendix E and summarized below in Exhibit 8-10. The estimated nominal compressive resistance is established using a resistance factor of 0.4, assuming the FHWA Gates equation will be used to establish the pile driving criteria (see Section 8.6.1.1). The estimated settlements are less than 1 inch at the maximum loads applied to the piles under the strength limit state loadings; therefore, the nominal resistance may be considered the nominal service limit state resistance as well. We recommend a resistance factor of 1.0 be applied to the nominal resistance for the service limit state and extreme event limit state evaluations.

Exhibit 8-10: Estimated Driven Steel H-Pile Length and Compressive Resistance

Location	Bottom of Pile Cap Elevation (feet)	Estimated Driven Length[1] (feet)	Estimated Pile Tip Elevation (feet)	Nominal Axial Compressive Resistance (kips)	Pile Driving Criteria	Factored Axial Compressive Resistance (kips)		
						Strength Limit	Service Limit	Extreme Event Limit
Bent 1	12.5	64.5	-52	410	Gates Eq.	165	410	310
Bent 2	12.5	64.5	-52	400	Gates Eq.	160	400	300

NOTES:

1 Estimated driven pile length, taken as the distance between bottom of pile cap and estimated pile tip elevation, is assumed to be +/-2 feet of the table value.

The geotechnical resistances of an individual pile under strength, service, or extreme event limit state shall not be greater than the pile's structural capacity. The structural engineer should evaluate the pile structural capacity in accordance with AASHTO LRFD Section 6.

The estimated nominal axial resistances assume the piles are spaced at least 2.5 pile diameters apart, measured center-to-center. Based on this assumption, the pile group effects

are not considered. If, during final design, the pile spacing is reduced, the appropriate pile efficiency factor must be established and applied, as recommended by AASHTO LRFD Section 10.7.3.9.

8.5.3.3 Driven Pile Seismic Downdrag Loads

Based on post-liquefaction settlement analysis, the driven piles at both bents will experience seismic downdrag loads. At both bents, the seismic downdrag load will be on the order of 80 and 70 kips at Bents 1 and 2, respectively, per pile. A load factor of 1.0 is recommended to be applied to this seismic downdrag force in the Extreme Event limit state.

In accordance with AASHTO LRFD Section 10.7.1.6.2, the downdrag load does not impact the nominal geotechnical resistance and does not need to be considered in geotechnical limit state evaluations. However, the downdrag load should be considered in structural limit state evaluations of the pile section.

8.5.3.4 Driven Pile Lateral Resistance

The driven pile foundations will be subjected to lateral loads resulting from live, seismic, and abutment global instability loading. We understand that the laterally loaded pile analyses will be performed with the aid of the LPILE computer program. Geotechnical input parameters for LPILE are provided in Table 2, attached at the end of this report, for the static/seismic.

The estimated lateral resistance parameters presented in Table 1 are recommended for driven piles with center-to-center spacing greater than five pile diameters (5D) and in a single row. Based on this assumption, the pile group effects are not considered. If pile spacing is less than 5D or multiple rows of piles are required, the appropriate P-Multiplier must be established and applied, as recommended by AASHTO LRFD Section 10.7.2.4.

8.6 Preliminary Foundation Design Recommendations for Beltz Creek Bridge

8.6.1 General

The Beltz Creek Bridge is at the 30 percent design phase. Based on our discussion with DEA, the foundation for the Beltz Creek Bridge will be designed in a later design phase. We understand that the proposed Beltz Creek Bridge structure may be similar to Reneke Bridge, and therefore, we assumed the proposed Beltz Creek Bridge will be supported on steel H-piles. We also assumed that each abutment of the proposed single-span Beltz Creek Bridge will be supported on eight (8) steel H-piles. We provided design recommendations for HP 14x117 and HP 16x141.

8.6.2 Bridge Abutment Global Stability

As described in section 8.3.1, global stability at Beltz Creek Bridge abutment is evaluated in a similar manner.

To mitigate potential scour and maintain abutment stability, a 4-foot-thick riprap layer is recommended along the creek bed, replacing the existing marsh and alluvial fan deposits near the abutment. Global stability analysis results and soil parameters are presented in Exhibit 8-11 and in Figures D9 through D11 in Appendix D. Based on our analysis, the proposed bridge abutments will satisfy the minimum global stability FS requirements for static and post-seismic conditions. However, the FS for seismic conditions is less than 1.1.

Based on discussion with DEA, we understand the bridge abutments will be designed to tolerate the anticipated additional lateral loads due to seismic effects.

Exhibit 8-11: Global Stability Analysis Results at Beltz Creek Bridge

Type of Analysis	Required Factor of Safety	Factor of Safety
Static Condition	1.5	1.5
Seismic Condition	1.1	1.0
Post-Seismic Condition	1.1	1.5

Based on our global stability analysis, the bridge foundation should be designed to resist the additional seismic lateral loads at each abutment for seismic conditions. The calculated factors of safety account for the lateral resistance that will need to be provided by the bridge foundations due to seismic active lateral earth pressures on the abutment wall (see Section 8.6.3.4). This resistance is modeled as a point load on the abutment wall at the location of the resultant force from the triangularly distributed earth pressures.

8.6.3 Driven Pile Design Recommendations

8.6.3.1 General

Based on observed subsurface conditions encountered in boring B-5, the pile axial compressive resistances were evaluated for HP 14x117 and HP 16x141. The recommended pile lengths are estimated as the distance between the bottom of the pile cap elevation and the estimated pile tip elevation required to resist the anticipated loading.

Based on liquefaction analysis, the highly variable potential liquefiable layers extend to an approximate elevation of -5 feet, and therefore, we recommend that the H-piles should be extended below the elevation of -5 feet to Siltstone/Sandstone (Alsea Formation at approximate elevation of -35 feet.

8.6.3.2 Driven Pile Axial Resistance

We recommend that the steel piles conform to the requirements of ASTM A572, Grade 50. Mill certification of the steel should be provided by the supplier. Because driving of the piles on rock is anticipated, the H-pile tips should be reinforced with prefabricated cast steel points. All portions of pile design and construction, including pile tip reinforcement, should meet the requirements of OSSC Section 00520 and its project special provision. Exhibit 8-12 presents the typical pile section design properties.

Exhibit 8-12: Steel H-Pile Section Properties

Pile Diameter x Thickness (inches)	Steel Grade (kipsinch^2)	Section Area (inch^2)	End Condition
HP 14x117	A572 Grade 50	34.4	Reinforced
HP 16x141	A572 Grade 50	41.7	Reinforced

The estimated nominal and factored pile compressive resistances for the recommended HP 14x117 and HP 16x141 are presented in Figures E5 and E6 in Appendix E and summarized below in Exhibit 8-13. The estimated nominal compressive resistance is established using a resistance factor of 0.4, assuming the FHWA Gates equation will be used to establish the pile driving criteria (see Section 8.6.1.1). The estimated settlements are less than 1 inch at the maximum loads applied to the piles under the strength limit state loadings; therefore, the nominal resistance may be considered the nominal service limit state resistance as well. We recommend a resistance factor of 1.0 be applied to the nominal resistance for the service limit state and extreme event limit state evaluations.

Exhibit 8-13: Estimated Driven Steel H-Pile Length and Compressive Resistance

H-Pile	Bottom of Pile Cap Elevation (feet)	Estimated Driven Length[1] (feet)	Estimated Pile Tip Elevation (feet)	Nominal Axial Compressive Resistance (kips)	Pile Driving Criteria	Factored Axial Compressive Resistance (kips)		
						Strength Limit	Service Limit	Extreme Event Limit
14x117	26.0	73.0	-47	350	Gates Eq.	160	350	300
16x141	26.0	73.0	-47	390	Gates Eq.	180	390	350

NOTES:

1 Estimated driven pile length, taken as the distance between bottom of pile cap and estimated pile tip elevation, is assumed to be +/-2 feet of the table value.

The geotechnical resistances of an individual pile under strength, service, or extreme event limit state shall not be greater than the pile's structural capacity. The structural engineer should evaluate the pile structural capacity in accordance with AASHTO LRFD Section 6.

The estimated nominal axial resistances assume the piles are spaced at least 2.5 pile diameters apart, measured center-to-center. Based on this assumption, the pile group effects are not considered. If, during final design, the pile spacing is reduced, the appropriate pile efficiency factor must be established and applied, as recommended by AASHTO LRFD Section 10.7.3.9.

8.6.3.3 Driven Pile Seismic Downdrag Loads

Based on post-liquefaction settlement analysis, the driven piles at both bents will experience seismic downdrag loads. At both bents, the seismic downdrag load will be on the order of 40 kips per pile. A load factor of 1.0 is recommended to be applied to this seismic downdrag force in the Extreme Event limit state.

In accordance with AASHTO LRFD Section 10.7.1.6.2, the downdrag load does not impact the nominal geotechnical resistance and does not need to be considered in geotechnical limit state evaluations. However, the downdrag load should be considered in structural limit state evaluations of the pile section.

8.6.3.4 Driven Pile Lateral Resistance

The driven pile foundations will be subjected to lateral loads resulting from live, seismic, and abutment global instability loading. We understand that the laterally loaded pile analyses will be performed with the aid of the LPILE computer program. Geotechnical input parameters for LPILE are provided in Table 3, attached at the end of this report, for the static/seismic.

The estimated lateral resistance parameters presented in Table 1 are recommended for driven piles with center-to-center spacing greater than five pile diameters (5D) and in a single row. Based on this assumption, the pile group effects are not considered. If pile spacing is less than 5D or multiple rows of piles are required, the appropriate P-Multiplier must be established and applied, as recommended by AASHTO LRFD Section 10.7.2.4.

8.6.3.5 Additional Lateral Load Due to Global Instability

The global stability analysis results indicate that seismic FS is less than 1.1 for the proposed bridge abutments. In the global stability analysis, additional pile reinforcement was added in order to achieve the required FS. Exhibit 8-14 summarizes the additional lateral pile reinforcement needed for FS of 1.1 at seismic condditions. Global stability analysis results and additional lateral load are presented in Exhibit 8-14 and in Figure D12 in Appendix D. The distributions of these lateral pressures are shown on Figure 6, Estimated Additional Lateral Earth Pressure on Beltz Creek Bridge Abutment.

Exhibit 8-14: Global Stability Analysis with Pile Reinforcement at Beltz Creek Bridge

Type of Analysis	Factor of Safety	Additional Lateral Load Needed (lbf/feet)
Seismic Condition	1.1	1,700

8.6.4 Additional Field Explorations for Beltz Creek Bridge

Boring B-5 was located approximately at the south abutment of the Beltz Creek Bridge. We recommend that one boring be performed at approximately north abutment for the final design. The boring should advance to a depth of 85 feet. Oregon Department of Transportation Geotechnical Design Manual (ODOT GDM) and American Association of State Highway and Transportation Officials (AASHTO) require one boring at each abutment for foundation design.

8.7 Pile Foundation Construction Considerations

8.7.1 Pile Driving Criteria and Construction Considerations

8.7.1.1 Pile Driving Criteria

As previously stated, we recommend that pile driving and installation of piles follow the OSSC, Section 00520. If splicing pile lengths in the leads is necessary to install the piles, then splicing locations should be approved by the structural engineer. All pile splices should be made according to the OSSC and procedures for piling with lateral and tension loading conditions. Also, the piles should be driven no closer together than 2.5 pile diameters, measured from center to center, and within 6 inches of locations shown on the plans. The pile driving alignment tolerance should follow the OSSC.

The piles should be driven with an impact hammer such as a single or double-acting air, steam, or diesel hammer. The driving energy of a pile hammer used for piles with a nominal resistance less than or equal to 600 kips may be qualified using Table 00520-1 in the OSSC, Section 00520, which is based on the FHWA Gates Equation. According to the OSSC, a wave equation analysis is required if the piles are to be driven with a double-acting or differential hammer, regardless of required nominal resistance, or if the required nominal resistance is greater than 600 kips.

The piles should be driven to a minimum "last set" and to a minimum pile length of 12 feet as required by the OSSC, Section 00520.41c. If the "last set" is reached before the minimum tip elevation or minimum pile embedment, driving should continue until the minimum design requirements are reached. If pile refusal occurs before the minimum tip elevation is reached, the contractor should pre-bore the hole and continue to drive the pile.

Prior to construction, pile driving criteria, including "last set," should be established for the specific pile-driving equipment proposed for use. The hammer selected by the contractor should be capable of achieving the required nominal resistance at a blow count between 2 and 10 blows per inch, as determined by the FHWA Gates Equation specified in the OSSC, Section 00520.42(b). The FHWA Gates Equation is only applicable if the hammer is in good condition and operating in a satisfactory manner, the hammer has a free fall, a follower is not used, and the head of the pile is not broomed or crushed.

During pile driving, a continuous record of pile driving resistance (bpf) should be maintained for the full length of each pile driven, as well as other pertinent information, including observed hammer performance. If an open-end diesel hammer is used to drive the piles, the pile-driving contractor should supply a Saximeter™ during pile driving to determine the actual stroke height and pile driving energy. We recommend that an engineering staff representative, under the guidance of a professional geotechnical engineer, monitor pile driving in order to evaluate the suitability of each pile driven.

8.7.1.2 Pile Driving Vibration Impacts

We understand the nearest existing building is approximately 1/2-mile from the proposed pile driving location. Therefore, pile driving vibration on nearby existing buildings and residences may not be a significant concern. However, we recommend the Contractor perform a pre-construction condition survey of the existing structures around the pile driving area to document pre-pile driving conditions and evaluate any possible post-pile driving building damage.

8.7.1.3 Pile Driving Noise

Pile driving will be very noisy and may disturb nearby residents and the environment. The Agency should evaluate any potential noise impacts to surrounding residents. In addition, construction noise should meet local noise ordinances.

8.8 Bridge Abutment and Wing Walls

8.8.1 General

Wing walls for Pedestrian Bridge heights are up to 8 feet and approximately 9 feet in length, parallel to the roadway. Wing walls for Reneke Creek Bridge heights are up to 6 feet and approximately 12 feet in length, parallel to the roadway. Wing walls for Beltz Creek Bridge heights are estimated up to 6 feet, parallel to the road, based on 30 percent design plans. It is understood that these wing walls will be cantilevered from the pile caps and bridge abutments beneath the bridge deck. The wing walls will have a level backslope. This will impose additional loads on the piles supporting the bridges, although lateral loads due to

earth pressures will be partially offset due to the wing wall on the opposite end of the pile cap.

For design purposes, we have assumed that subdrainage systems will be installed to prevent hydrostatic pressure from developing behind all retaining walls. Also, we have assumed that the backfill behind the walls is level.

8.8.2 Lateral Earth Pressures

The lateral earth pressures on the retaining walls depend on the type of wall (i.e., yielding or non-yielding), the type and method of placement of backfill against the wall, the magnitude of surcharge weight on the ground surface adjacent to the wall, the slope of the backfill, and the design criteria. We recommend that at-rest (non-yielding wall) earth pressures be considered at the wing wall/abutment connection. At the ends of the wing walls, active (yielding wall) earth pressures may be used if the length is sufficient to allow yielding of the wall. The earth pressure loads can be linearly interpolated along the length of the wing wall between these values. The abutment walls should be designed as non-yielding walls.

Based on the structural design information and the above assumptions, the lateral earth pressures on the walls were developed according to the ODOT GDM (ODOT, 2024) and AASHTO LRFD Bridge Design Specifications (AASHTO, 2024). The static lateral earth pressure acting on walls consists of two components: static earth pressure and static surcharge pressure. The seismic lateral earth pressure on walls consists of three components: static earth pressure, static surcharge pressure, and seismic earth pressure. A horizontal acceleration coefficient, k_h, equal to the site peak ground acceleration (F_{PGA} x PGA), A_s, was used to determine the seismic earth pressure for non-yielding walls. A k_h equal to 1/2 of A_s was used to determine the seismic earth pressure for yielding walls, where 1 to 2 inches of lateral deformation is acceptable. The distributions of these lateral pressures are shown in Figure 7, Lateral Earth Pressure Distribution on Abutment and Wing Walls.

8.8.3 Subdrainage

Suitable drainage for walls can be provided by granular backfill material and a wall base subdrain system consisting of a 6-inch-diameter perforated or slotted drainpipe wrapped in an envelope of filter material at least 12 inches thick and confined by a separation geotextile. The filter material is specified in Section 02610.10(a) of the ODOT OSSC. The subdrain should be above the typical groundwater level, convey any collected seepage to the end of the wall, and daylight at low spots below the wall elevation.

8.8.4 Backfill Material and Compaction

The wall backfill material should be in accordance with standard ODOT Granular Wall Backfill (Section 00510.12 of the ODOT OSSC). Heavy compaction equipment should not be allowed closer than 3 feet to the retaining wall to prevent high lateral earth pressures and/or wall yielding and/or damage. Required compaction of wall backfill within 3 feet of the walls shall be obtained using hand-operated compaction equipment, such as a vibrating plate compactor.

8.8.5 Lateral Resistances

We understand the static and seismic lateral resistance of the abutment wall can be provided by the pile foundations. We assume the lateral resistance for the wing walls will be generated through the structural connection with the pile cap. If it is determined that bridge foundations cannot adequately support the abutment/wing wall loads, specific foundation design recommendations will be provided upon request.

8.9 No Name Creek Culvert Design Recommendations

8.9.1 General

The following sections present our design recommendations, including bearing resistance, estimated settlements, and lateral earth pressures on culvert walls, and construction considerations for the culverts. If the structure or foundation types and configurations change after this report, Shannon & Wilson should be contacted to provide updated recommendations. We understand that AASHTO LRFD does not require seismic design for pre-cast box culverts. Therefore, no seismic hazard evaluations were performed for this portion of the project.

8.9.2 Culvert Foundation Over-excavation and Subgrade Preparation

Based on boring B-7, very soft to soft and high plasticity organic soils (Marsh Deposit) are encountered between depths of approximately 2 and 11 feet bgs. The proposed bottom of the box culvert will be placed at approximately a depth of 10 feet. The Marsh Deposits are highly compressible. To reduce potential settlement, we recommend that the Marsh Deposits be over-excavated to silty sand soil and replaced with ODOT Granular Wall Backfill in accordance with OSSC Section 00510.12. The crushed rock should be placed and compacted in accordance with OSSC Section 00330. The over-excavation may serve as a crushed rock working mat for the culvert.

8.9.3 Foundation Bearing Resistance and Settlement

We performed bearing resistance analysis in accordance with AASHTO and the ODOT GDM. Based on the explored subsurface conditions, both culverts will be placed on silty sand of Fluvial and Estuarian Deposits. The proposed box culverts may be designed for factored bearing resistances in Exhibit 8-15.. A resistance factor of 0.45 was used in calculating bearing resistance for the strength limit state. A resistance factor of 1.0 was used for the service and extreme event limit states.

Exhibit 8-15: Factored Bearing Resistances for No Name Creek Culvert

Structure	Factored Axial Compressive Resistance (kips)		
	Strength Limit (ksf)	Service Limit (ksf)	Extreme Event Limit (ksf)
No Name Creek Culvert	12.0	2.0	30.0

Total Settlement is estimated to be less than 1 inch, and differential settlement is estimated to be less than 0.5 inch.

8.9.4 Lateral Earth Pressures

Based on the above assumptions, the lateral earth pressures on the box culvert walls were developed according to AASHTO and the ODOT GDM. The lateral earth pressure acting on the culverts consists of two components: earth pressure and surcharge pressure. The lateral earth pressure diagram for the culverts is shown in Figure 8. We assume that the culverts will be backfilled using 3/4-inch-minus ODOT Granular Structure Backfill in accordance with OSSC, Section 00510.13 (ODOT, 2021). We have provided lateral earth pressures dependent on the groundwater level and height of fill above the top of the culvert. The hydraulic engineer should determine the appropriate groundwater level for the culvert design. The critical groundwater level for the culvert design may occur during the design flood drawdown condition, which represents the condition just after a design flood has occurred and the water elevation inside the culvert has receded to the ordinary high water level, but the groundwater level outside the culvert is still at the design flood level. The earth pressures presented are independent of other structural considerations, including wall stiffness and load factoring.

8.10 Trailhead Embankment Stability for Pedestrian Bridge Construction

A trolley similar to a Specialized Hauling Vehicles (VHS) SU6 may be used to transport the prefabricated Pedestrian Bridge superstructure from the staging area (existing parking lot) to the bridge site along the trailhead. The trolley consists of 6 axles total, two (2) axles in the front and four (4) axles in the back. Each of the axle sets will have 5-foot spacing, but there

will be approximately 150 feet between the 2 axles in front and 4 axles in back. Each axle load is approximately 30 kips.

We performed static slope stability of the existing trailhead embankment to support the trolley during construction of the Pedestrian Bridge using the computer program SLOPE/W, Version 2024.2.1. The analysis was performed for static conditions. For live load, we assumed two line loads, and each line load is 530 pounds per foot.

For a temporary slope stability analysis, we used a factor of safety (FS) of 1.3 for the static condition.

Based on our visual observation, the side slopes of the existing trailhead embankment are approximately 2H:1V to 2.5H:1V. We assumed that the materials of the existing trailhead embankment are similar to the materials encountered in boring B-1 and B-2. Global stability analysis results and soil parameters are presented in Exhibit 8-16 and in Figure D13 in Appendix D.

Exhibit 8-16: Global Stability Analysis Results for Trailhead

Type of Analysis	Embankment Side Slopes	Required Factor of Safety	Factor of Safety
Static Condition	2.5H:1V	1.3	1.3

Based on our slope stability analysis, the side slopes of the existing trailhead embankment will need to be maintained at least 2.5H:1V or flatter. To place sliver fill on the side slopes of the existing embankment, pit run or 4-inch minus materials may be used.

8.11 Backfill Recommendations for Existing Culvert Removal

An existing 4' x 4' concrete box culvert is located on the trailhead embankment, to the east of the proposed Pedestrian Bridge. The top of the existing concrete box culvert is located at approximately 5.0 feet elevation. The existing trail head elevation in this location is approximately 15 feet, and therefore, the existing concrete box culvert has approximately 10 feet of fill on top of it. We understand that the existing concrete box culvert will be removed and backfilled to the existing geometry of the trailhead embankment.

The excavation should be backfilled with Satisfactory soil materials for levee-type fill are defined as those complying with ASTM D-2487 soil classification groups CL and ML (predominantly silt and lean clay), and should be verified by lab testing. Unsatisfactory soils include those complying with ASTM D-2487 soil classification groups GW, GP, and GM (predominantly gravel); SM, SW, and SP (predominantly sand); MH and CH (predominantly elastic silt and fat clay); OL and OH (predominantly organic); and Marsh Deposits. With additional testing, the use of MH, OH, and Marsh Deposits materials may

be allowed; however, additional laboratory testing and stability analysis would be required to use these materials.

Unless otherwise noted, fill should be compacted to 95 percent of the standard Proctor test (ASTM D698). The fill should be placed in maximum lifts of 8 inches of loose material. The fill materials should be compacted within the range of ±2 percent of the optimum moisture content value or with moisture content allowing compaction requirements to be obtained.

Conceptually, we anticipate that the coffer dam and dewatering will be below groundwater, and temporary shoring and dewatering will be required. Sheet pile is feasible to use to construct a coffer dam. We anticipate that dewatering could be accomplished using sump pumping with a sheet pile coffer dam. Design of temporary shoring and dewatering is not included in our scope of services.

9 GEOTECHNICAL CONSTRUCTION CONSIDERATIONS

9.1 Site Preparation and Earthwork

Site preparation will include (1) clearing, grubbing, and roadside cleanup; (2) removal of existing structures and underground utilities; and (3) subgrade preparation and excavation. These construction activities should generally be accomplished in accordance with the ODOT OSSC. If temporary shoring is needed, the design of such shoring is traditionally the responsibility of the contractor.

After site stripping and preparation activities are completed, the exposed subgrade to receive fill should be proof-rolled with a fully loaded 10- to 12-yard dump truck or similar heavy rubber-tired construction equipment to identify soft, loose, or unsuitable areas. The proof-roll should be conducted prior to fill placement.

The site stripping and proof-rolling should be observed by a qualified geotechnical engineer or representative, who should determine stripping depth, evaluate the suitability of the subgrade, and identify areas of yielding. If loose and/or wet, soft soil zones are identified during proof-rolling, the soils should be removed and replaced with compacted structural fill.

Disturbance of subgrade soil due to construction equipment and activities could affect the support of the proposed walls and embankment. The contractor should take necessary steps to protect the subgrade from becoming disturbed.

9.2 Temporary Cut-and-Fill Slopes

The culvert replacement will be excavated using the open-cut trench method. In general, temporary earth slopes may be cut near vertical for excavations less than 4 feet in depth. If the excavation is more than 4 feet in depth, flatter slopes will be required in accordance with the Occupational Safety and Health Administration (OSHA).

Temporary shoring is typically the responsibility of the contractor and should comply with applicable local, state, and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards. Temporary shoring techniques and dewatering techniques should be evaluated together, because one temporary shoring technique may work well with certain dewatering techniques and not well with others.

9.3 Water Control and Dewatering

To control water in the creeks, the water should be diverted around the excavation to downstream of the excavation using a pipe or channel. Groundwater should be expected to be encountered in the excavation for the culverts and bridge substructures. The design of the temporary dewatering systems is traditionally the responsibility of the Contractor. Any water collected during dewatering and any excavated soil should be treated and disposed of in a manner meeting local, state, and federal environmental regulations and requirements.

To control water in the No Name Creek, the water should be diverted around the excavation to downstream of the excavation using a pipe or channel. Groundwater should be expected to be encountered in the excavation for the culverts. We further recommend that the working mat 12-in-thick working mat provided in Section 8.8.2 be increased to 24-in-thick. Based on the subsurface conditions, we anticipate that dewatering could be accomplished using sump pumping with a crushed rock drainage layer by increasing the working mat in Section 8.8.2 from 12 inches thick to 24 inches thick.

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Table 1 - Recommended LRFLE Geotechnical Input Parameters for Pedestrian Bridge

Location	Material Boundaries	Approximate Bottom of Layer (depth)	Generalized Soil Class	Recommended P-Y Curve	Effective Unit Weight (pcf)	Friction Angle (deg)	p-y multiplier, k (pci)	Undrained Cohesion (psf)	ε_{50}
Pedestrian Bridge Bent 1	B-1	5	[redacted] FILL	Sand (Reese)	105	27	0	–	–
		12.5	[redacted] FILL	Sand (Reese)	42	27 (E)	0 (E)	–	–
		22	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	52	30 (M)	30 (E)	–	–
		35.5	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	56	33	60	–	–
		46	ALLUVIAL FAN DEPOSITS	Sand (Reese)	59	35	90	–	–
		66	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	56	33	60	–	–
		76	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	52	30 (M)	30 (M)	–	–
		86.5	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	56	33	75	–	–
Pedestrian Bridge Bent 2	B-2	6.5	[redacted] FILL	Sand (Reese)	105	27	10	–	–
		9.5	[redacted] FILL	Sand (Reese)	42	27 (E)	10 (E)	–	–
		20	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	52	30 (M)	10 (E)	–	–
		28	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	56	33	60	–	–
		35	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	53	30 (M)	20 (E)	–	–
		39	ALLUVIAL FAN DEPOSITS	Sand (Reese)	58	35	90	–	–
		42	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	53	30 (M)	20 (E)	–	–
		52	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	56	33	70	–	–
		58	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	53	30 (M)	40 (E)	–	–
		60.5	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	58	33	75	–	–

NOTES:
1. Depths are referenced to existing ground surface at start of survey. If ground surface at boring location is greater than top of first layer elevation, extrapolate top of first layer to top of foundation elevation.
2. Layers to be used should have their friction angle and p-y modifiers (k) chosen to be consistent w/ post-scour conditions.

deg = degrees, pcf = pounds per cubic foot, pci = pounds per cubic inch

Table 2 - Recommended LRFD Geotechnical Input Parameters for Reneke Creek Bridge

Location	Abutment Boundary	Approximate Bottom of Layer (depth)	Generalized Soil Unit	Recommended P-Y Curve	Effective Unit Weight (pcf)	Friction Angle (deg)	p-y multiplier, k (pci)	Undrained Cohesion (psf)	ε_{50}
Reneke Creek Bridge Bent 1	B-1	65	MARSH DEPOSITS	Soft Clay (Matlock)	100	–	–	500	0.02
		75	MARSH DEPOSITS	Soft Clay (Matlock)	38	–	–	500	0.02
		28	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	62	36 (D)	20 (D)	–	–
		45	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	56	38	100	–	–
		57	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	62	36 (D)	40 (D)	–	–
		64	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	56	38	60	–	–
		78	ALSEA FORMATION	Sand (Reese)	72	40	125	–	–
Reneke Creek Bridge Bent 2	B-4	65	MARSH DEPOSITS	Soft Clay (Matlock)	100	–	–	500	0.02
		80	MARSH DEPOSITS	Soft Clay (Matlock)	38	–	–	500	0.02
		12	FLUVIAL AND ESTUARINE DEPOSITS	Soft Clay (Matlock)	38	–	–	250	0.02
		28	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	62	36 (D)	5 (D)	–	–
		42	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	56	38	60	–	–
		59	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	62	36 (D)	40 (D)	–	–
		64	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	56	38	70	–	–
		78	ALSEA FORMATION	Sand (Reese)	72	40	125	–	–

NOTES:
1 Depths are referenced to existing ground surface at each bridge. If ground surface is greater than top of first layer elevation, extrapolate top of first layer to top of foundation elevation.
2 Layers in gray should have their Friction Angle and p-y modifiers (p) chosen to be consistent for post-seismic condition.
deg = degrees, pcf = pounds per cubic foot, psi = pounds per square inch

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Table 3 - Recommended LPILE Geotechnical Input Parameters for Beltz Creek Bridge

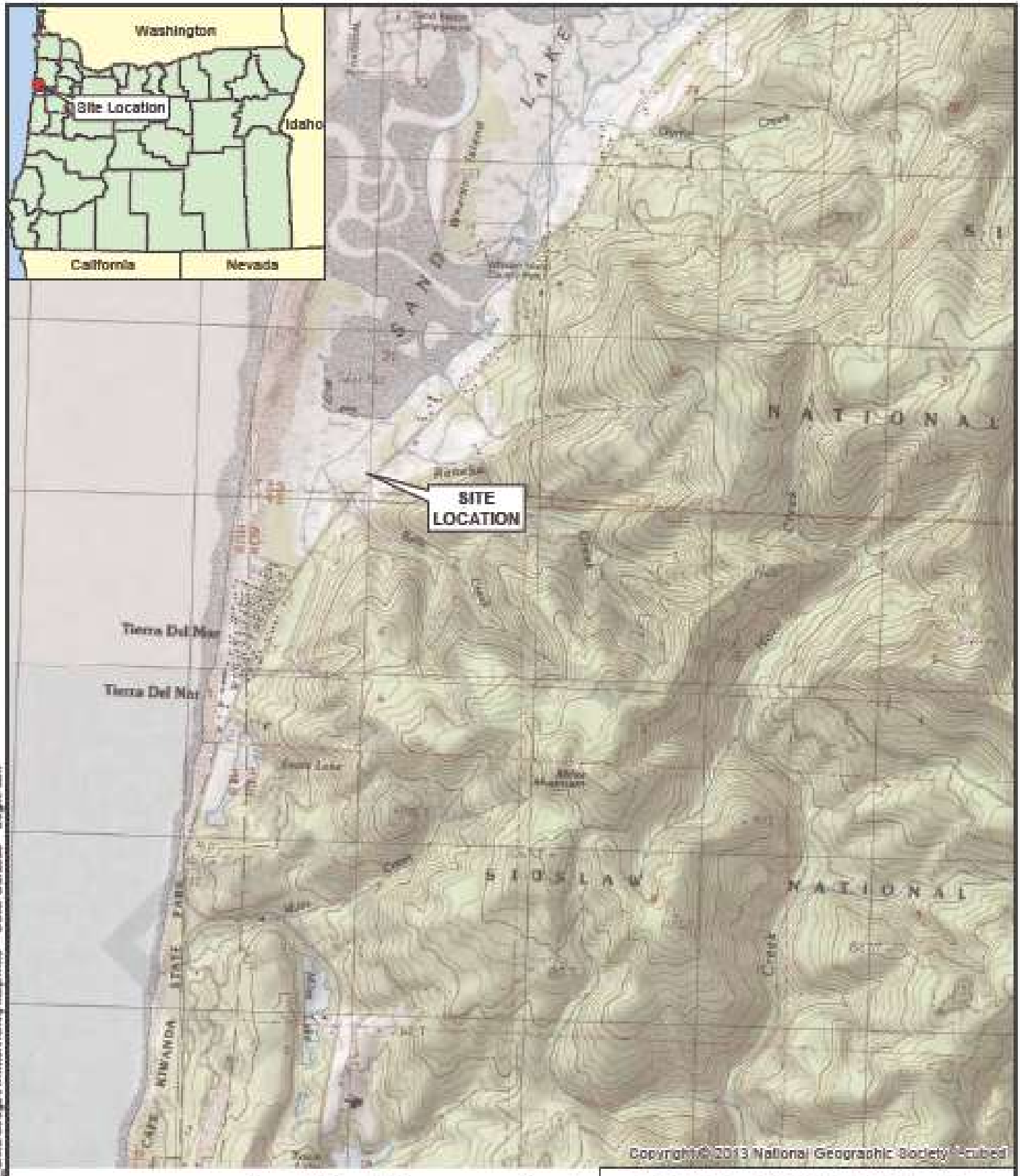
Location	Material Boundaries	Approximate Bottom of Layer (depth)	Generalized Soil Class	Recommended P-Y Curve	Effective Unit Weight (pcf)	Friction Angle (deg)	p-y modulus, k (pci)	Undrained Cohesion (psf)	ε_{50}
Beltz Creek Bridge Abutments	0-5	5	ROADWAY FILL	Soft Clay (Matlock)	100	–	–	500	0.02
		100	MARSH DEPOSITS	Soft Clay (Matlock)	100	–	–	250	0.02
		50	ALLUVIAL FAN DEPOSITS	Sand (Reese)	115	33	60	–	–
		54	ALLUVIAL FAN DEPOSITS	Sand (Reese)	120	35	125	–	–
		28	ALLUVIAL FAN DEPOSITS	Sand (Reese)	60	30	75	–	–
		325	ALLUVIAL FAN DEPOSITS	Sand (Reese)	60	30 (37)	45 (20)	–	–
		375	ALLUVIAL FAN DEPOSITS	Stiff Clay with Free Water (Reese)	60	–	–	1000	0.01
		42	ALLUVIAL FAN DEPOSITS	Sand (Reese)	60	31	90	–	–
		67	FLUVIAL AND ESTUARINE DEPOSITS	Sand (Reese)	60	30	110	–	–
		74	ALSEA FORMATION	Sand (Reese)	70	40	135	–	–

NOTES:

1. Depths are referenced to existing ground surface at each boring. If ground surface at Beltz location is greater than top of Bed Layer elevation, extrapolate top of Bed Layer to bay (Constitution) elevation.
2. Layers in bold shaded boxes liquefy. Angles are (typ assumed, (): shown in the boundaries for post-seismic condition.

deg = degrees, pcf = pounds per cubic foot, psi = pounds per square inch

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0 0.25 0.5 1

Scale in Miles

N

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

VICINITY MAP

September 2025 112880

SHANNON & WILSON FIG. 1

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Filename: P:\GIS\PDX\112650\d112650_Silva Sedge\Arcmap\Site Plan.mxd Date: 9/10/2025 Login: ash



LEGEND

B-1 Designation and Approximate Location of Boring

A A' Designation and Location of Interpretive Subsurface Profile

0 150 300 600

Scale in Feet

NOTES

1. Aerial imagery obtained through Google Maps Satellite.
2. Existing contours derived from 2009 LIDAR data obtained through DOGAMI.

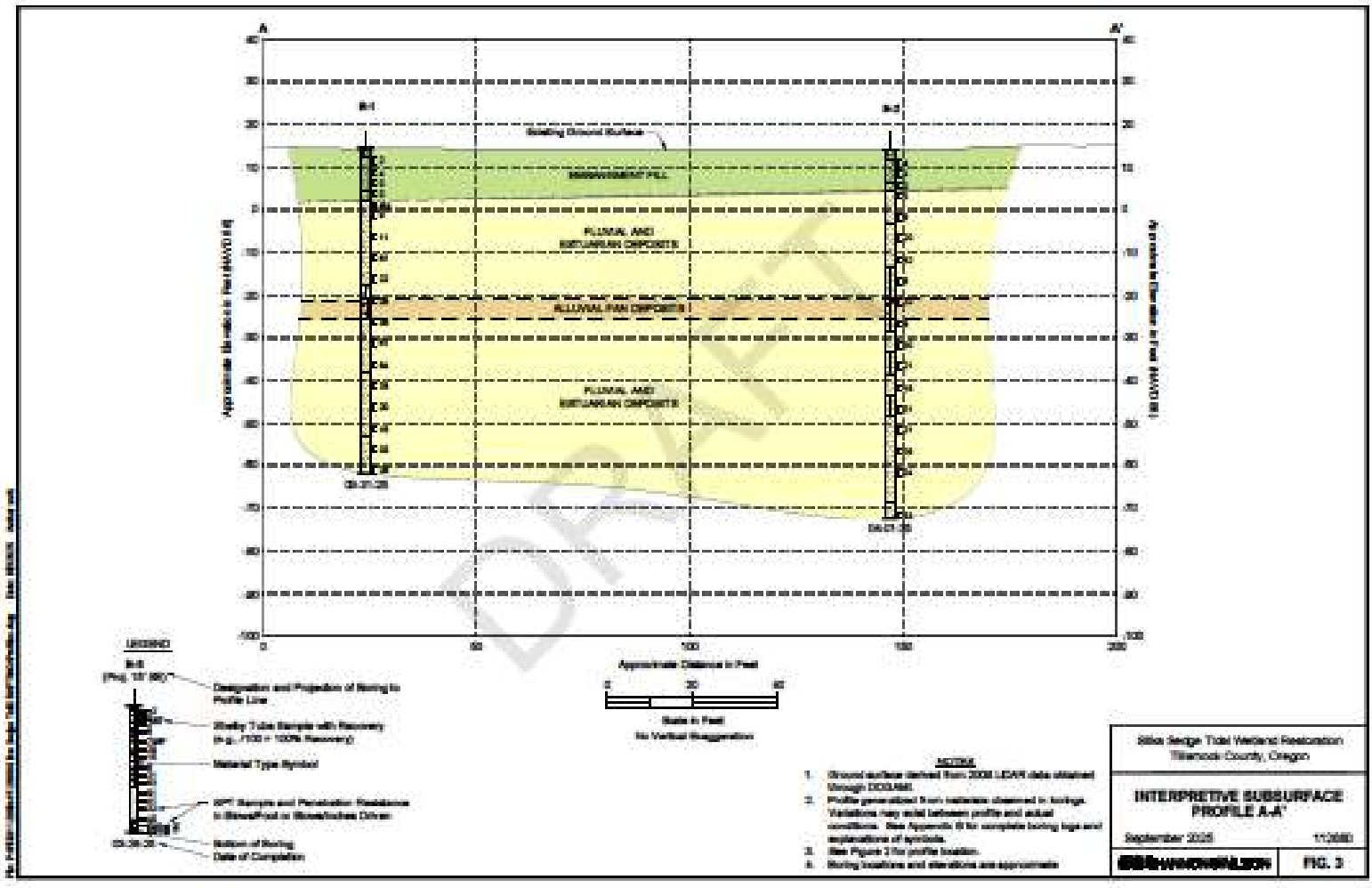
Silva Sedge Tidal Wetland Restoration
Tillamook County, Oregon

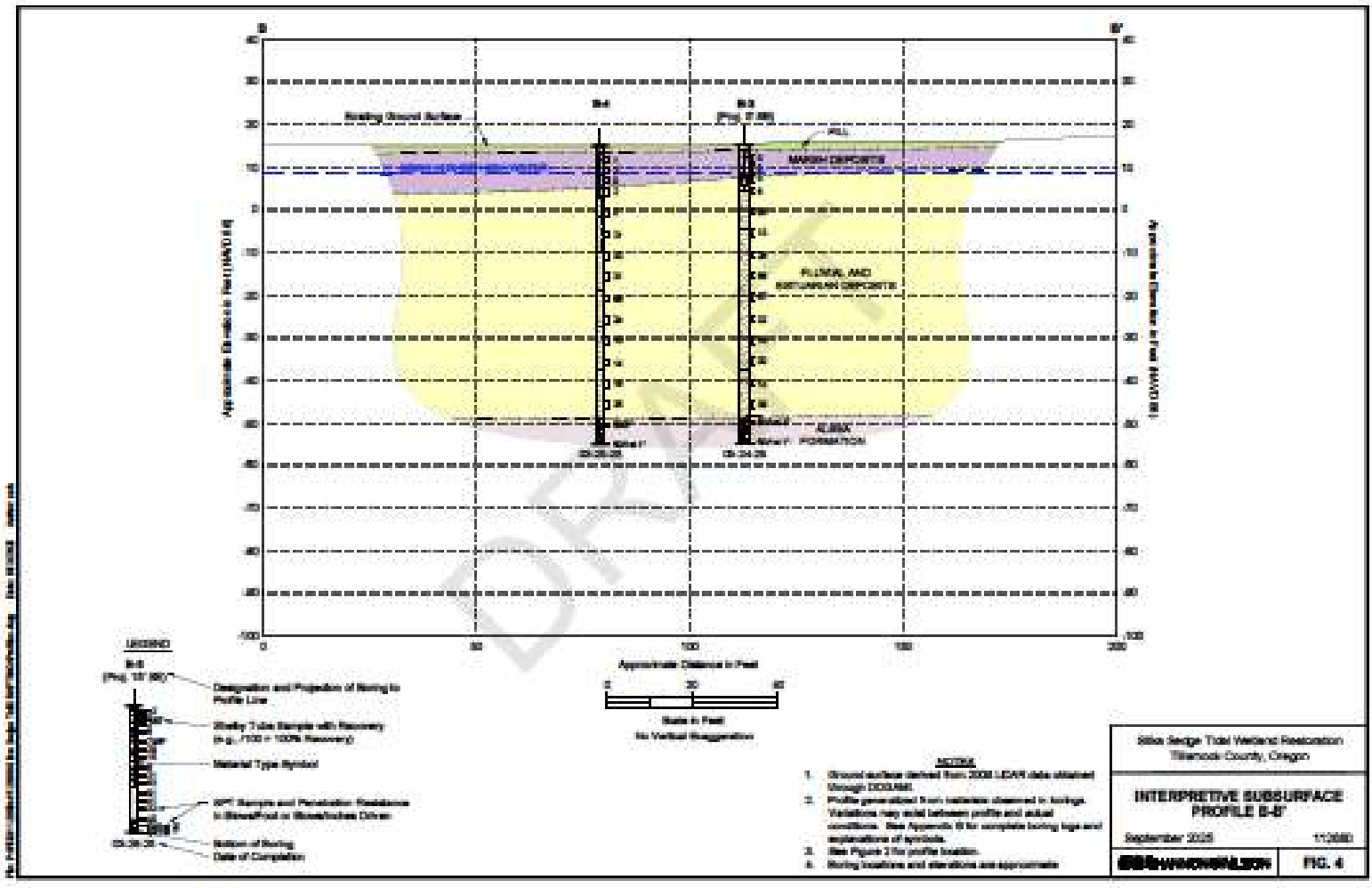
SITE AND EXPLORATION PLAN

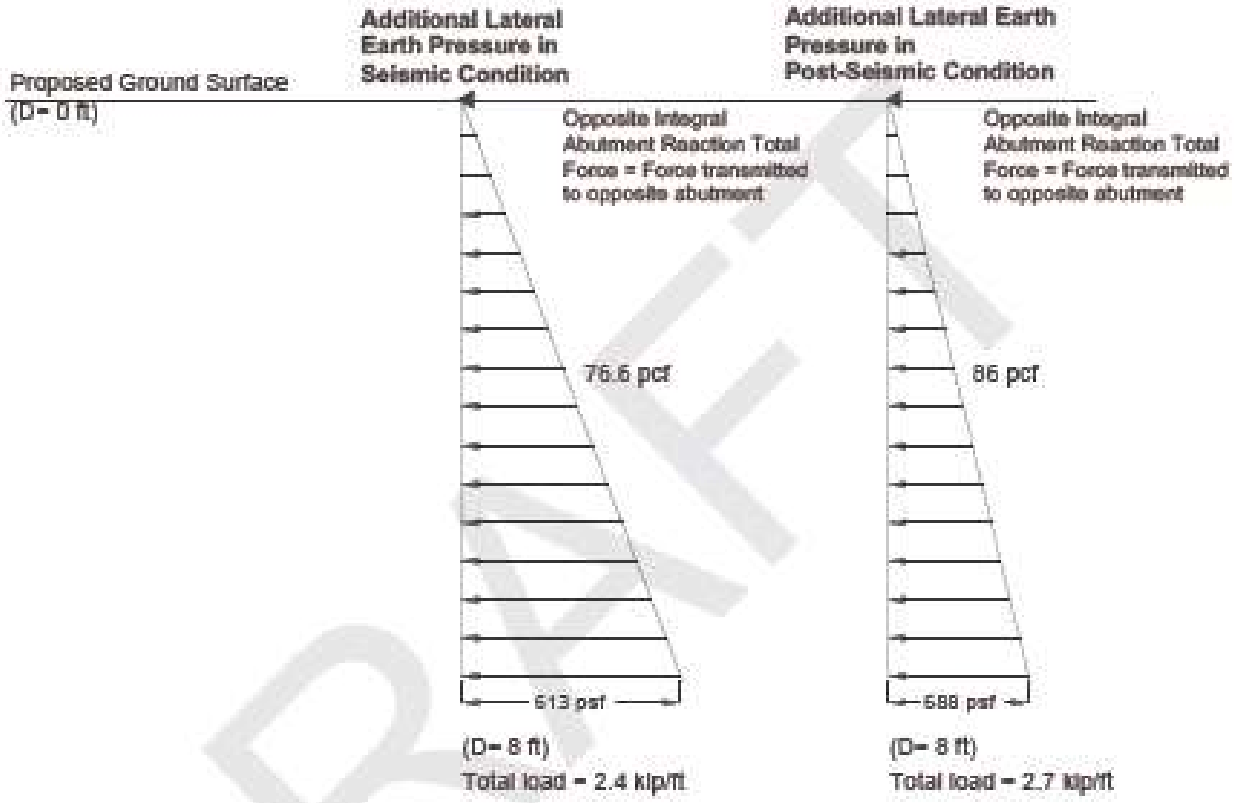
September 2025 112650

SHANNON & WILSON FIG. 2

FIG. 2


A
A'
B-1
B-2
Existing Ground Surface
EMBANKMENT FILL
FLUVIAL AND ESTUARINE DEPOSITS
ALLUVIAL FAN DEPOSITS
FLUVIAL AND ESTUARINE DEPOSITS
Approximate Elevation in Feet (NAVD 88)
Approximate Distance in Feet
0
50
100
150
200
LEGEND
B-6
(Proj. 15' SE)
Designation and Projection of Boring to Profile Line
Shelby Tube Sample with Recovery
(e.g., 7/30 = 100% Recovery)
Material Type Symbol
SPT Sample and Penetration Resistance
in Blows/Foot or Blows/Inches Drive
Bottom of Boring
Date of Completion
Scale in Feet
No Vertical Exaggeration
NOTES
1. Ground surface derived from 2008 LiDAR data obtained through DOGAMI.
2. Profile generalized from materials observed in borings. Variations may exist between profile and actual conditions. See Appendix B for complete boring logs and explanations of symbols.
3. See Figure 2 for profile location.
4. Boring locations and elevations are approximate.
Siltcoos Sedge Tidal Wetland Restoration
Tillamook County, Oregon
INTERPRETIVE SUBSURFACE PROFILE A-A'
September 2025
112680
FIG. 3
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B
B'
Existing Ground Surface
B-4
B-3
[Proj. 7.0ft]
FILL
MARSH DEPOSITS
FLUVIAL AND
ESTUARINE DEPOSITS
ELEVATION (ft NAVD 88)
Approximate Distance in Feet
0
50
100
150
200
LEGEND
Designation and Projection of Boring to Profile Line
Shelby Tube Sample with Recovery
(e.g., 7/30 = 100% Recovery)
Material Type Symbol
SPT Sample and Penetration Resistance
in Blows/Foot or Blows/Inches Drive
Bottom of Boring
Date of Completion
NOTES
1. Ground surface derived from 2008 LiDAR data obtained through DOGAMI.
2. Profile generalized from materials observed in borings. Variations may exist between profile and actual conditions. See Appendix B for complete boring logs and explanations of symbols.
3. See Figure 2 for profile location.
4. Boring locations and elevations are approximate
Silos Sedge Tidal Wetland Restoration
Tillamook County, Oregon
INTERPRETIVE SUBSURFACE PROFILE B-B'
September 2025
112680
FIG. 4



Notes:
1. Equivalent fluid pressures should be applied to the abutment.
2. Resolve lateral earth pressures into forces acting on the abutment wall by multiplying the resultant earth pressure force on the abutment wall by the abutment width.
3. Total reaction force should be distributed evenly to each pile.
4. Loading should be applied perpendicular to the pile.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

ESTIMATED ADDITIONAL LATERAL EARTH PRESSURES ON PEDESTRIAN BRIDGE ABUTMENT

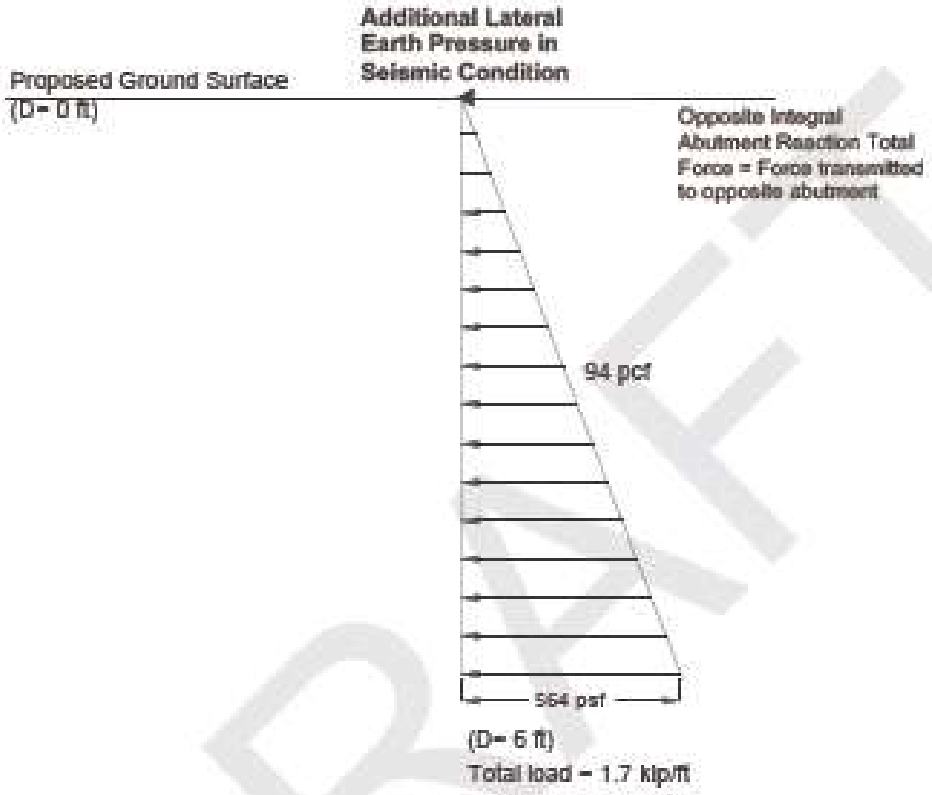
October 2025 112880

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FIG.5

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File: I:\WP\Projects\24-1 Portland\3800\3842 OR38 & US101 Culvert Replacement Project\Graphics\CAD\Additional EP.dwg Date: 03/02/2016 Author: epp



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Notes:
1. Equivalent fluid pressures should be applied to the abutment.
2. Resolve lateral earth pressures into forces acting on the abutment wall by multiplying the resultant earth pressure force on the abutment wall by the abutment width.
3. Total reaction force should be distributed evenly to each pile.
4. Loading should be applied perpendicular to the pile.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

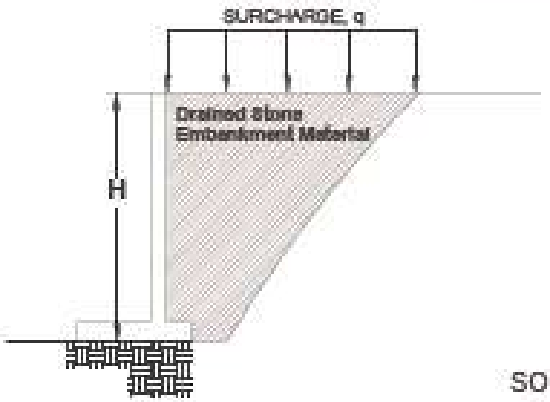
ESTIMATED ADDITIONAL LATERAL EARTH PRESSURES ON BELTZ BRIDGE ABUTMENT

October 2025 112880

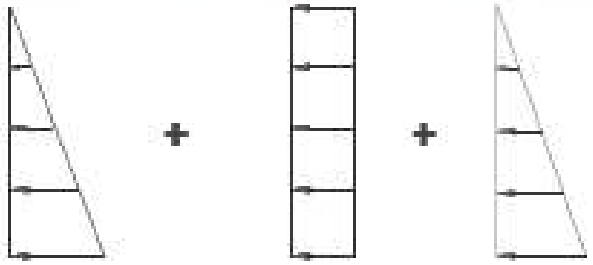
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FIG.6

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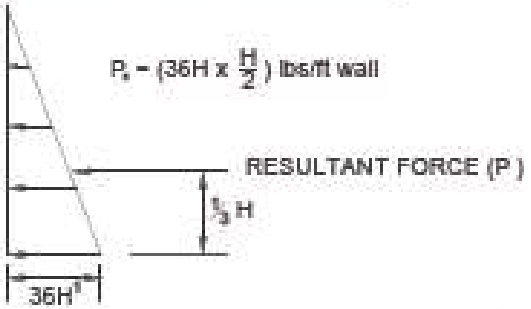


TOTAL LATERAL EQUIVALENT FLUID PRESSURES

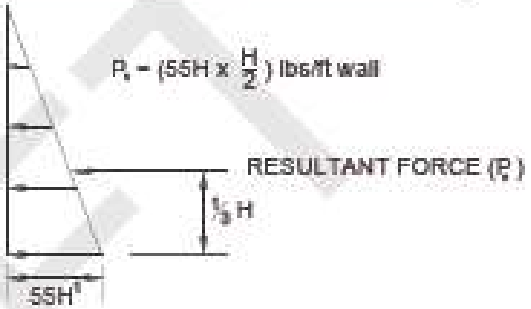


SOIL BACKFILL COMPONENT + SURCHARGE COMPONENT + SEISMIC BACKFILL COMPONENT

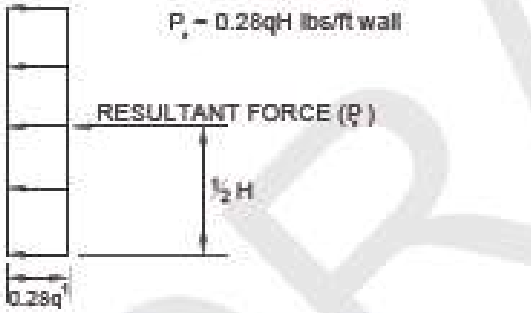
YIELDING WALL SOIL COMPONENT



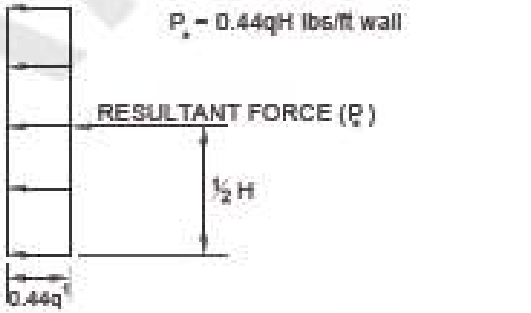
NON-YIELDING WALL SOIL COMPONENT



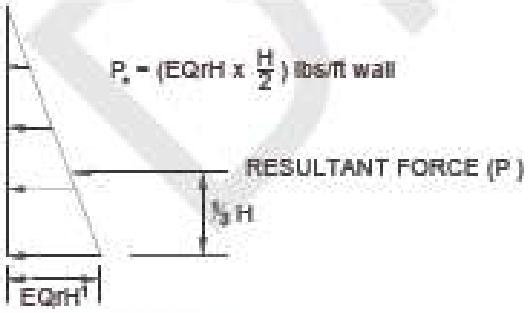
YIELDING WALL SURCHARGE COMPONENT



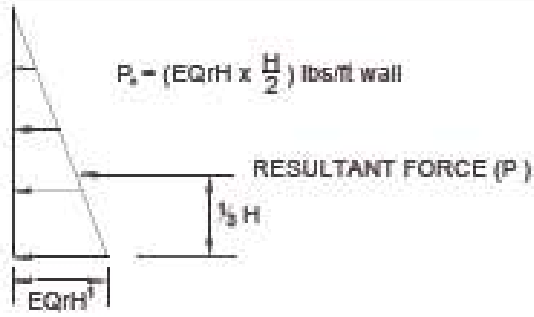
NON-YIELDING WALL SURCHARGE COMPONENT



YIELDING SEISMIC BACKFILL COMPONENT



NON-YIELDING SEISMIC BACKFILL COMPONENT



NOTES

1. Units are pounds per square foot (psf).
2. Backfill unit weight of 125 pcf.
3. Backfill friction angle is 34 deg.
4. Retained wall backfill is assumed to be drained stone embankment material.
5. Seismic pressures provided for peak ground accelerations associated with AASHTO "Risk-Targeted" earthquake level. See Table 1 for values.

TABLE 1

YIELDING EQr (pcf)	NON YIELDING EQr (pcf)
37	90

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

RECOMMENDED LATERAL PRESSURES FOR ABUTMENT AND WING WALLS

June 2025 112880

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG.7

DRAFT

File: P:\PDX\104000s\104743 Tigard - North Dakota St Bridge\Analysis\Deep Foundations\LEP\LEP Diagrams.dwg Date: 03-30-2023 Author: DEP

APPENDIX A: DESIGN DRAWINGS

Appendix A

Design Drawings

Contents

- Sitka Sedge Tidal Wetland Restoration, Draft 60% Design
- Beltz Creek Stream restoration, 30% Construction Documentation
- No Name Creek Culvert Replacement, 30% Construction Documentation

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TILLAMOOK ESTUARIES PARTNERSHIP

SITKA SEDGE TIDAL WETLAND RESTORATION

60% DESIGN - PRELIMINARY - NOT FOR CONSTRUCTION

PROJECT LOCATION: TIERRA DEL MAR, OREGON

SHEET LIST

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G2.1	EXISTING CONDITIONS AND SURVEY CONTROL
G2.2	SITE PLAN AND OVERVIEW
G2.3	ACCESS, STAGING, PERMIT, AND PROTECTION
G2.4	BRIDGE CREEK EAST NO. 1 CONSTRUCTION & SURVEY CONTROL
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C1.2	CMP BRIDGE CHANNEL PROFILE
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C1.4	CHANNEL ADJUST #1 REGRADE PLAN AND SECTION - 2
C2.1	CHANNEL GRADING AND TYPICAL CROSS SECTIONS
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	BRIDGE CREEK BRIDGE SHEETS
S0.1	BRIDGE BRIDGE PLAN AND ELEVATION
S0.2	BRIDGE BRIDGE SECTION AND TYPICAL DETAILS
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S0.10	PEDESTRIAN BRIDGE GENERAL NOTES
S0.11	PEDESTRIAN BRIDGE FOUNDATION PLAN
S0.12	PEDESTRIAN BRIDGE RAIL DETAILS
S0.13	PEDESTRIAN BRIDGE ABUTMENT DETAILS
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S2.1	TIDE GATE STRUCTURE PLAN AND PROFILE
S2.2	TIDEGATE DETAIL
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S2.3.3	BRIDGE CONTROL CUT SLAB - 1
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L2.0	REVEGETATION SURVEY OVERVIEW
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L2.2	REVEGETATION AREA 2
L2.3	REVEGETATION AREA 3
L3.0	PLANTING SCHEDULE
L3.1	PLANTING SCHEDULE
L3.2	PLANTING SCHEDULE
L4.0	TYPICAL PLANTING PROFILES
L5.0	PLANTING DETAILS
L5.1	PLANTING DETAILS



VICINITY MAP

PROJECT TEAM

DRAFT

Dig|Safely.

SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

COVER

G1.0

GENERAL CONSTRUCTION

1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]
5. [illegible]
6. [illegible]
7. [illegible]
8. [illegible]
9. [illegible]
10. [illegible]

SITE WORK

1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]
5. [illegible]
6. [illegible]
7. [illegible]
8. [illegible]
9. [illegible]
10. [illegible]
11. [illegible]

TIMING FOR IN-WATER WORK

1. [illegible]

ABBREVIATIONS

[illegible]

NAVD88 NORTH AMERICAN VERTICAL DATUM OF 1988

[illegible]

DRAFT

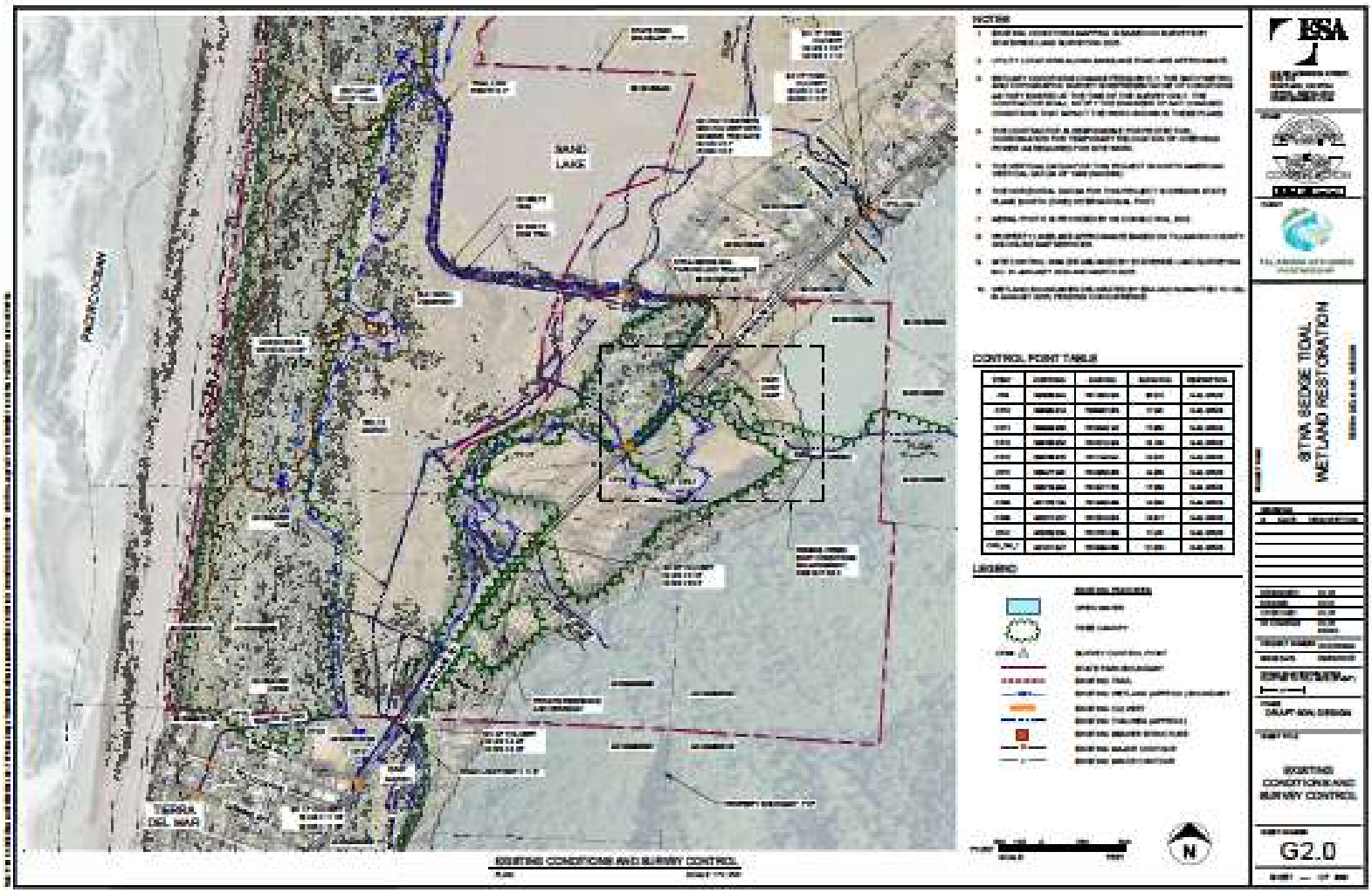
ESA

SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

GENERAL NOTES, LEGENDS, AND ABBREVIATIONS

G1.1



NOTES

CONTROL POINT TABLE

LEGEND

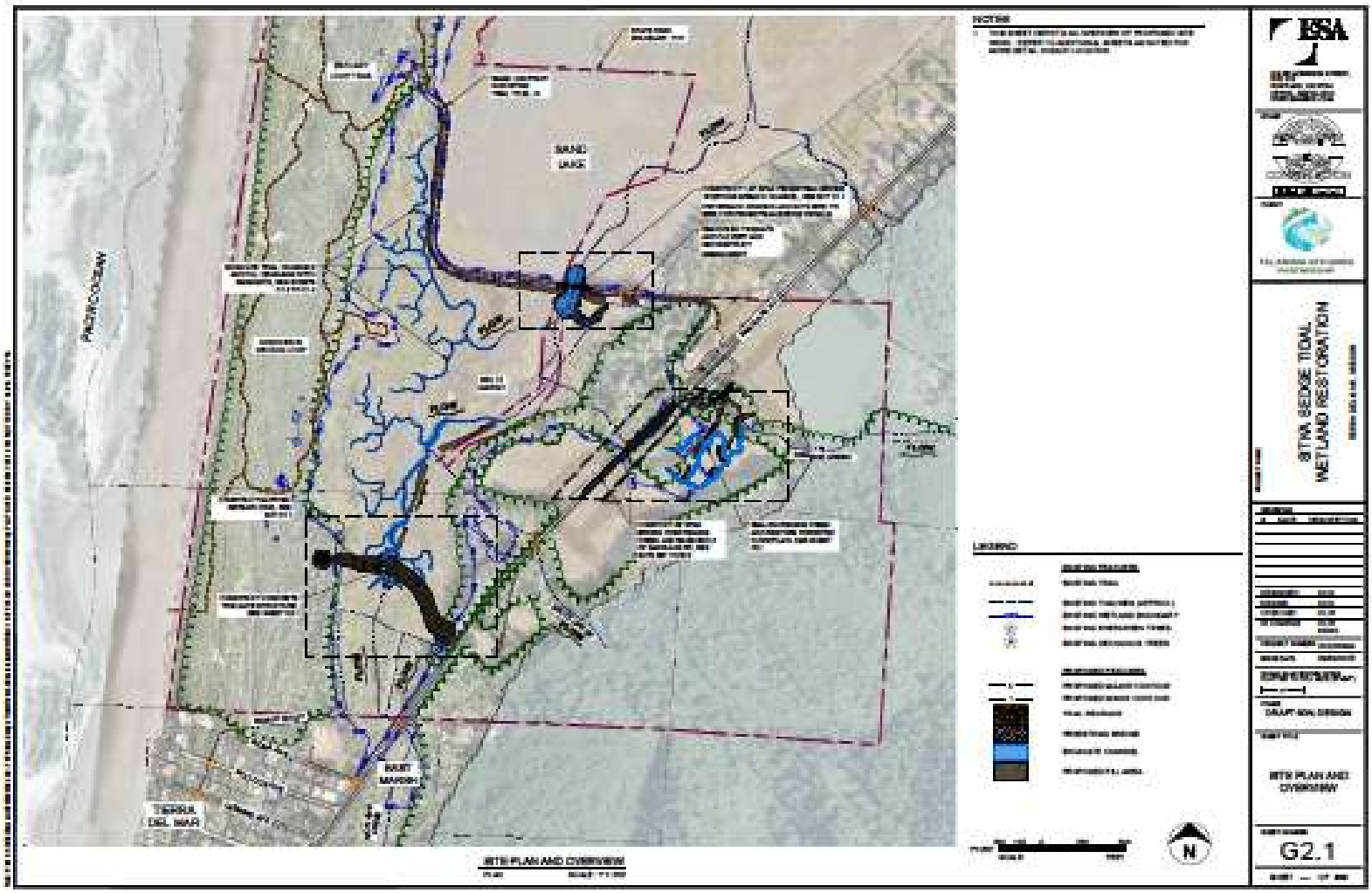
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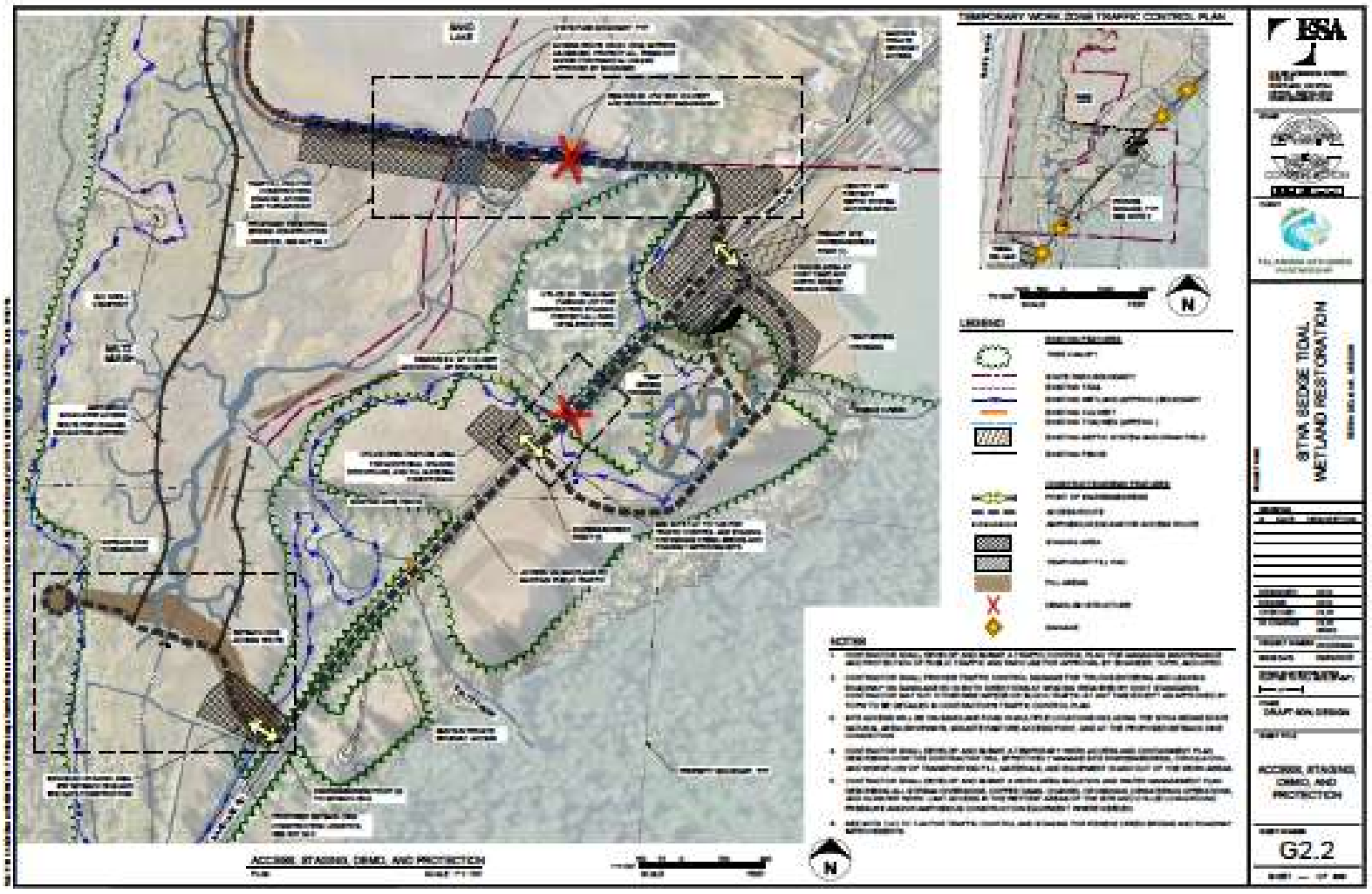
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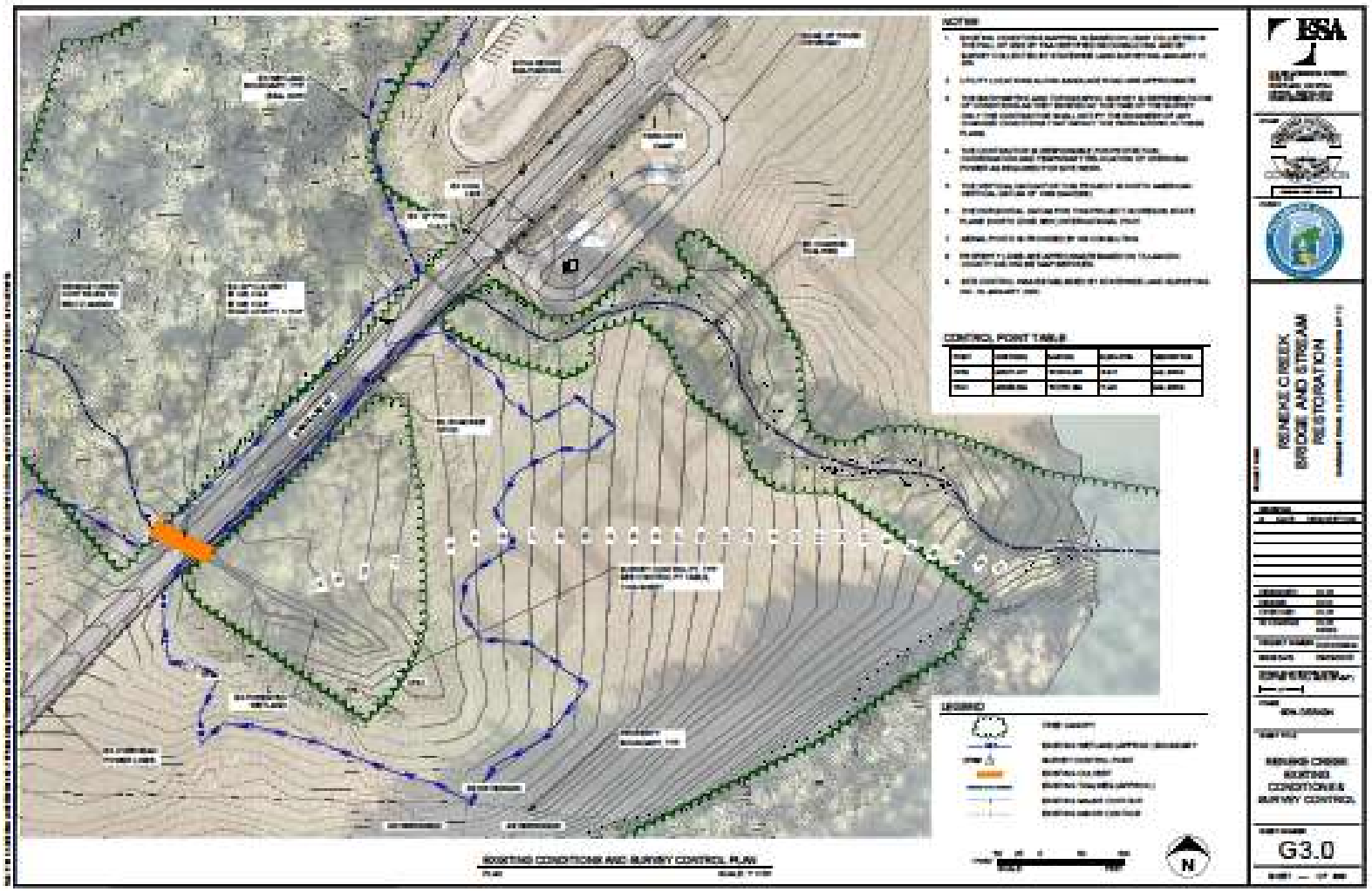
DRAFT 90% DESIGN

EXISTING CONDITIONS AND SURVEY CONTROL

G2.0


NOTES
LEGEND
BTS PLAN AND OVERVIEW
ESSA
BTS PLAN AND OVERVIEW
G2.1


TEMPORARY WORK ZONE TRAFFIC CONTROL PLAN
LEGEND
NOTES
ACCESS, STAGING, DEMO, AND PROTECTION
PLAN
ESSA
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT-90% DESIGN
ACCESS, STAGING, DEMO, AND PROTECTION
G2.2


NOTES
CONTROL POINT TABLE
LEGEND
EXISTING CONDITIONS AND SURVEY CONTROL PLAN
ESSA
REMEKE CREEK
BRIDGE AND STREAM
RESTORATION
REMEKE CREEK
EXISTING
CONDITIONS &
SURVEY CONTROL
G3.0



CONSTRUCTION, BROUNCHING AND LOGISTICS - PEDESTRIAN BRIDGE

PLAN

SCALE: 1"=50'

NOTES

1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]
5. [illegible]
6. [illegible]
7. [illegible]
8. [illegible]

LEGEND

[illegible]

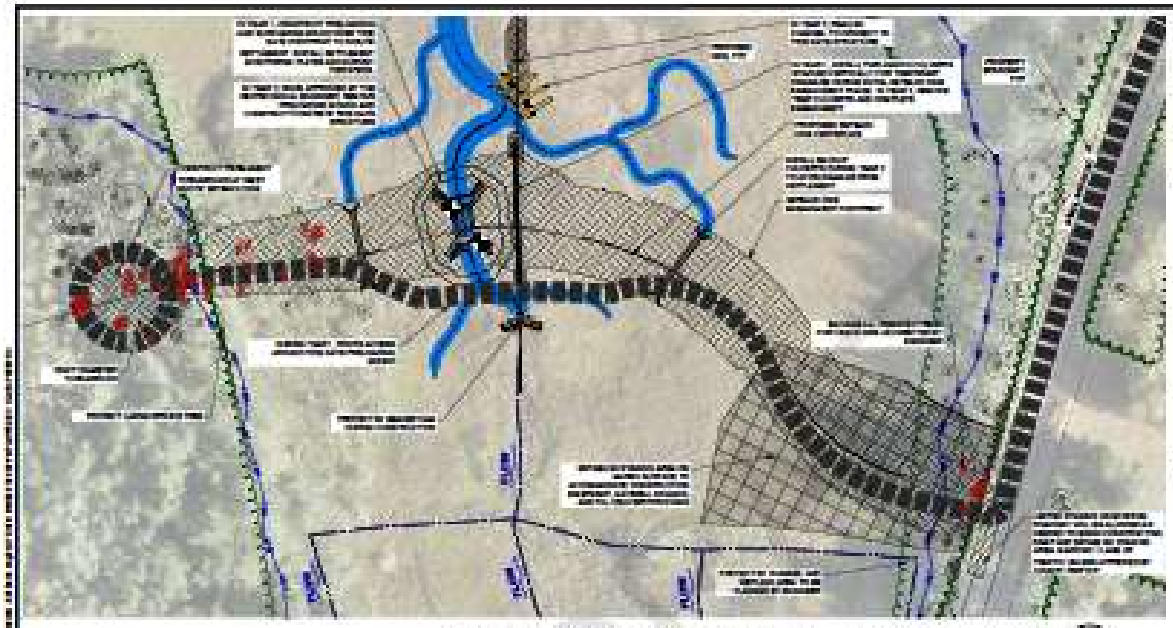
ESSA

STRA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

CONSTRUCTION LOGISTICS - PEDESTRIAN BRIDGE

G4.1



CONSTRUCTION SEQUENCING AND LOGISTICS - SETBACK DIKE

NOTES

LEGEND

SETTLEMENT MONITORING PLATE

SETTLEMENT MONITORING LEGEND

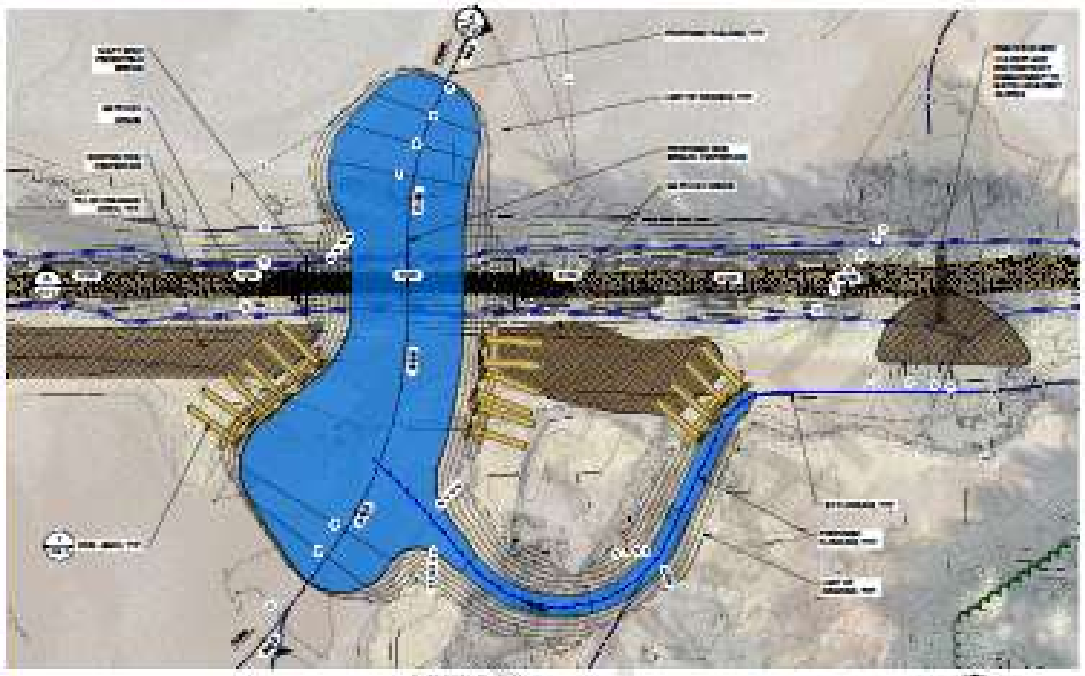
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STRA SEDGE TIDAL WETLAND RESTORATION

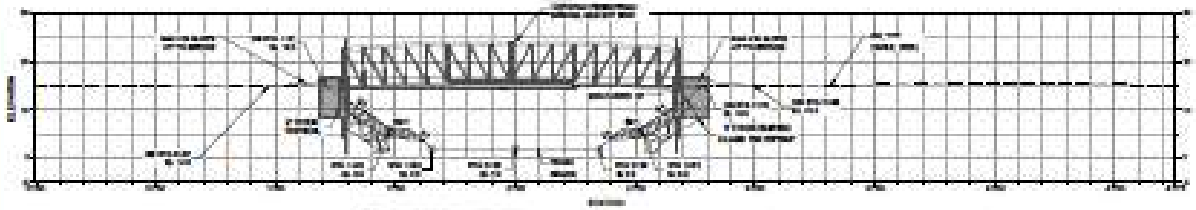
DRAFT-90% DESIGN

CONSTRUCTION LOGISTICS - SETBACK DIKE

G4.2



DIKE BREACH PLAN



DIKE BREACH SECTION

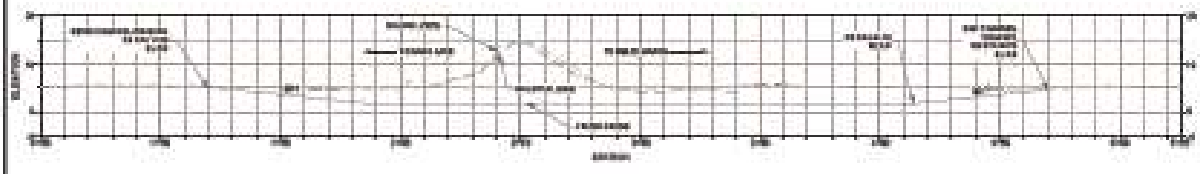
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STRA SEDGE TIDAL WETLAND RESTORATION

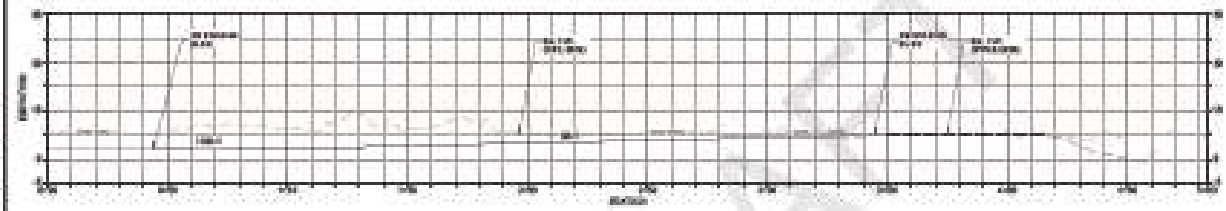
DRAFT 60% DESIGN

DIKE BREACH PLAN AND SECTION

C1.1



A DIKE BREACH CHANNEL PROFILE



B SIDE CHANNEL PROFILE

NOTES

1. [illegible]

DRAFT

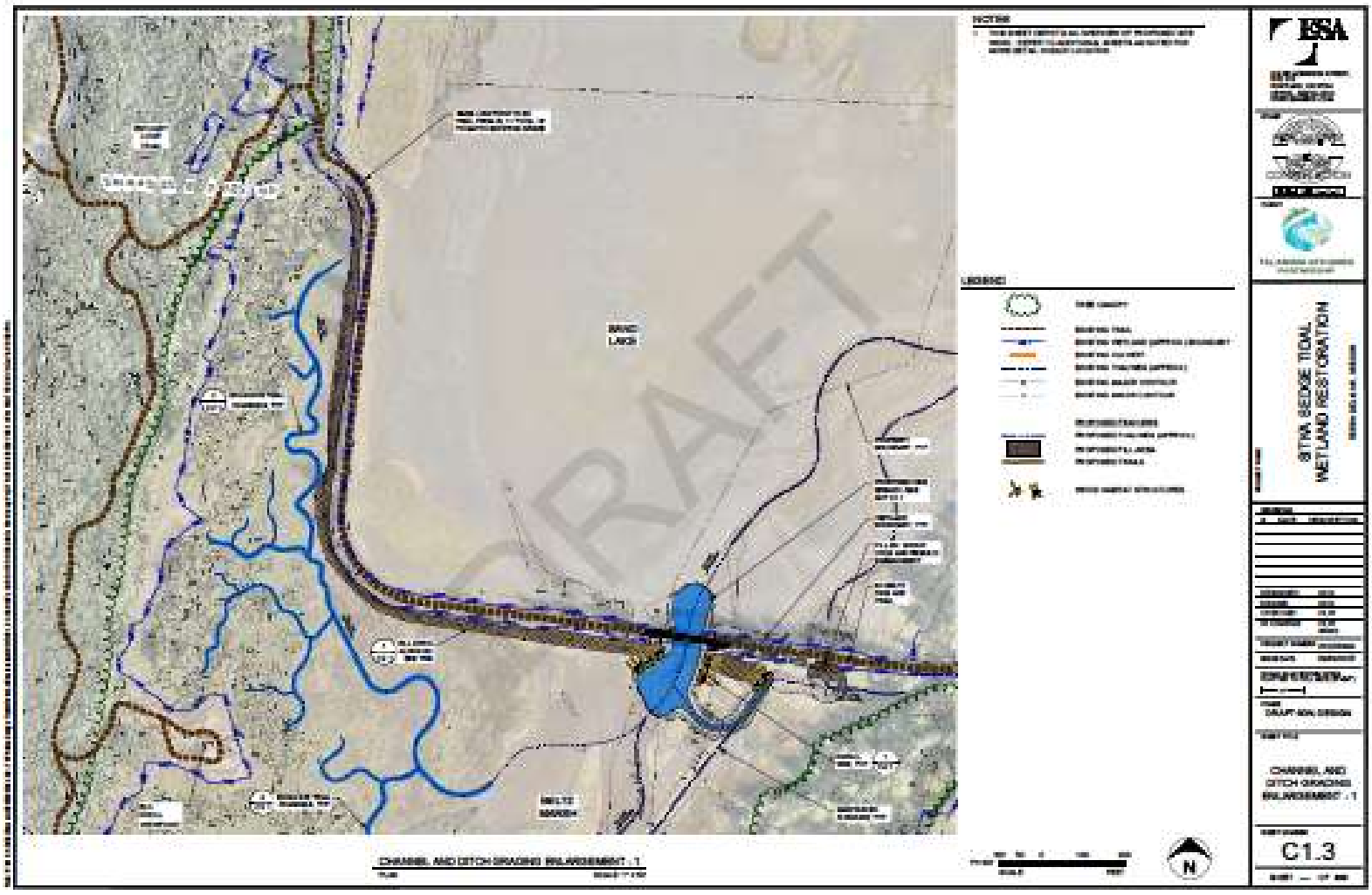
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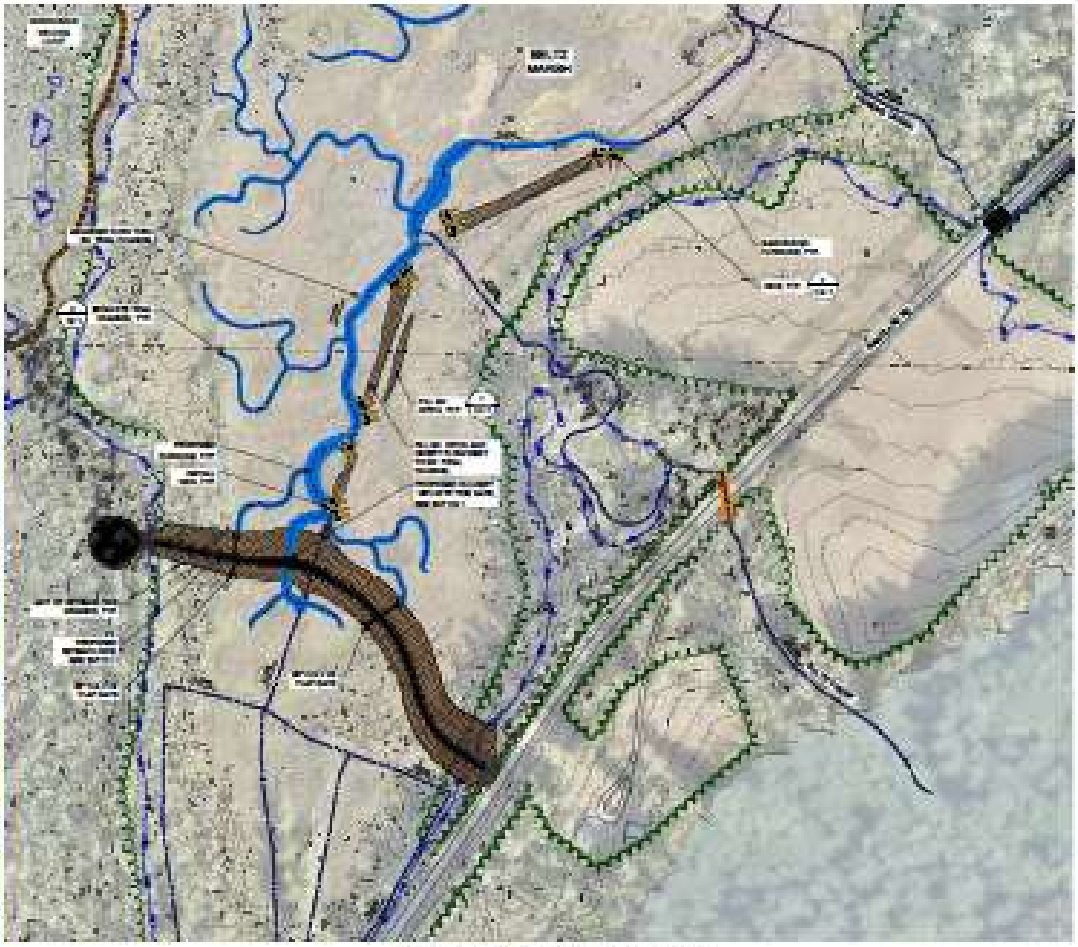
SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

DIKE BREACH CHANNEL PROFILE

C1.2


NOTES
LEGEND
BRAD LAKES
MARSH
ESSA
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT NOT FOR CONSTRUCTION
CHANNEL AND DITCH GRADING ENLARGEMENT - 1
C1.3
CHANNEL AND DITCH GRADING ENLARGEMENT - 1
N



CHANNEL AND DITCH GRADING INLARGEMENT - 2

PLAN

NOTES

1. [illegible]

LEGEND

KEY MAP

ESSA

STRA SEDGE TIDAL WETLAND RESTORATION

DRAFT NON-DESIGN

CHANNEL AND DITCH GRADING ENLARGEMENT - 2

C1.4

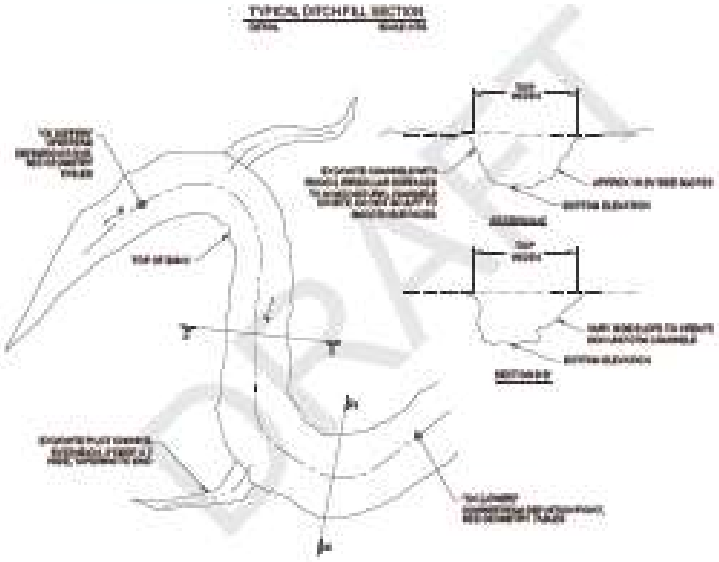
NOTES

1.

2.



TYPICAL DITCHFILL SECTION



TYPICAL NEW CHANNEL PLAN AND CROSS SECTION

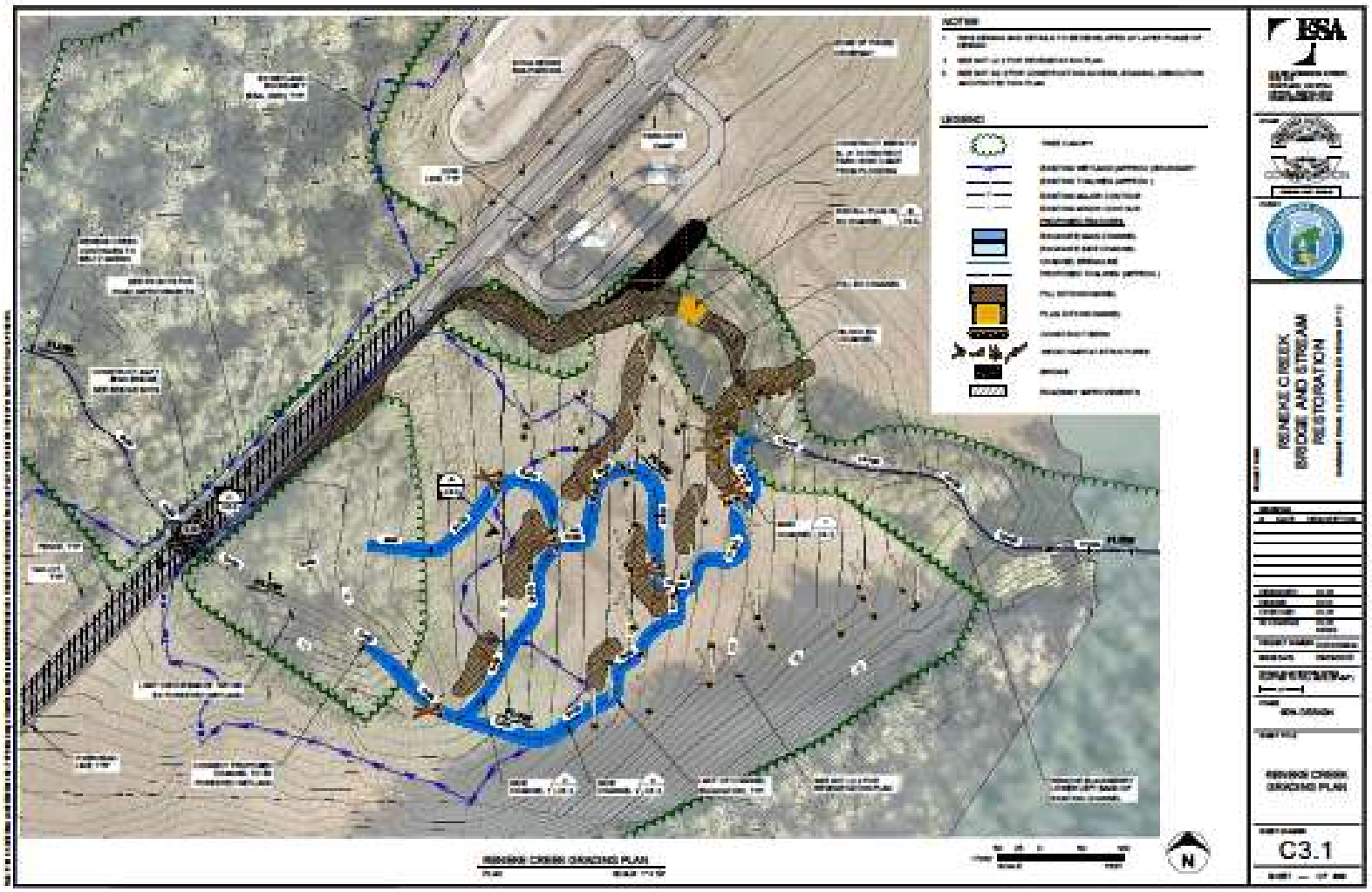
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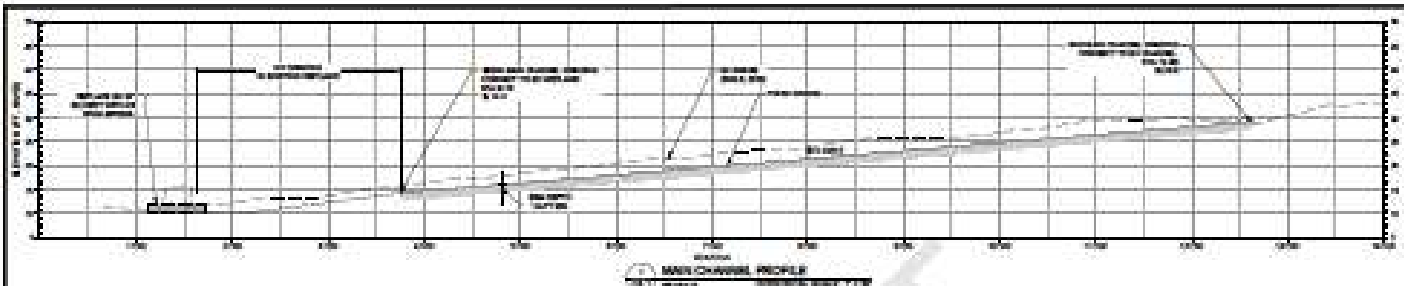
SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT-50% DESIGN

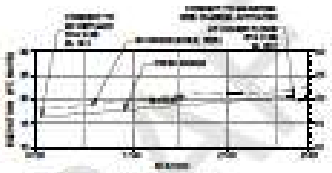
CHANNEL GRADING SECTIONS AND DETAILS

C2.1

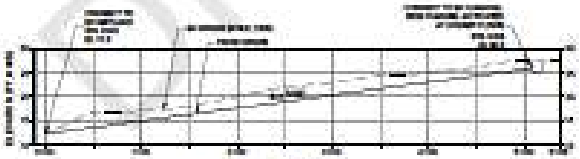

NOTES
LEGEND
RENWICK CREEK
BRIDGE AND STREAM
RESTORATION
RENWICK CREEK GRADING PLAN
C3.1
N



1 MAIN CHANNEL PROFILE



2 SIDE CHANNEL 1 PROFILE



3 SIDE CHANNEL 2 PROFILE

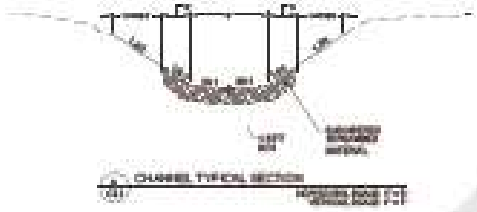
DRAFT

ESSA

PENNYCUIK CREEK
BRIDGE AND STREAM
RESTORATION

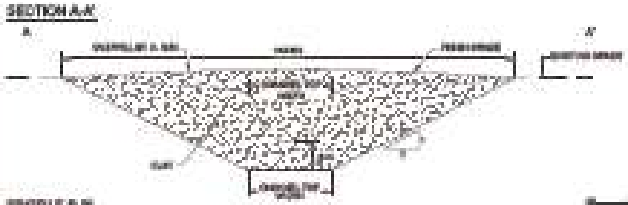
RIPARIAN CROSS-
CHANNEL PROFILES

C3.2

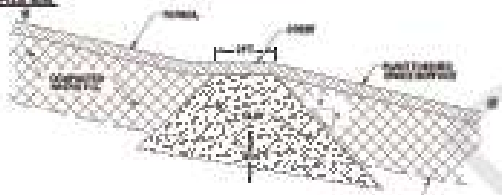


CHANNEL TYPICAL SECTION

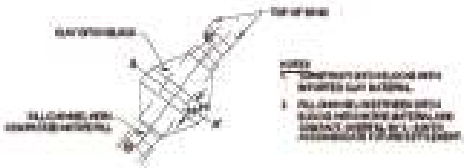
SECTION A-A'



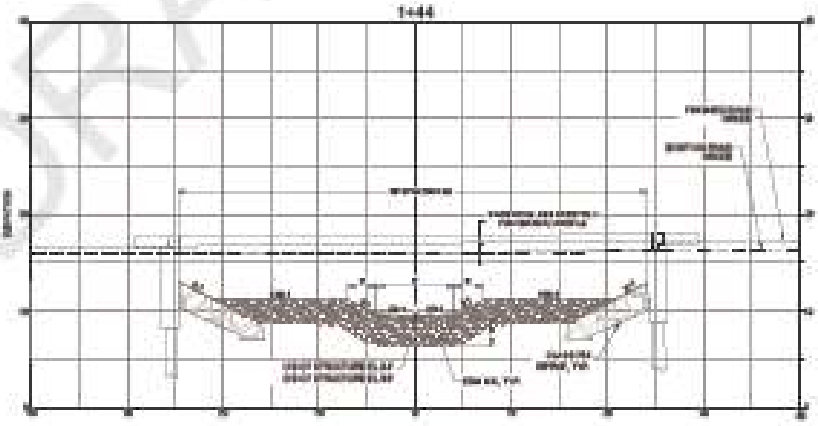
SECTION B-B'



PLAN



DITCH BLOCK



PROPOSED 40-FT SPAN BRIDGE

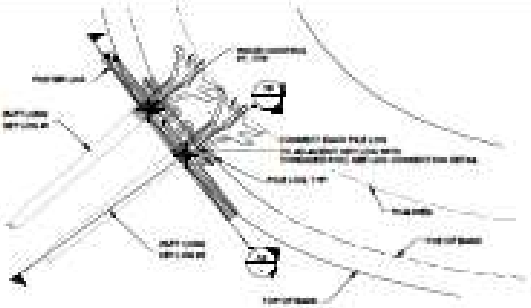
DRAFT

ESSA

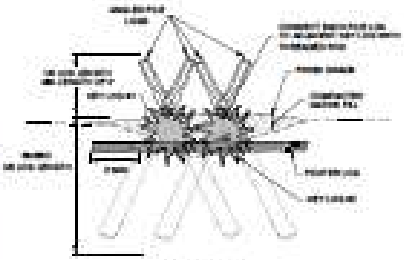
RENDIVE CREEK BRIDGE AND STREAM RESTORATION

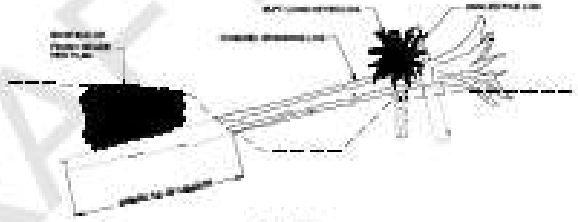
RENDIVE CREEK GRADING SECTIONS

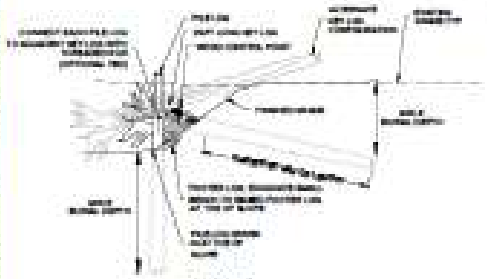
C3.3













DRAFT

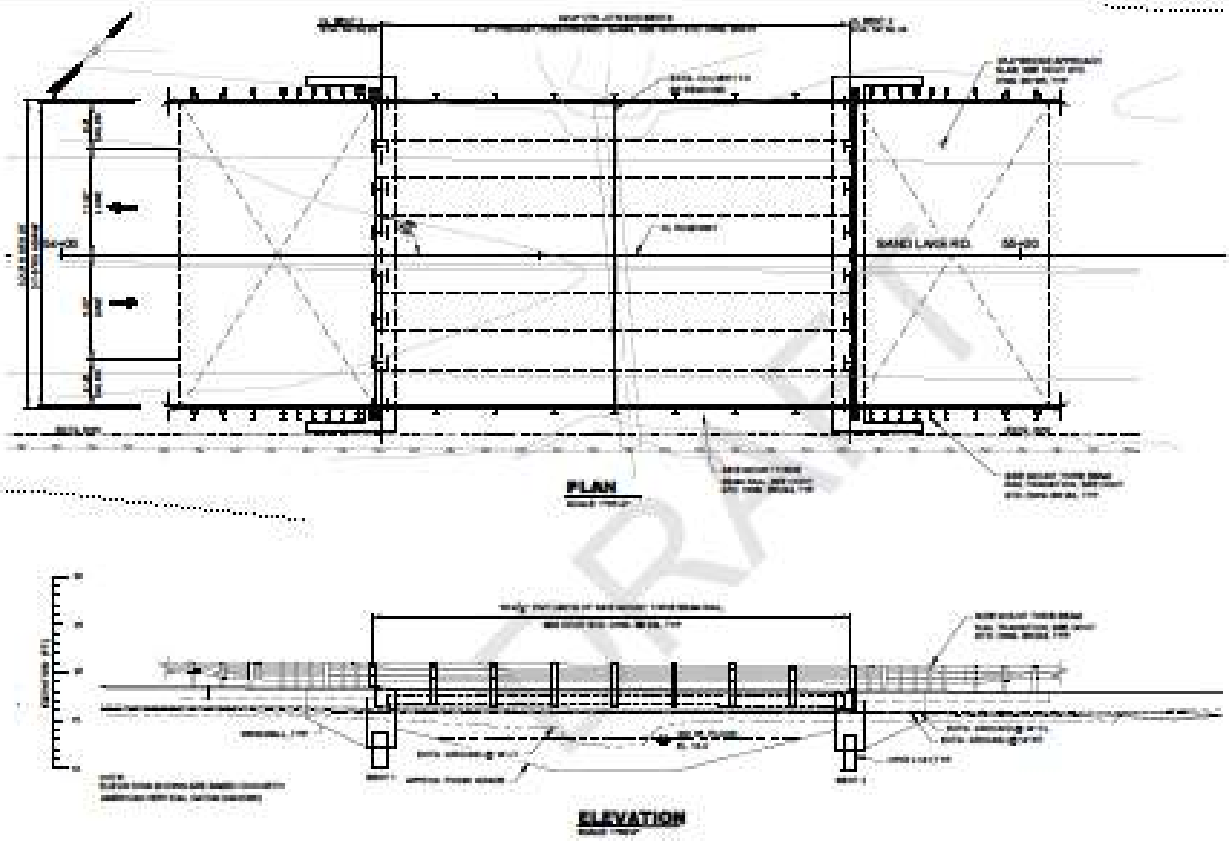
ESSA

STRA SEDGE TIDAL WETLAND RESTORATION

DRAFT-90% DESIGN

HABITAT DETAILS

C4.1



PRELIMINARY
NOT FOR CONSTRUCTION

RENEVE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

AS-CONSTRUCTION DOCUMENTATION

PLAN AND ELEVATION

B01

GENERAL NOTES

TYPE OR-STP-080Y

TYPE OR-STP-080Y

TYPE OR-STP-080

PERMIT TRUCK DIAGRAMS

TYPICAL SECTION

EXCAVATION/BACKFILL DIAGRAM

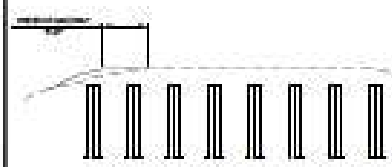
PRELIMINARY
NOT FOR CONSTRUCTION

RENEKE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

NON-CONSTRUCTION DOCUMENTATION

GENERAL NOTES AND TYPICAL SECTION

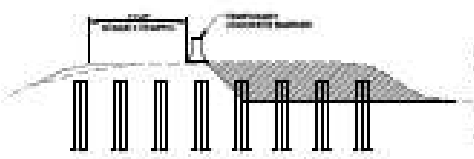
B02



SINGLE LANE CLOSURE UNDER FLAGGING

STAGE I

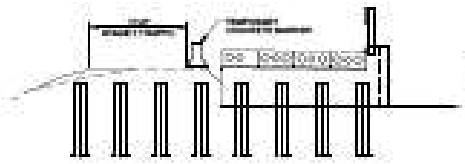
STAGE I NOTES



WEST BOUND LANE CLOSURE

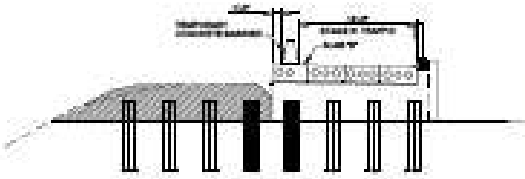
STAGE II

STAGE II NOTES



STAGE III

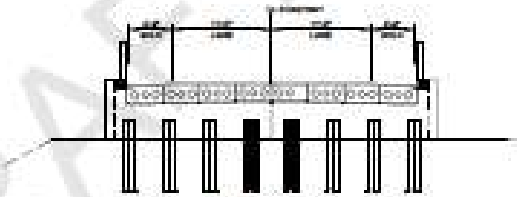
STAGE III NOTES



SOUTH BOND LANE CLOSURE

STAGE IV

STAGE IV NOTES



STAGE V

STAGE V NOTES

DRAFT

PRELIMINARY

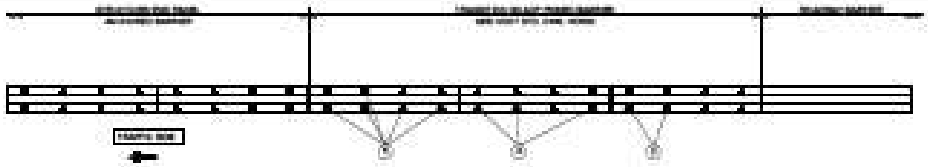
NOT FOR CONSTRUCTION

RENEKE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

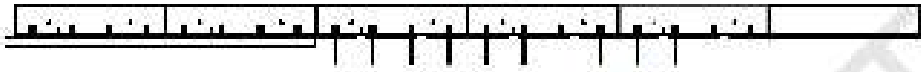
AS CONSTRUCTED DOCUMENTATION

CONSTRUCTION STAGING

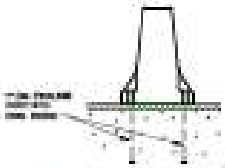
B03



RAIL PLAN



RAIL ELEVATION



PINNED BARRIER SECTION

GENERAL NOTES

DRAFT

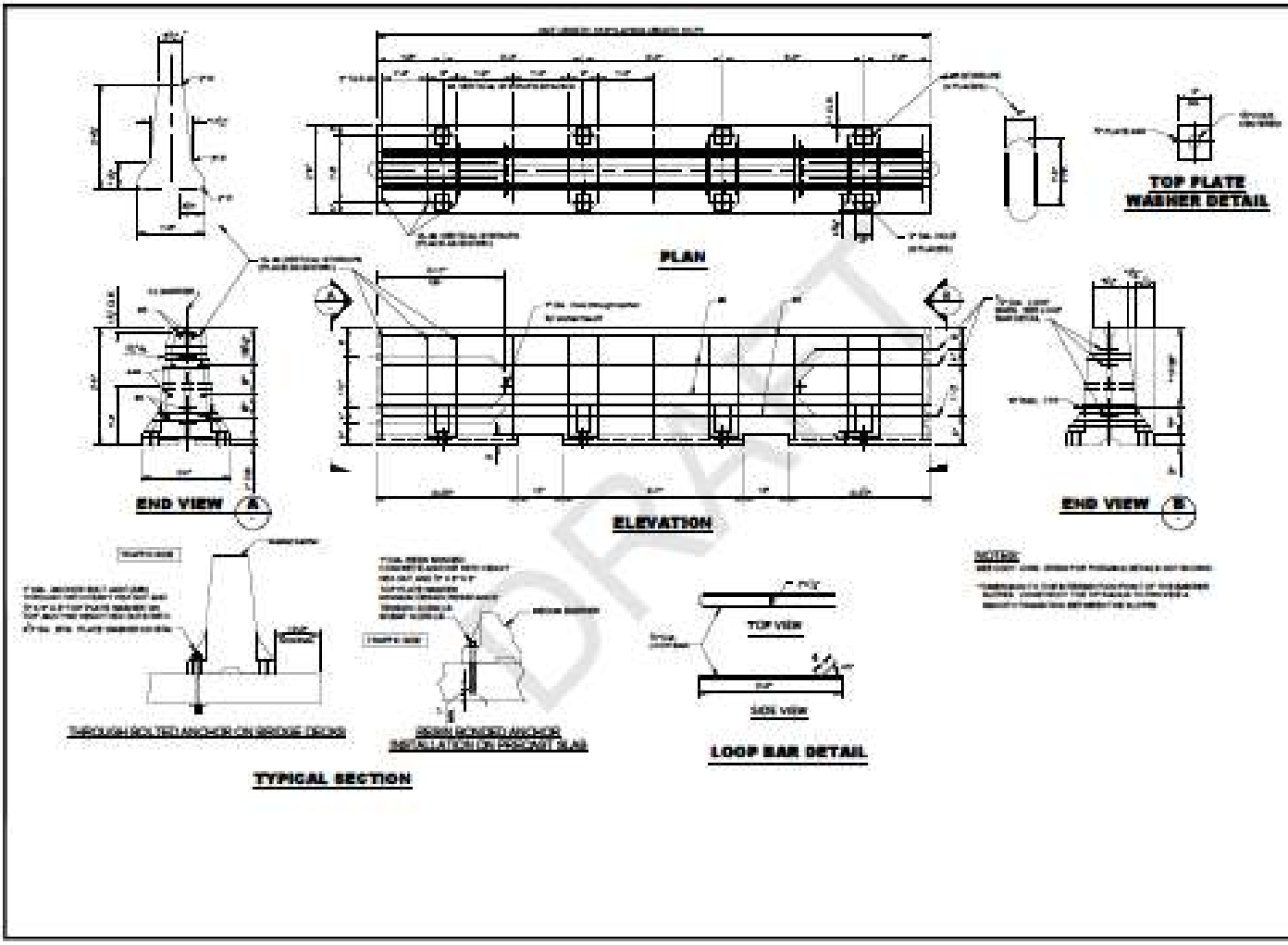
PRELIMINARY
NOT FOR CONSTRUCTION

RENEVE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

FOR CONSTRUCTION DOCUMENTATION

CONSTRUCTION STAGING DETAILS

B04



DRAFT

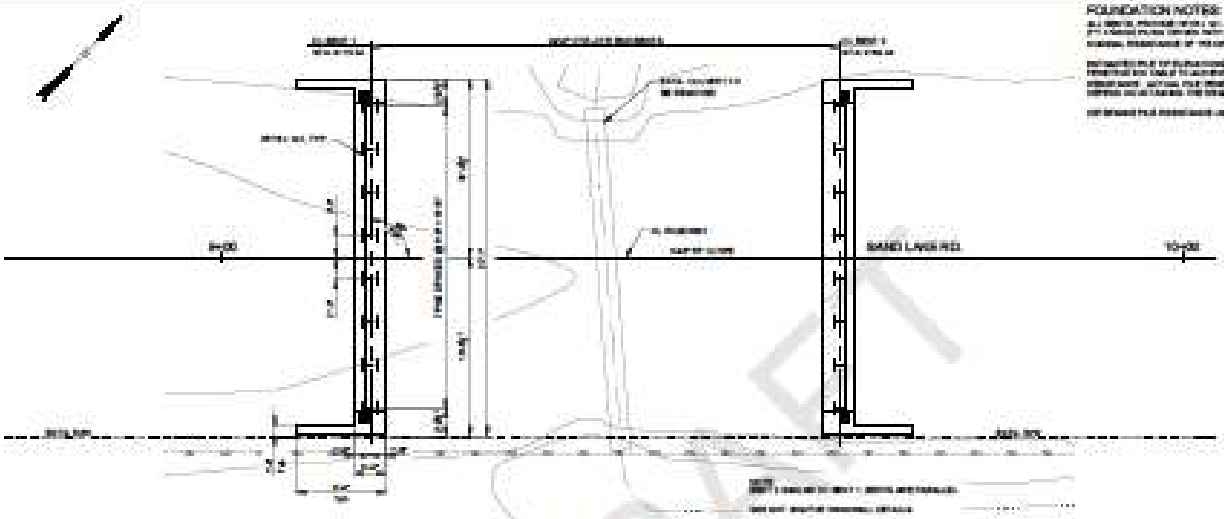
PRELIMINARY
NOT FOR CONSTRUCTION

RENEHE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

NON-CONSTRUCTION DOCUMENTATION

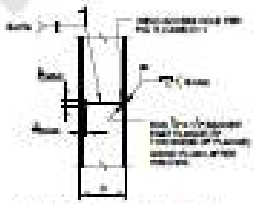
CONSTRUCTION STAGING DETAILS

B05



FOUNDATION PLAN

FOUNDATION NOTES:



H-PILE SPLICE DETAIL

PILE PENETRATION TABLE

DRAFT

PRELIMINARY
NOT FOR CONSTRUCTION

RENEVE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

90% CONSTRUCTION DOCUMENTATION

FOUNDATION PLAN

B06

21" STANDARD PRECAST PRESTRESSED SLABS

A	1	1	44-#5	11	5	55	3	3.01	4.09	9.5	4.1	19.4	3/4"	1/2"	-	3"	3"	3/4"
B	1	1	44-#5	11	5	55	3	3.01	4.09	9.5	4.1	19.4	3/4"	1/2"	-	3"	3"	3/4"

ACP BUILD-UP DETAIL

SLAB SHEAR KEY DETAIL

NOTES:

1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]
5. [illegible]

TYPICAL 21" SLAB "A" SECTION

TYPICAL 21" SLAB "B" SECTION

CONCRETE PAD DETAIL

BEARING DETAIL

PLAN

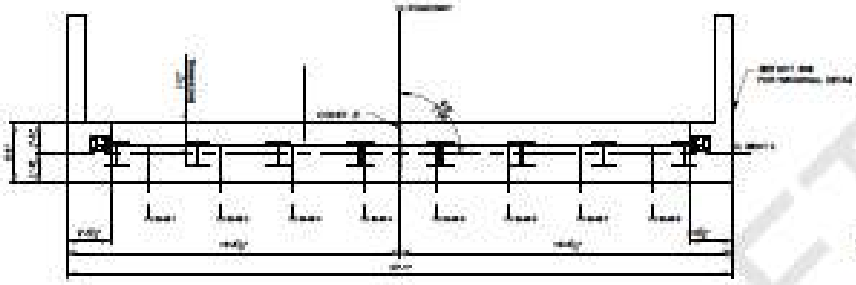
RENEVE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

PRELIMINARY
NOT FOR CONSTRUCTION

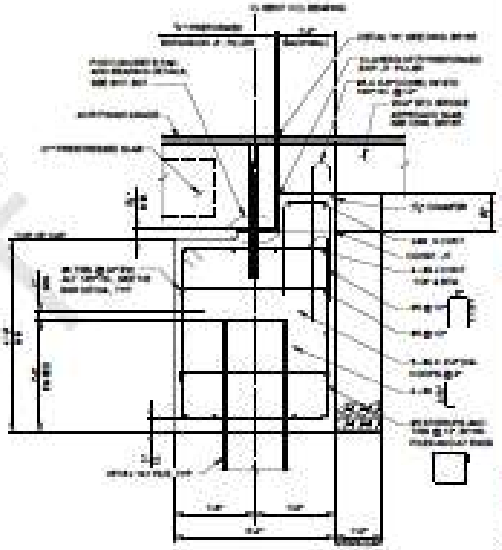
NON-CONSTRUCTION DOCUMENTATION

PRECAST SLAB DETAILS

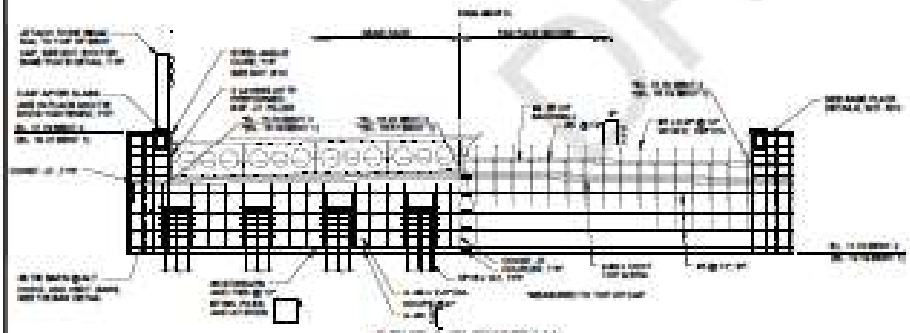
B07



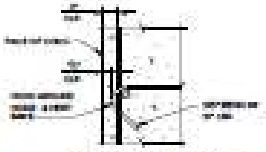
BENT 2 PLAN



TYPICAL BENT SECTION



BENT 2 ELEVATION



TIE BAR DETAIL

DRAFT

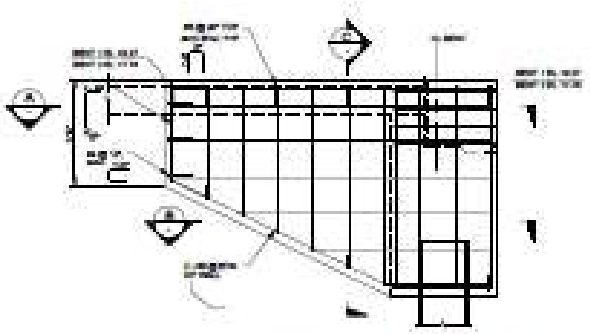
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RENEVE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

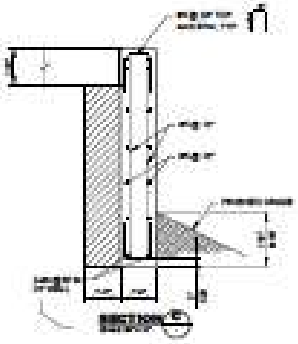
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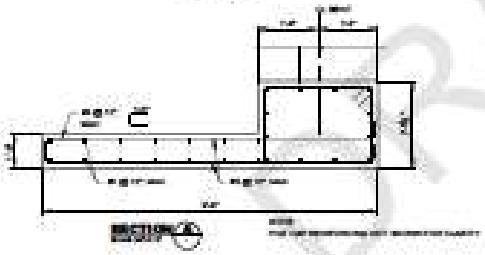
BENT DETAILS

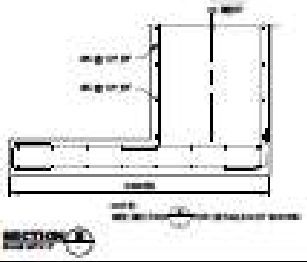
B08



TYPICAL WINGWALL ELEVATION







DRAFT

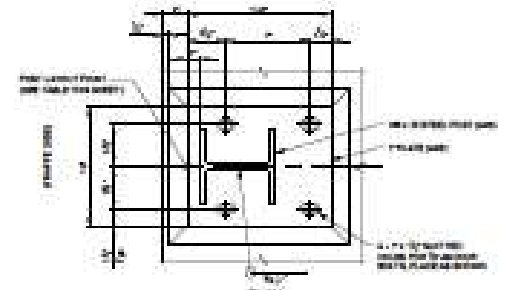
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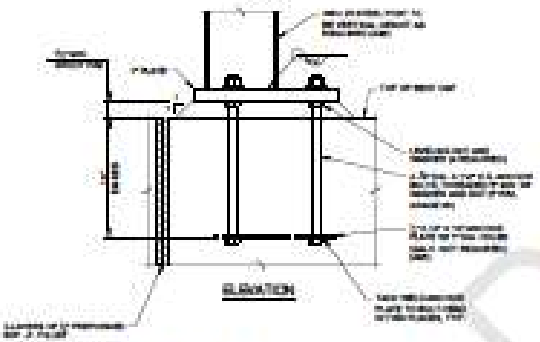
RENEVE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

FOR CONSTRUCTION DOCUMENTATION

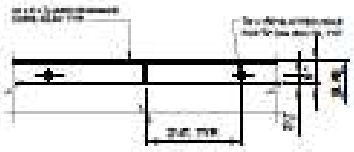
WINGWALL DETAILS

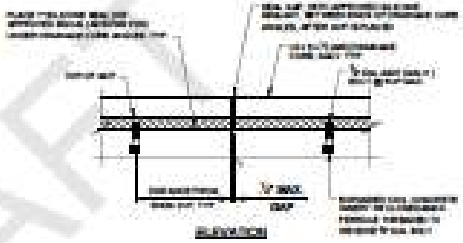
B09

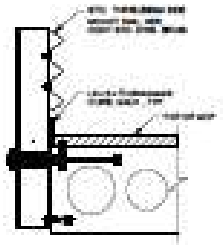




RAIL POST BASE PLATE DETAILS







DRAINAGE CURB DETAIL

DRAFT

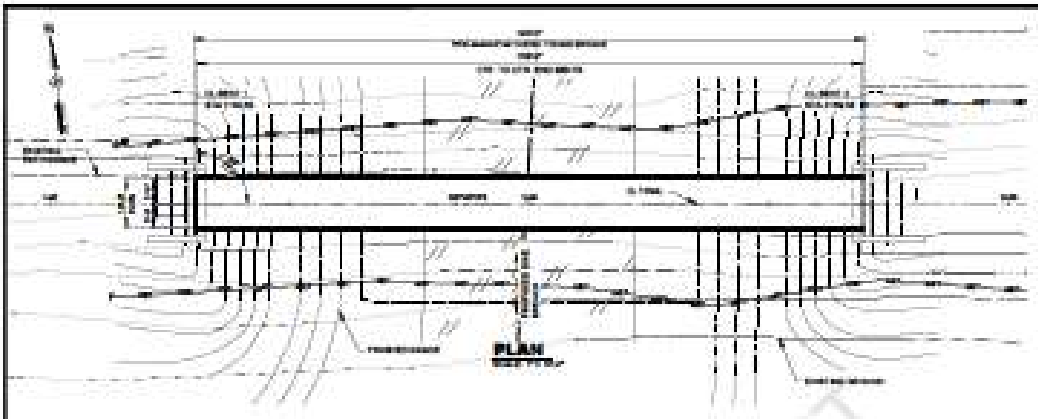
PRELIMINARY
NOT FOR CONSTRUCTION

RENEKE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

FOR CONSTRUCTION DOCUMENTATION

MISCELLANEOUS DETAILS

B10



PLAN



ELEVATION

DRAFT

PRELIMINARY
NOT FOR CONSTRUCTION

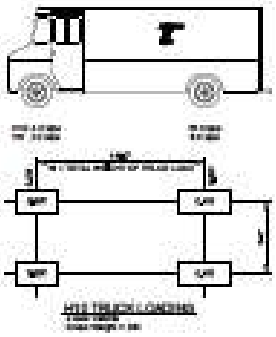
SITKA SEDGE
PEDESTRIAN BRIDGE

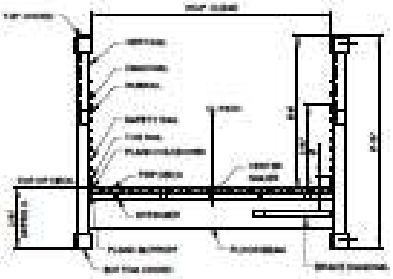
FOR CONSTRUCTION DOCUMENTATION

PLAN AND ELEVATION

BR01

GENERAL NOTES





TYPICAL SECTION

DRAFT

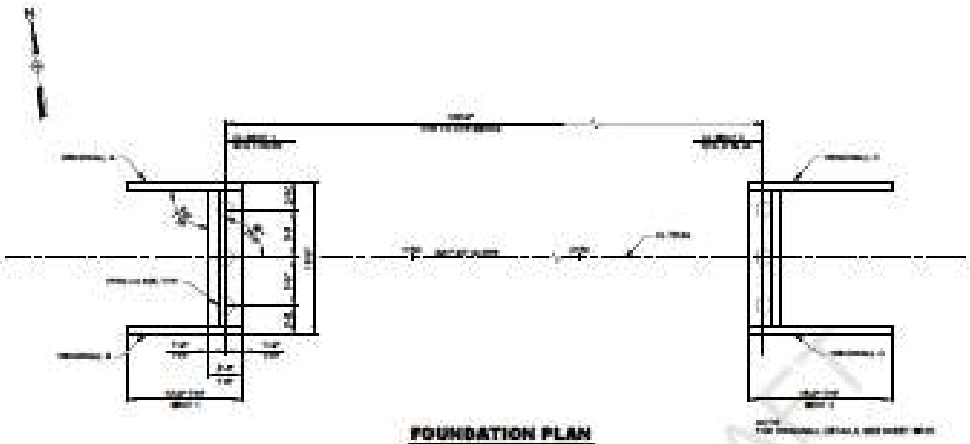
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NOT FOR CONSTRUCTION

STKA SEDGE
PEDESTRIAN BRIDGE

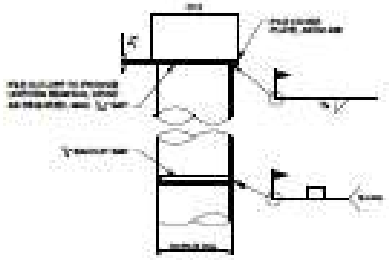
FOR CONSTRUCTION DOCUMENTATION

GENERAL NOTES

BR02



FOUNDATION PLAN



PIPE PILE DETAIL

FOUNDATION NOTES

PILE FABRICATION TABLE					

DRAFT

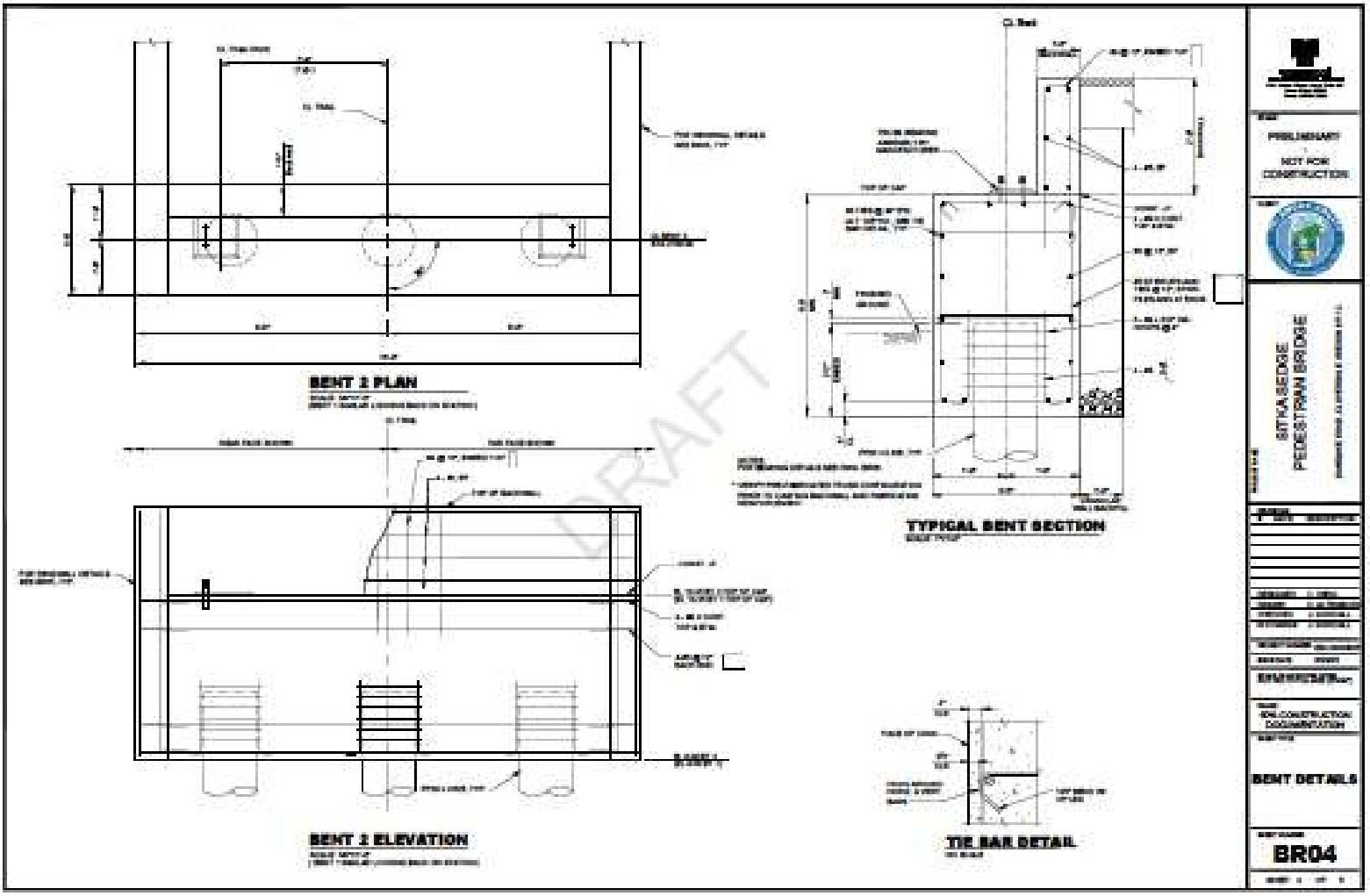
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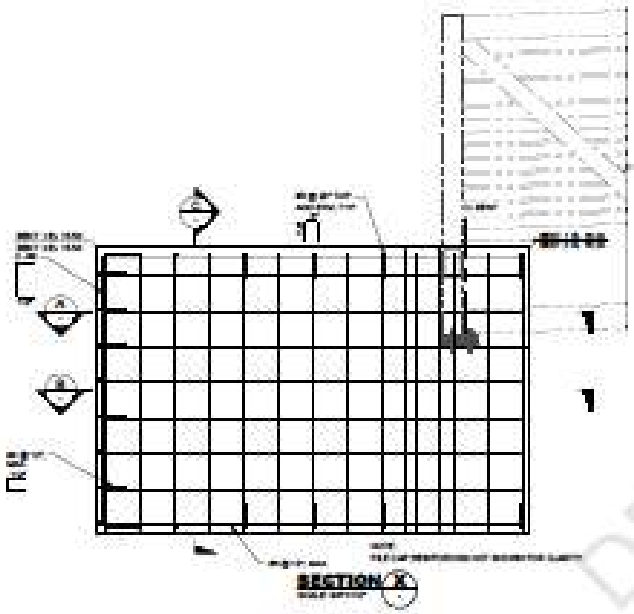
SITKA SEDGE
PEDESTRIAN BRIDGE

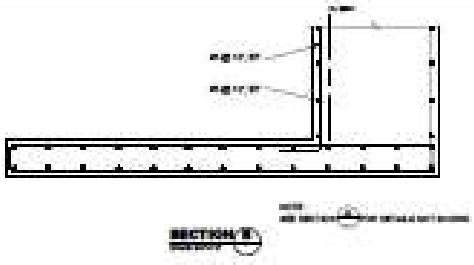
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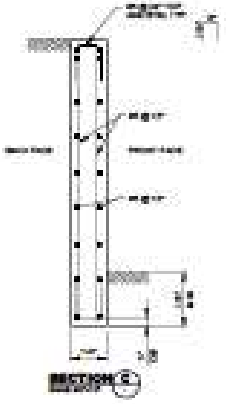
FOUNDATION PLAN

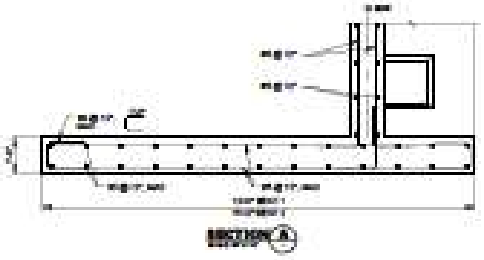
BR03


BENT 2 PLAN
TYPICAL BENT SECTION
BENT 2 ELEVATION
TIE BAR DETAIL
DRAFT
PRELIMINARY
NOT FOR CONSTRUCTION
SITKA SEDGE
PEDESTRIAN BRIDGE
FOR CONSTRUCTION DOCUMENTATION
BENT DETAILS
BR04









PRELIMINARY

NOT FOR CONSTRUCTION

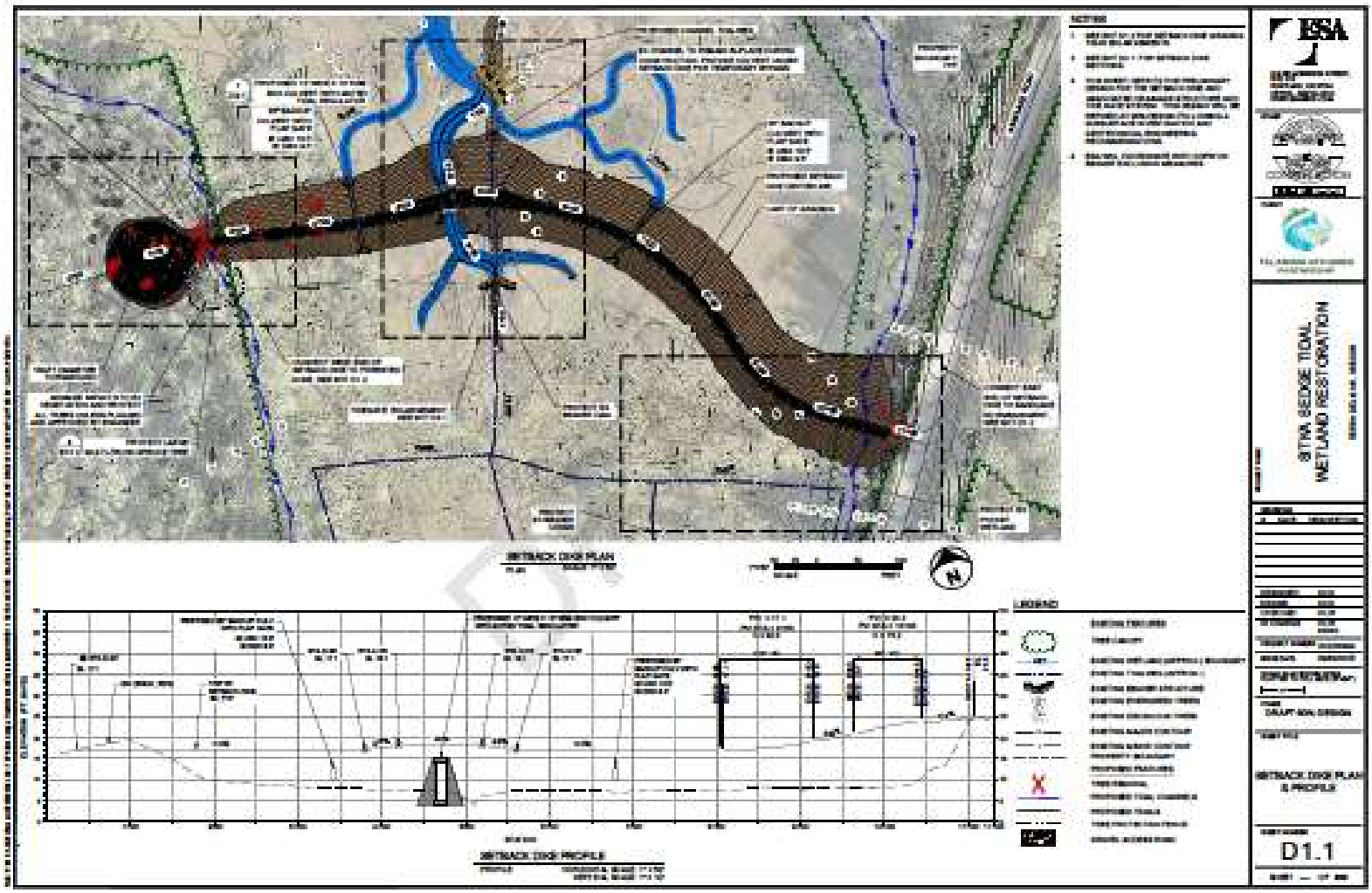
STKA SEOKIE
PEDESTRIAN BRIDGE

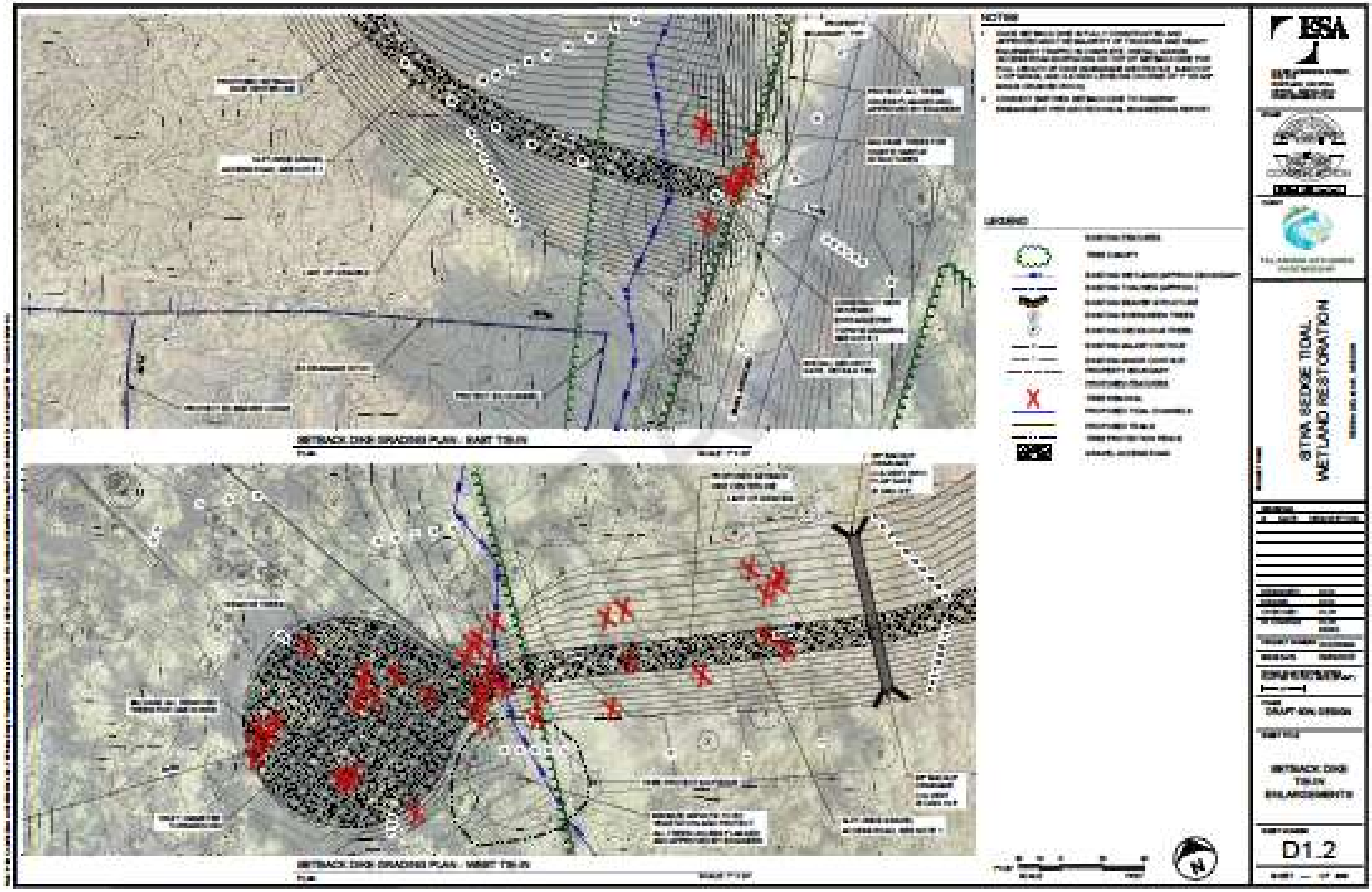
FOR CONSTRUCTION DOCUMENTATION

WINGWALL DETAILS

BR05

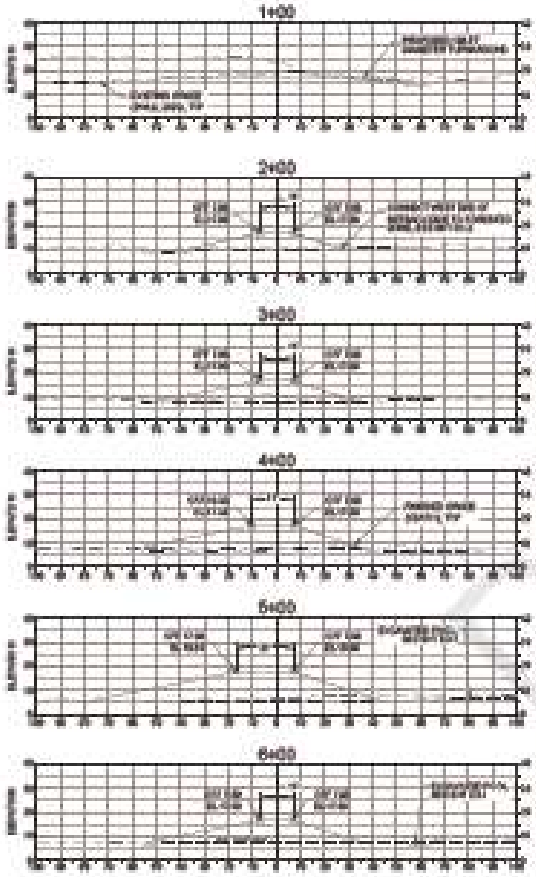
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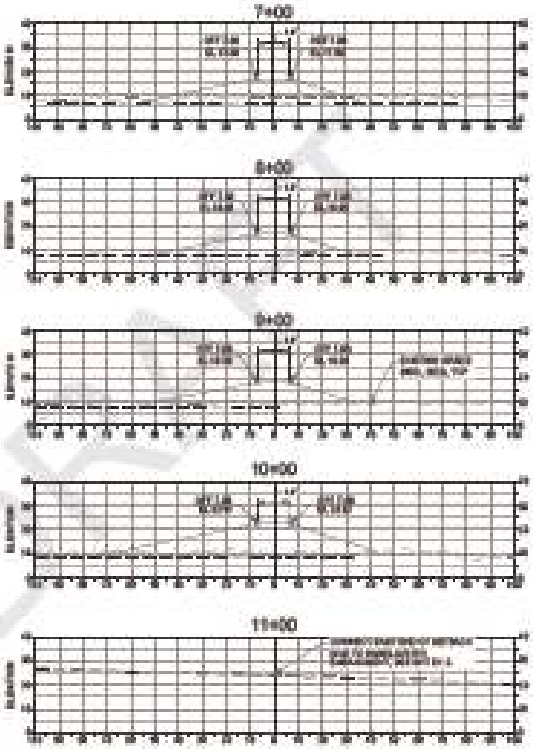

ESSA
STRA SEDGE TIDAL WETLAND RESTORATION
SETBACK DIKE PLAN
SETBACK DIKE PROFILE
LEGEND
DRAFT 60% DESIGN
SETBACK DIKE PLAN & PROFILE
D1.1


NOTES
LEGEND
ESA
D1.2

NOTES

1. SEE NOTES 1 FOR SETBACK DIKE PLAN & PROFILE





PROPOSED SETBACK DIKE
SECTIONS

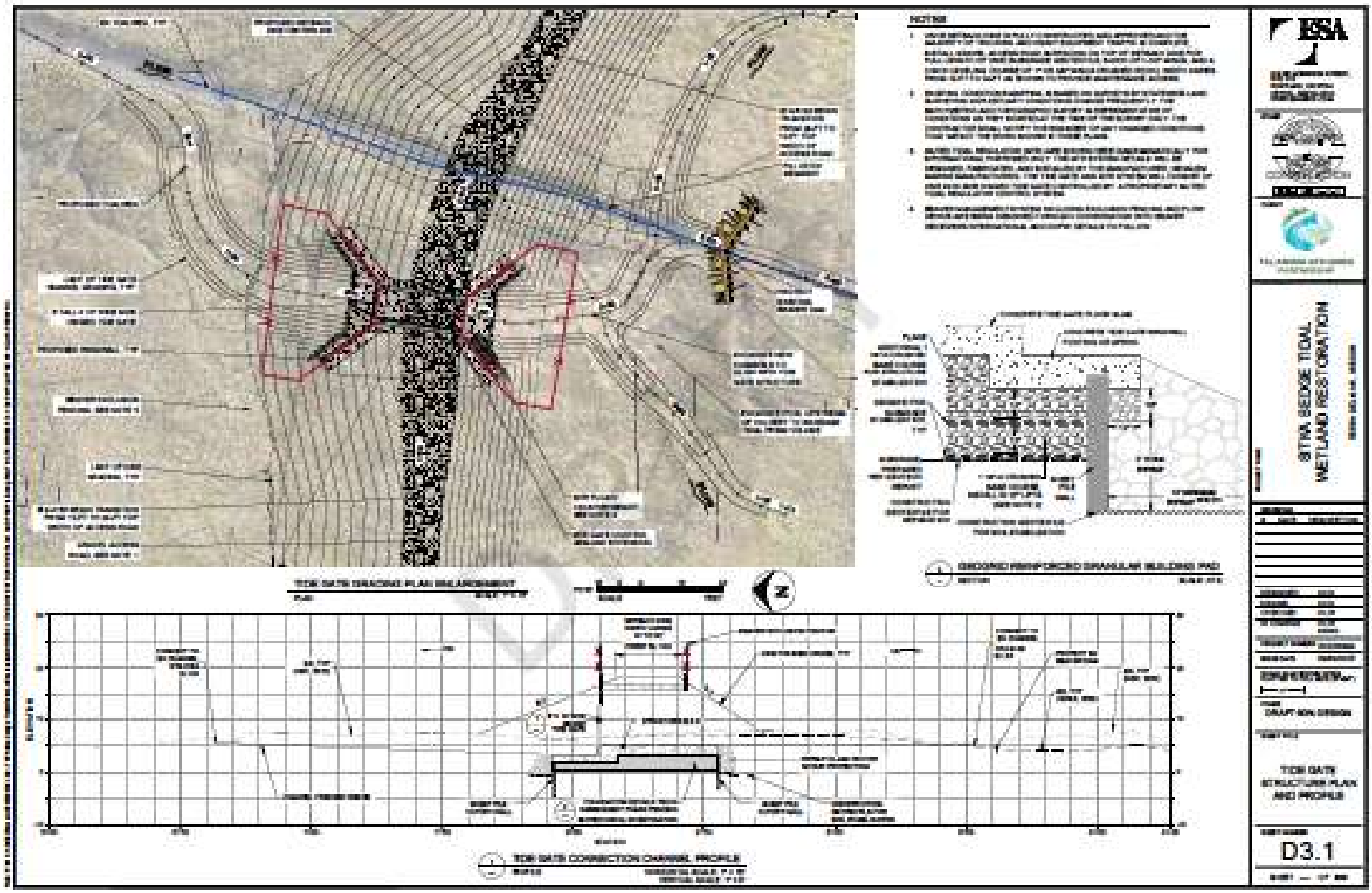
ESSA

SITKA SEDGE TIDAL WETLAND RESTORATION

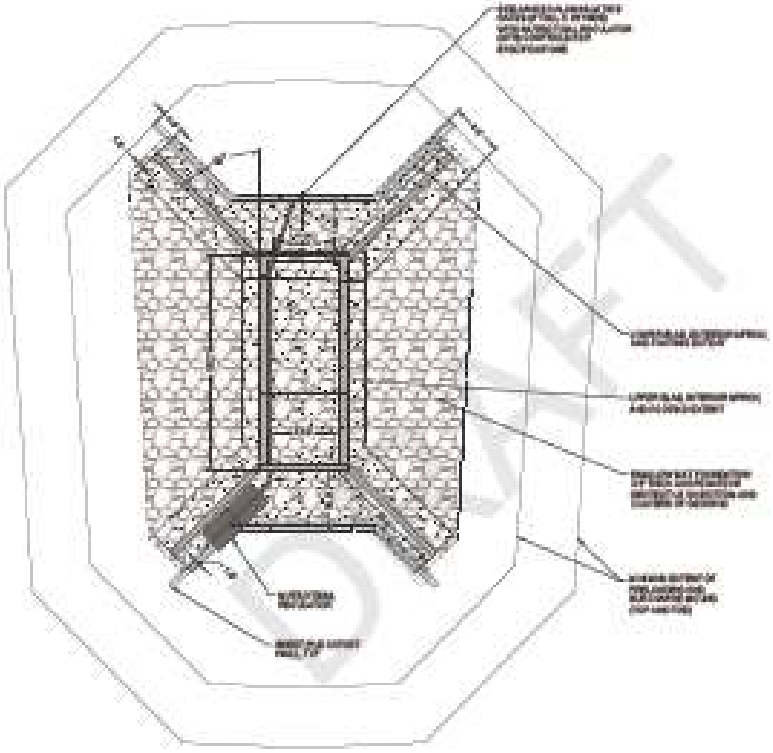
DRAFT 60% DESIGN

SETBACK DIKE SECTIONS

D2.1


ESSA
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT-90% DESIGN
TIDE GATE STRUCTURES PLAN AND PROFILE
D3.1

NOTES



ESSA

STRA SEDGE TIDAL WETLAND RESTORATION

DRAFT EIA DESIGN

TIDEGATE DETAIL

D3.2

SITKA SEDGE
TIDAL WETLAND RESTORATION
1200-C PERMIT

SHEET LIST

PROJECT TEAM

VICINITY MAP



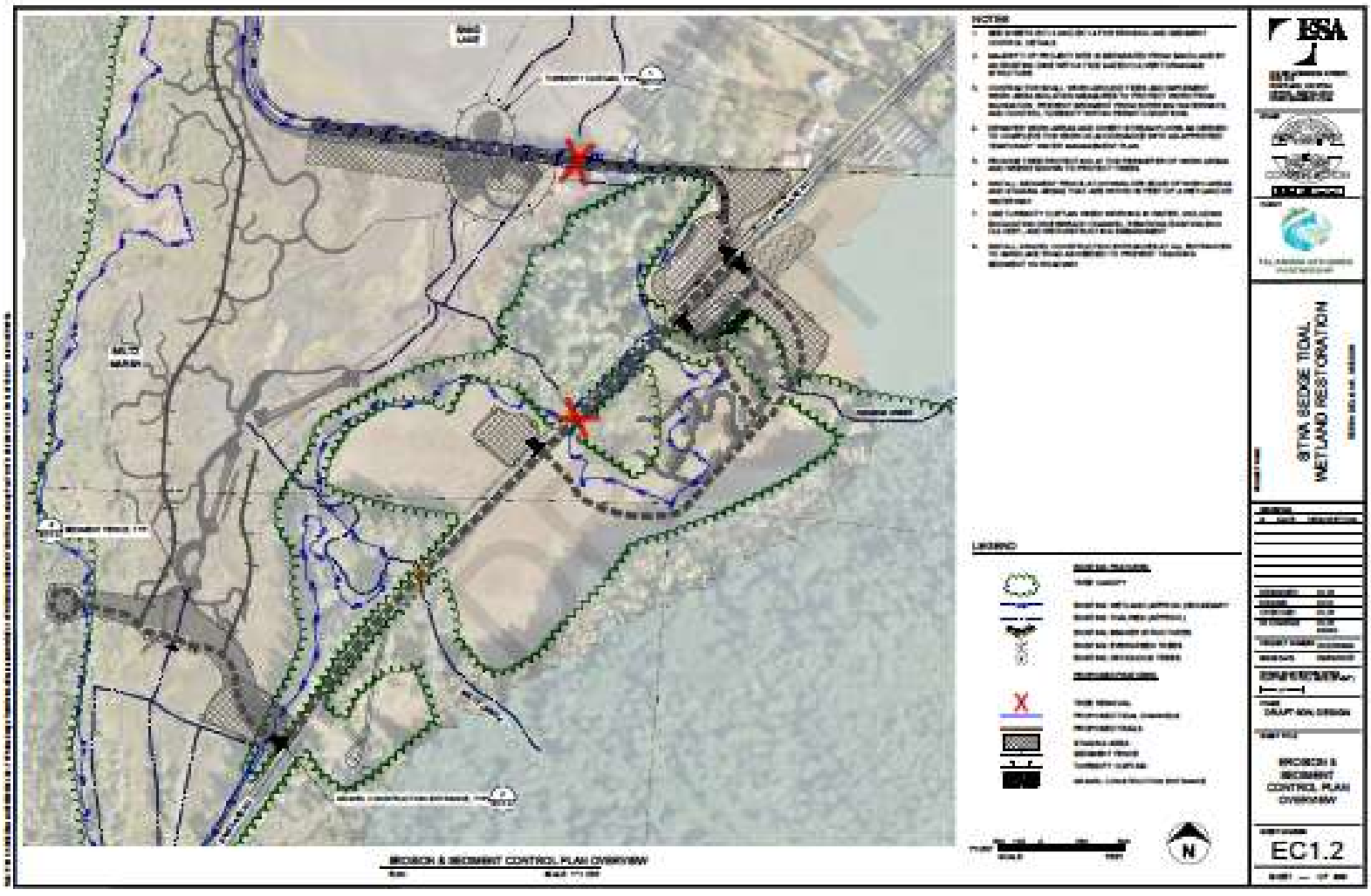
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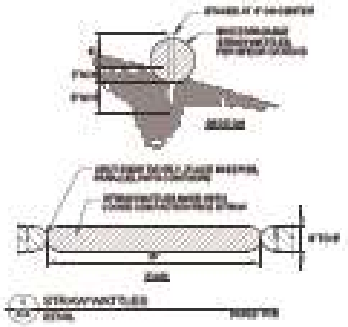
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SITKA SEDGE TIDAL WETLAND RESTORATION

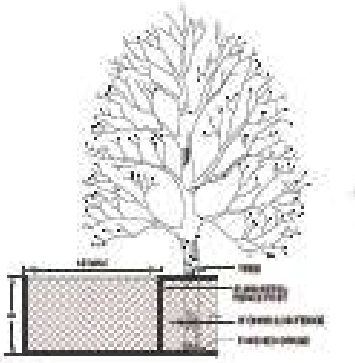
1200-C ESC COVER

EC1.1

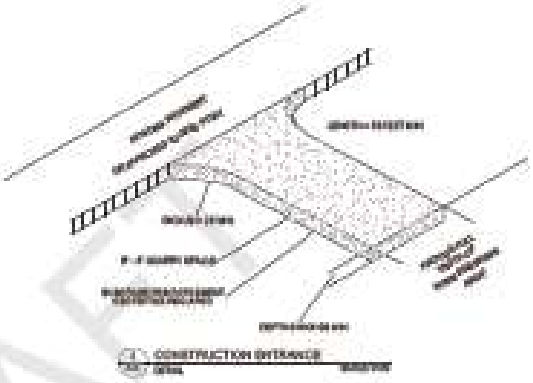

NOTES
LEGEND
BROACH & RECHARGE CONTROL PLAN OVERVIEW
ESSA
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT-90% DESIGN
BROACH & RECHARGE CONTROL PLAN OVERVIEW
EC1.2



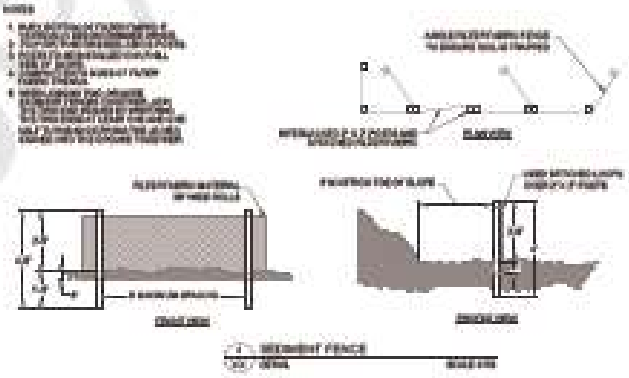
1 STRAW BALE DETAIL



2 TREE PROTECTION FENCE



3 CONSTRUCTION ENTRANCE



4 SEDIMENT FENCE

DRAFT

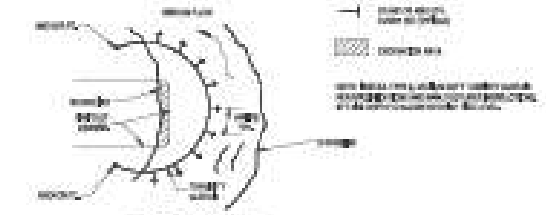
ESSA

SITKA SEDGE TIDAL WETLAND RESTORATION

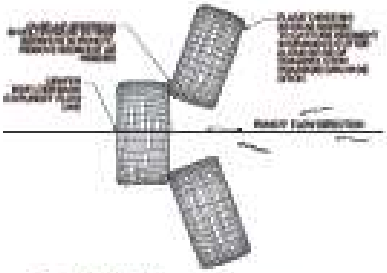
DRAFT EDA DESIGN

EROSION CONTROL DETAILS - 1

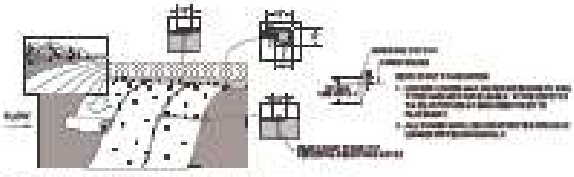
EC1.3



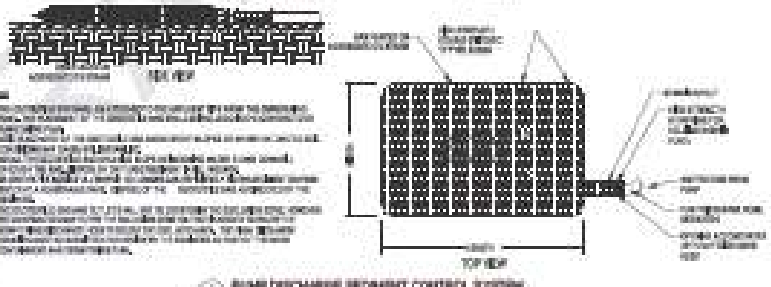
TURBIDITY CURTAIN



BIO FILTER BAG



COIR FABRIC



PUMP DISCHARGE SEDIMENT CONTROL SYSTEM



DEWATERING SYSTEM

ESSA

SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT-ESA DESIGN

EROSION CONTROL DETAILS - 2

EC1.4

NOTES

GENERAL NOTES

1. [redacted]

2. [redacted]

3. CONTRACTOR SHALL MAINTAIN COPIES OF THE APPROVED PLANS ON SITE AT ALL TIMES.

EROSION CONTROL NOTES

4. CONTRACTOR SHALL IMPLEMENT A STORM WATER POLLUTION PREVENTION PLAN (SWPPP) THAT INCLUDES, BUT IS NOT LIMITED TO, THE FOLLOWING:
   - COMPLY WITH ALL APPLICABLE LOCAL OR STATE AND FEDERAL REGULATIONS GOVERNING WATER AND POLLUTION.
   - FOLLOW APPLICABLE AGENCY STATE EXCEPTIONS, INCLUDING THE EXCEPTIONS OF THE VIRGINIA MARINE RESOURCES COMMISSION, AND VIRGINIA DEPARTMENT OF AGRICULTURE.
   - PERFORM ALL WORK IN ACCORDANCE WITH ALL LOCAL, COUNTY, AND STATE ORDINANCES, REGULATIONS, AND PERMIT REQUIREMENTS.

5. THE CONTRACTOR SHALL SUBMIT A WATER MANAGEMENT PLAN INCLUDING ALL PROPOSED CONSTRUCTION APPROVALS BY THE STATE REPRESENTATIVE PRIOR TO COMMENCEMENT OF ANY WORK. ANY CONSTRUCTION, WORK, OR USE NOT IN COMPLIANCE WITH THE APPROVED WATER MANAGEMENT PLAN MAY RESULT IN A STOP WORK ORDER AND/OR CIVIL PENALTIES. A COPY OF THE APPROVED PLAN SHALL BE KEPT ON SITE AT ALL TIMES DURING CONSTRUCTION.

6. [redacted]
   - [redacted]
   - [redacted]
   - [redacted]

7. ALL AREAS DISTURBED DURING CONSTRUCTION SHALL BE PROTECTED AND STABILIZED IN ACCORDANCE WITH THE APPROVED EROSION AND SEDIMENT CONTROL PLAN IMMEDIATELY AFTER EARTH DISTURBANCE ACTIVITIES CEASE. PERMANENT OR TEMPORARY SOIL STABILIZATION SHALL BE APPLIED TO DISTURBED AREAS WITHIN SEVEN (7) DAYS AFTER FINAL GRADE IS REACHED ON ANY PORTION OF THE SITE. TEMPORARY SOIL STABILIZATION SHALL BE APPLIED WITHIN SEVEN (7) DAYS TO ANY DISTURBED AREA THAT WILL REMAIN IDLE FOR MORE THAN 21 DAYS. STABILIZATION MEASURES SHALL BE APPROPRIATE FOR THE SEASON AND SITE CONDITIONS.

8. ALL EXCAVATED, FILLED, GRADED, AND COMPACTED AREAS SHALL BE PROTECTED AGAINST EROSION AND SEDIMENT TRANSPORT IN ACCORDANCE WITH THE APPROVED EROSION AND SEDIMENT CONTROL PLAN. [redacted]

9. ALL PERMANENT VEGETATION SHALL BE INSTALLED IN ACCORDANCE WITH THE APPROVED PLANTING PLAN AND SPECIFICATIONS. [redacted]

10. THE CONTRACTOR SHALL PROTECT ALL EXISTING VEGETATION, TREES, AND NATURAL FEATURES NOT DESIGNATED FOR REMOVAL. [redacted]

11. THE CONTRACTOR SHALL IMPLEMENT ADDITIONAL EROSION AND SEDIMENT CONTROL MEASURES AS DIRECTED BY THE ENGINEER OR REGULATORY AUTHORITY IF FIELD CONDITIONS REQUIRE. [redacted]

12. [redacted]

13. [redacted]

14. [redacted]

15. [redacted]

16. [redacted]

17. [redacted]

18. [redacted]

19. [redacted]

20. [redacted]

21. [redacted]

22. [redacted]

23. [redacted]

24. [redacted]

25. [redacted]

26. [redacted]

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28. [redacted]

29. [redacted]

30. [redacted]

31. [redacted]

DRAFT

ESSA

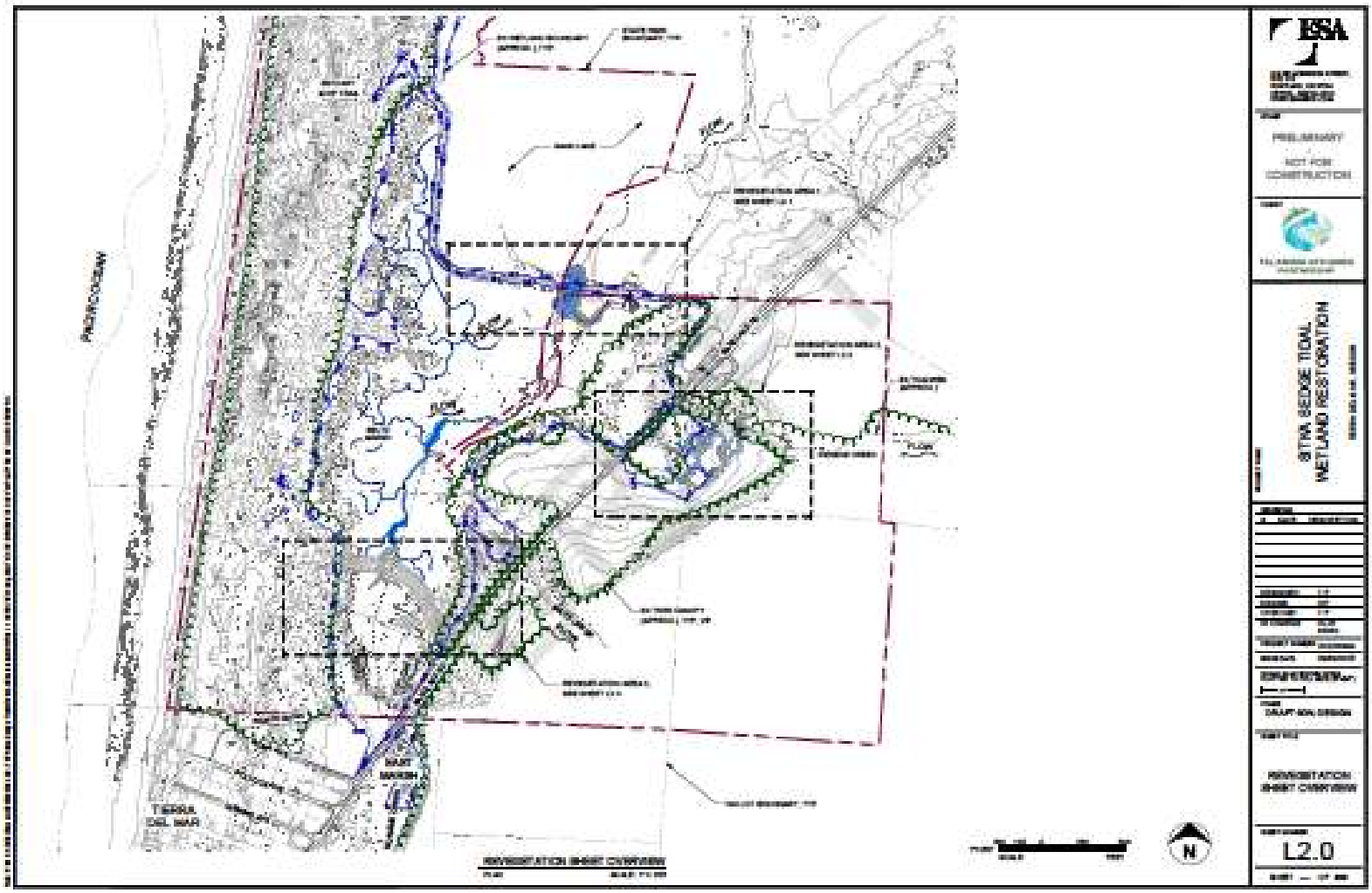
PRELIMINARY
-
NOT FOR CONSTRUCTION

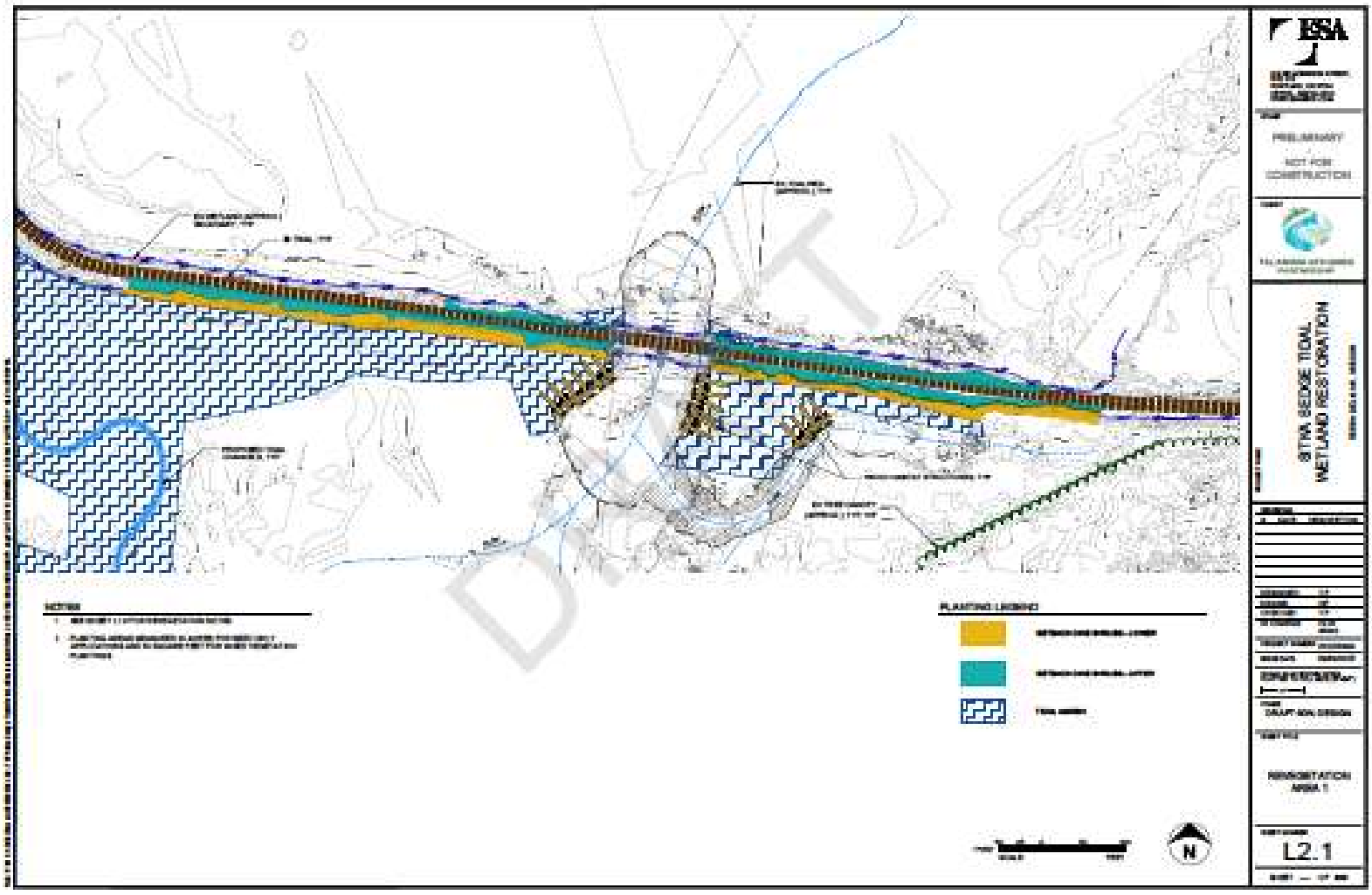
SITKA SEDGE TIDAL WETLAND RESTORATION

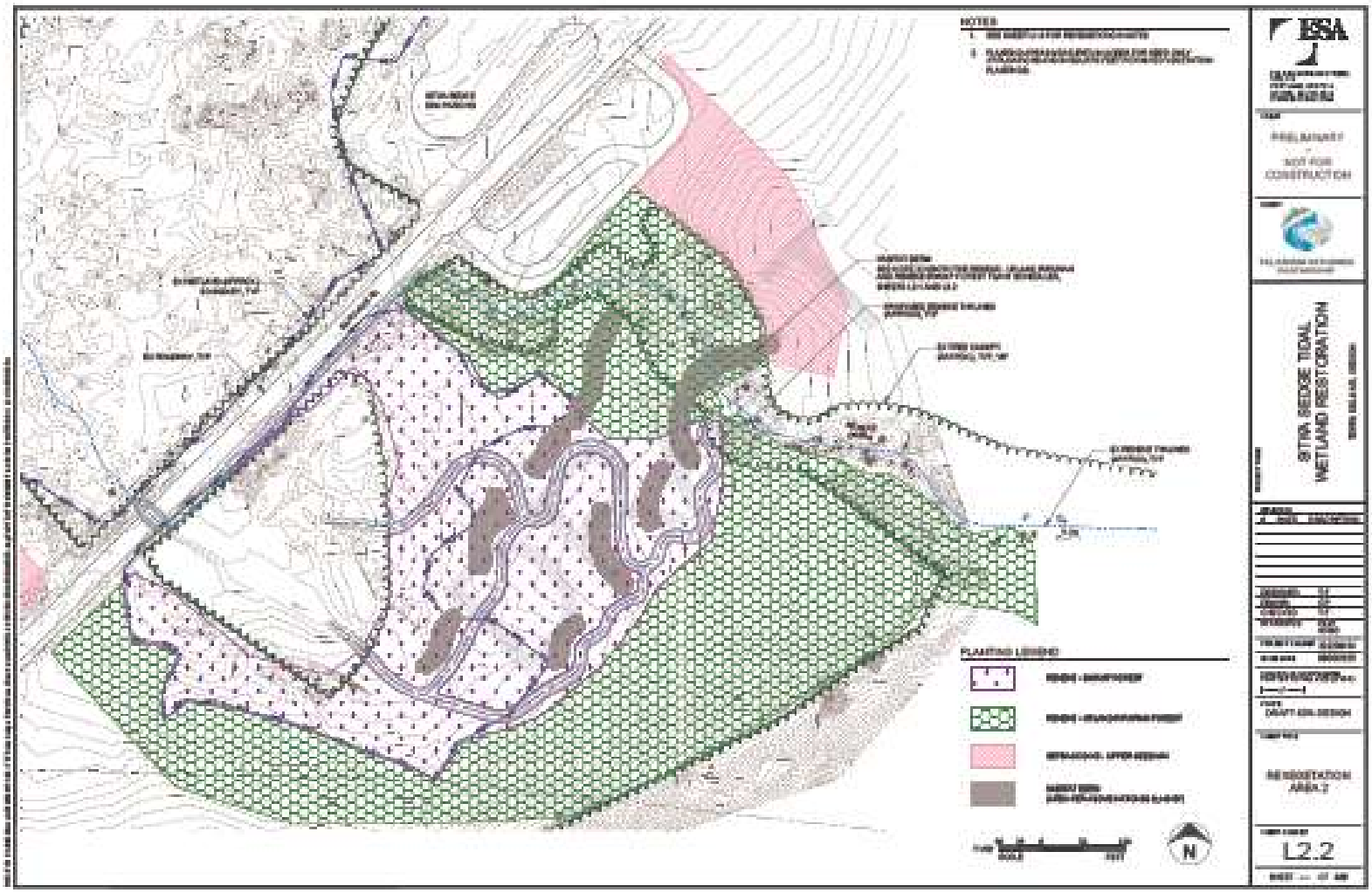
DRAFT ESA DESIGN

REVEGETATION NOTES

L1.0


PACIFIC OCEAN
TERRA DEL MAR
REVEGETATION SHEET OVERVIEW
ESSA
PRELIMINARY
NOT FOR CONSTRUCTION
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT 60% DESIGN
REVEGETATION SHEET OVERVIEW
L2.0


ESSA
PRELIMINARY
NOT FOR CONSTRUCTION
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT
NOTES
PLANTING LEGEND
DRAFT-90% DESIGN
REVEGETATION AREA 1
L2.1
N



ESSA

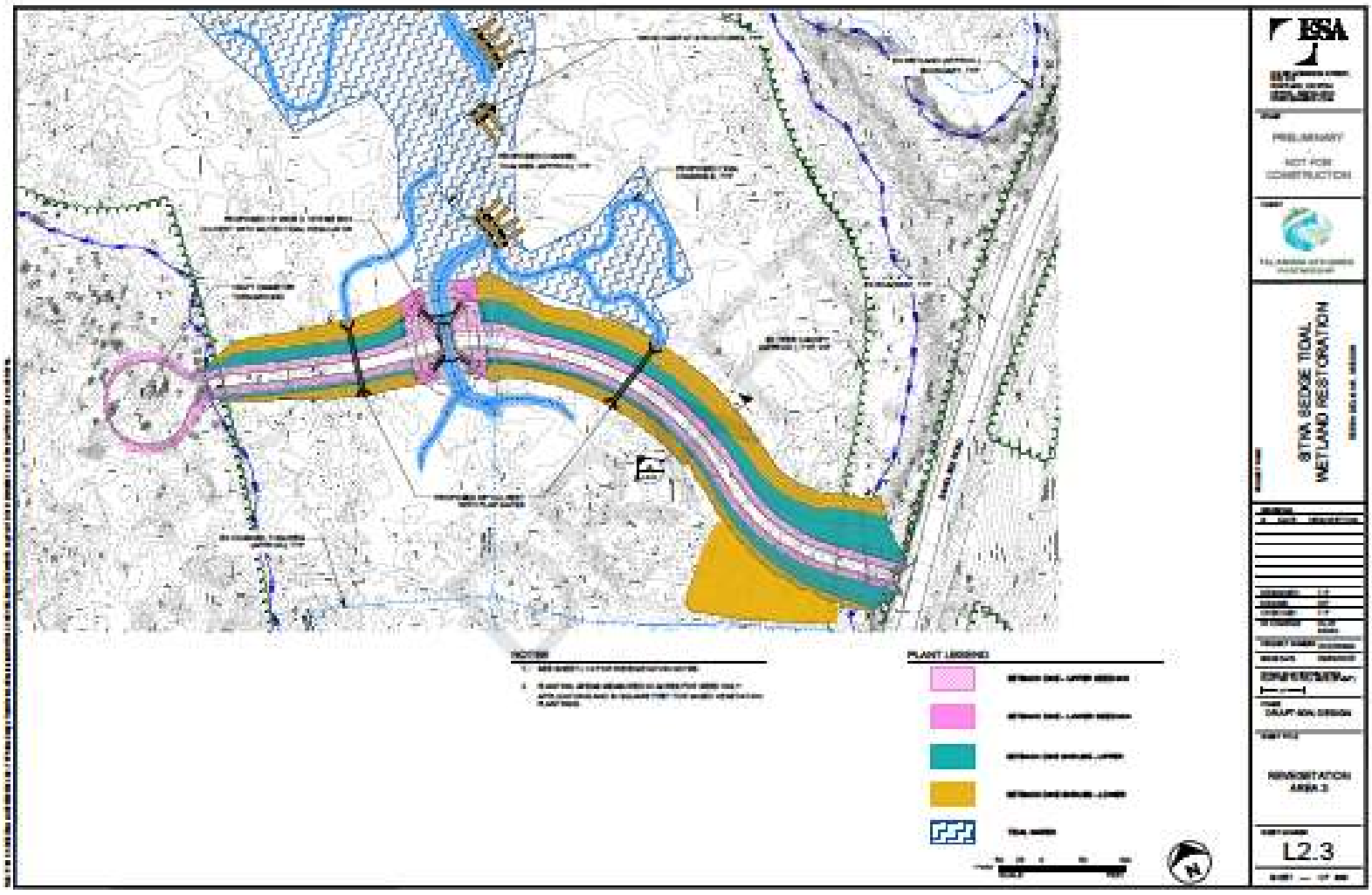
PRELIMINARY
-
NOT FOR CONSTRUCTION

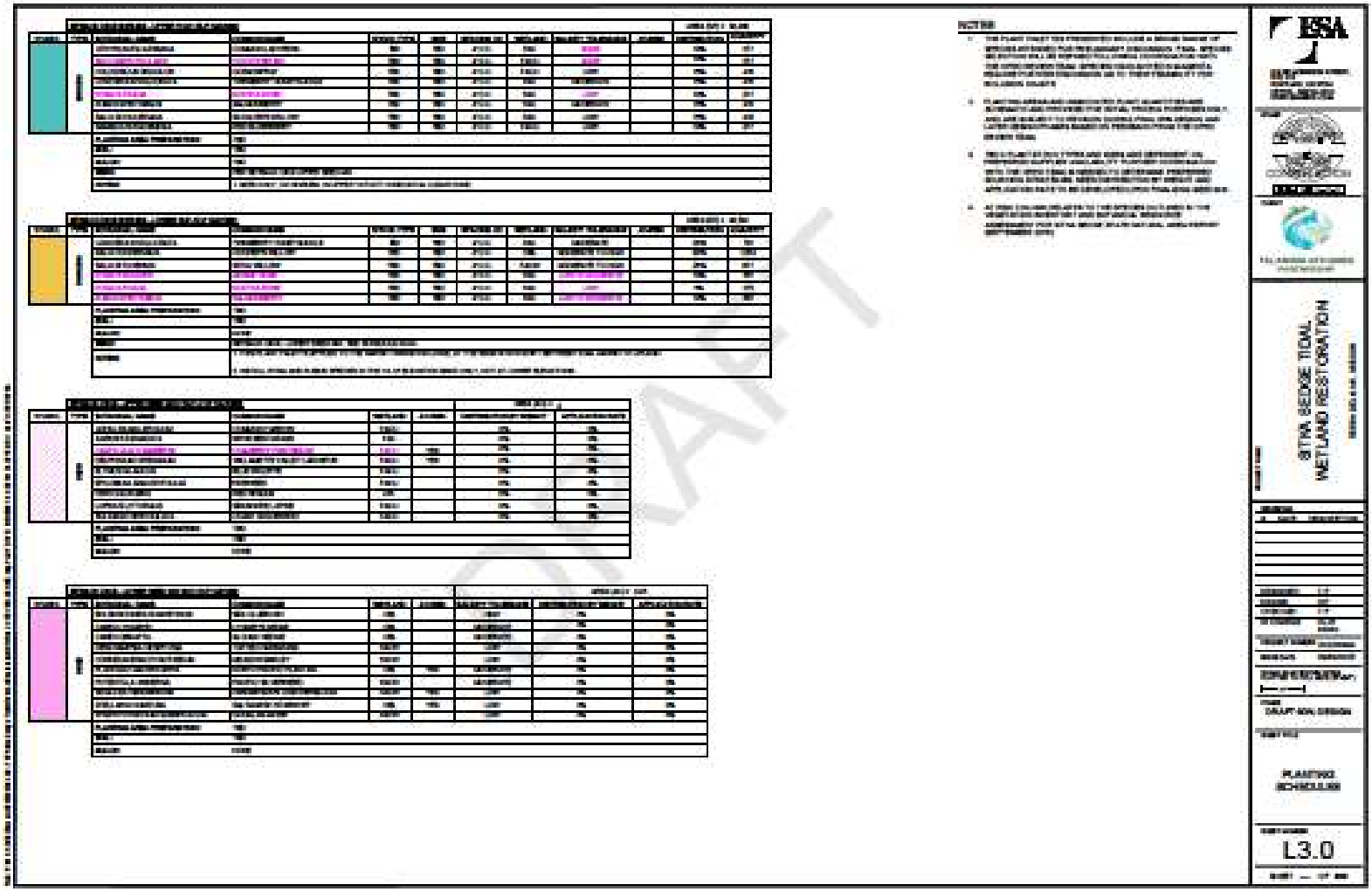
SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT EIA DESIGN

REVEGETATION AREA 2

L2.2


ESSA
PRELIMINARY
NOT FOR CONSTRUCTION
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT 60% DESIGN
REVEGETATION AREA 3
L2.3



NOTES

1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]

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[illegible]

STRA SEDGE TIDAL WETLAND RESTORATION

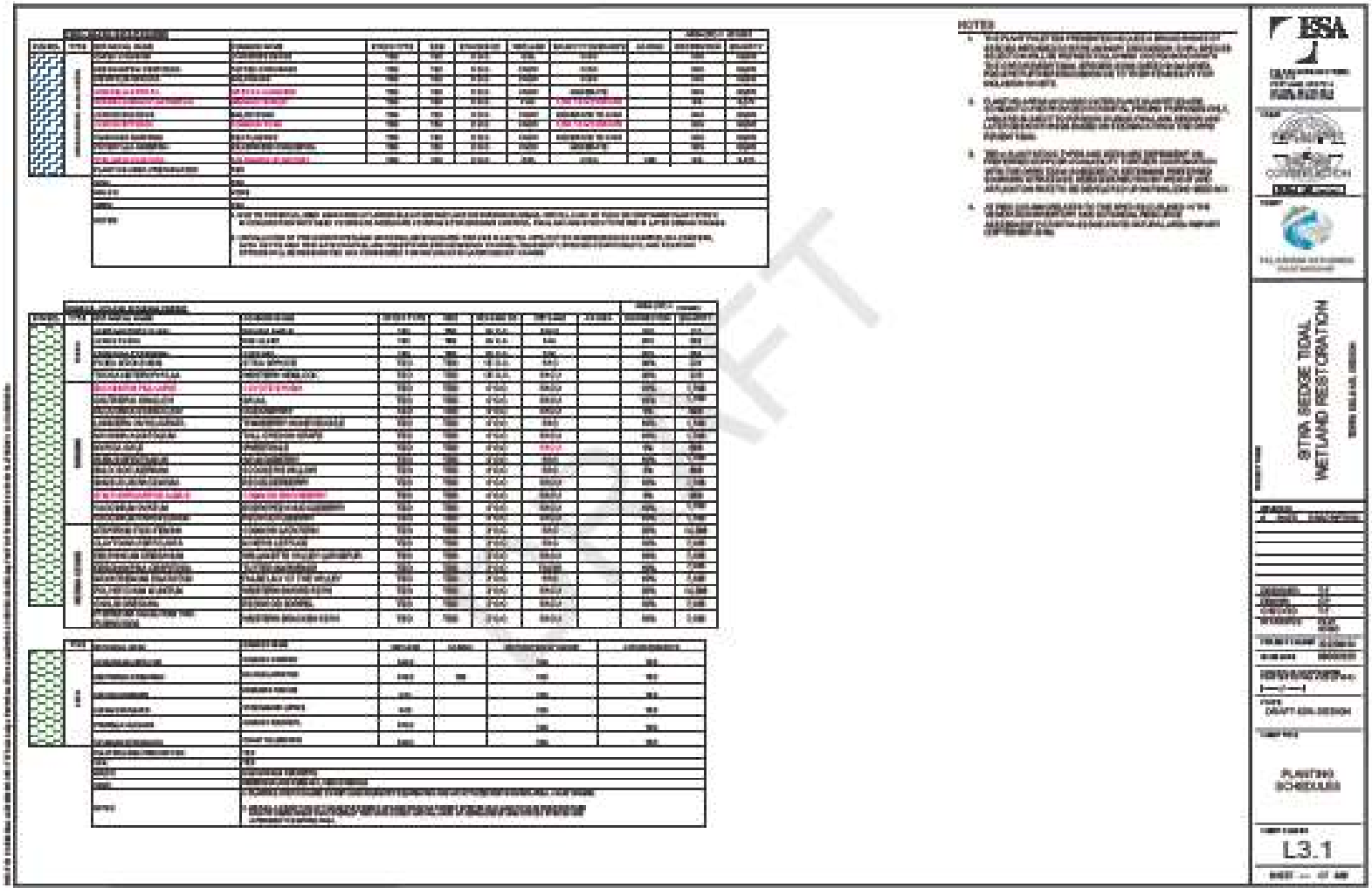
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DRAFT-NON-DESIGN

PLANTING SCHEDULES

L3.0

DRAFT



NOTES

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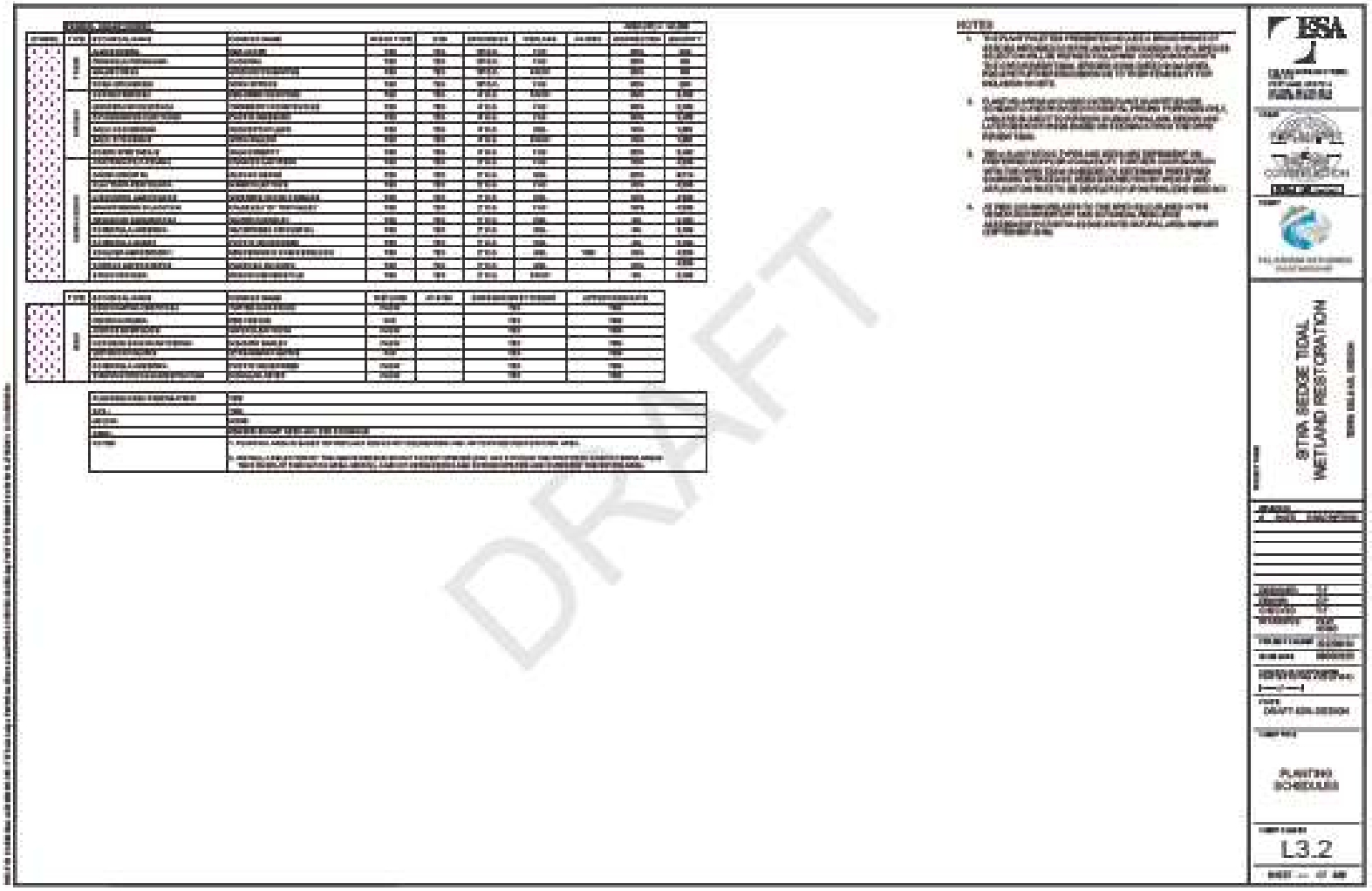
ESSA

STRA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

PLANTING SCHEDULES

L3.1



NOTES

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2.

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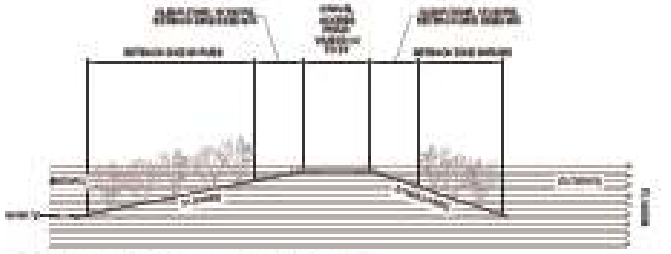
SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

PLANTING SCHEDULES

L3.2

NOTES



SETBACK DIKE TYPICAL CROSS SECTION - PLANTING

SCALE: 1/4" = 1'-0"

DRAFT

ESSA

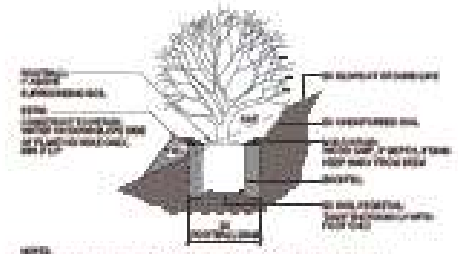
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SITKA SEDGE TIDAL WETLAND RESTORATION

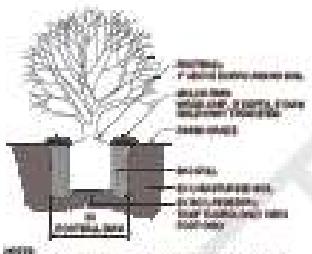
DRAFT 60% DESIGN

TYPICAL PLANTING PROFILES

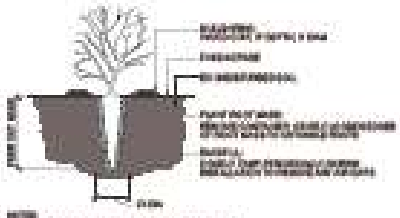
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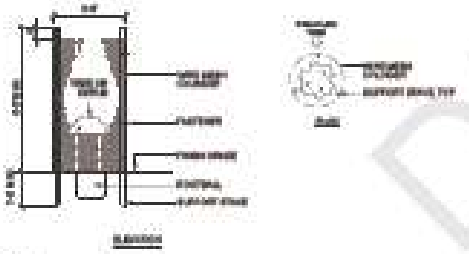
1 CONTAINER PLANTING – ON SLOPE
SCALE: NTS



2 CONTAINER PLANTING - STANDARD
SCALE: NTS



3 D40/BENTPA PLANTING
SCALE: NTS



4 PROTECTION CAGE
SCALE: NTS

DRAFT

ESSA

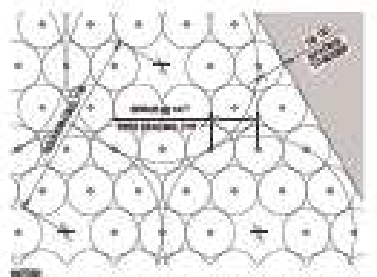
PRELIMINARY
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SITKA SEDGE TIDAL WETLAND RESTORATION

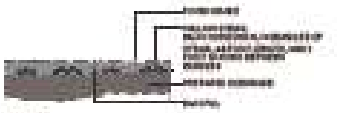
DRAFT ESA DESIGN

PLANTING DETAILS

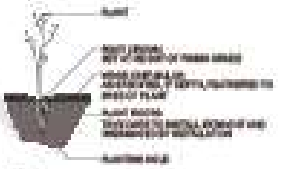
L5.0



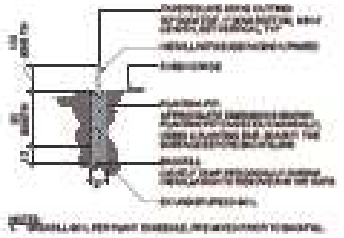
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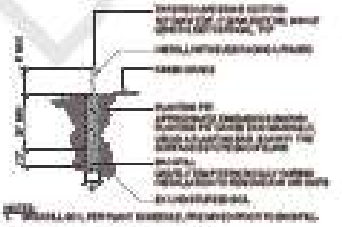
2 HORIZONTAL PLANTING



3 PLUG OR BARESOOT PLANTING



4 LIVE STAKE PLANTING - STANDARD



5 LIVE STAKE PLANTING - SHORT

DRAFT

ESSA

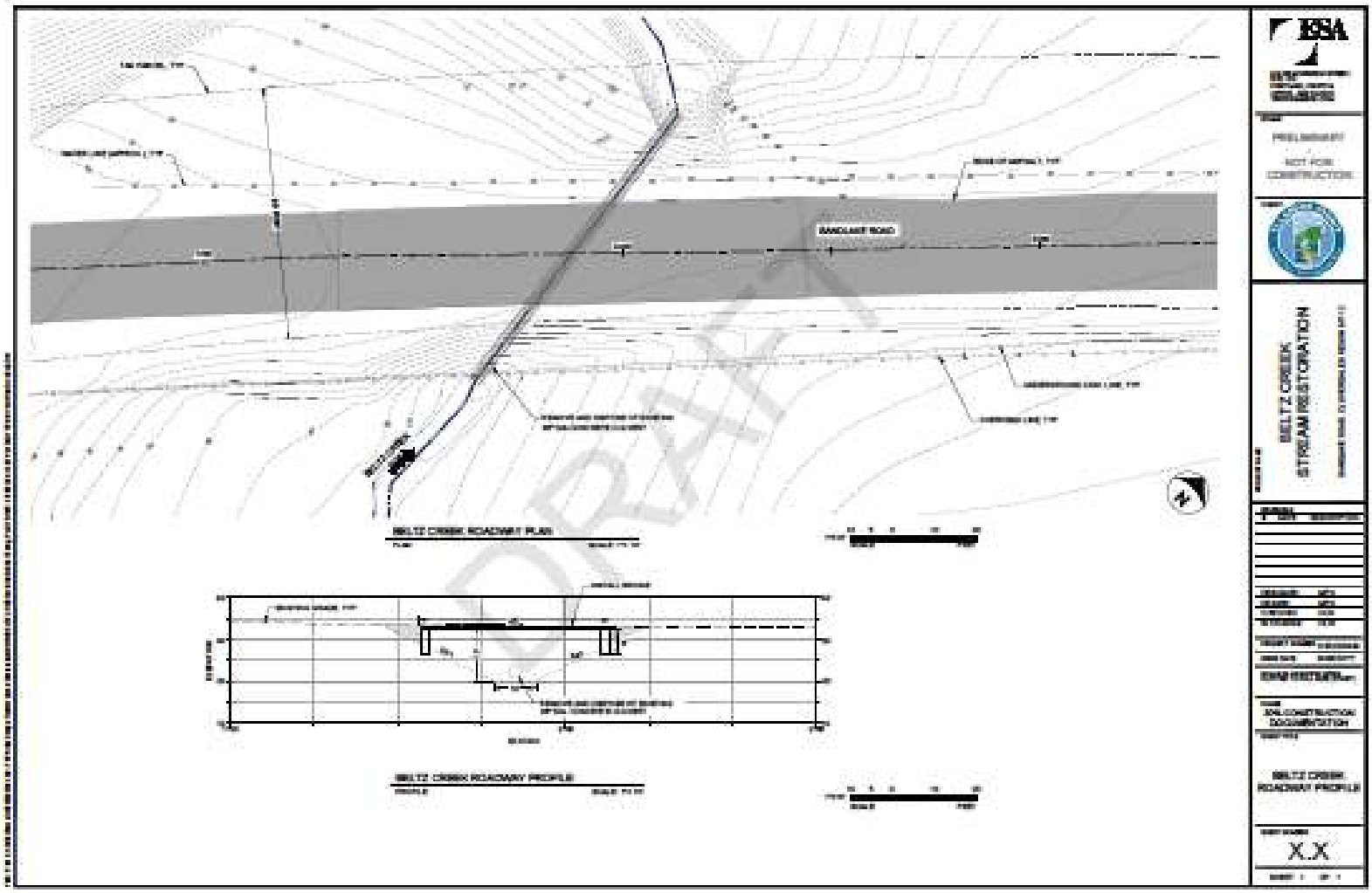
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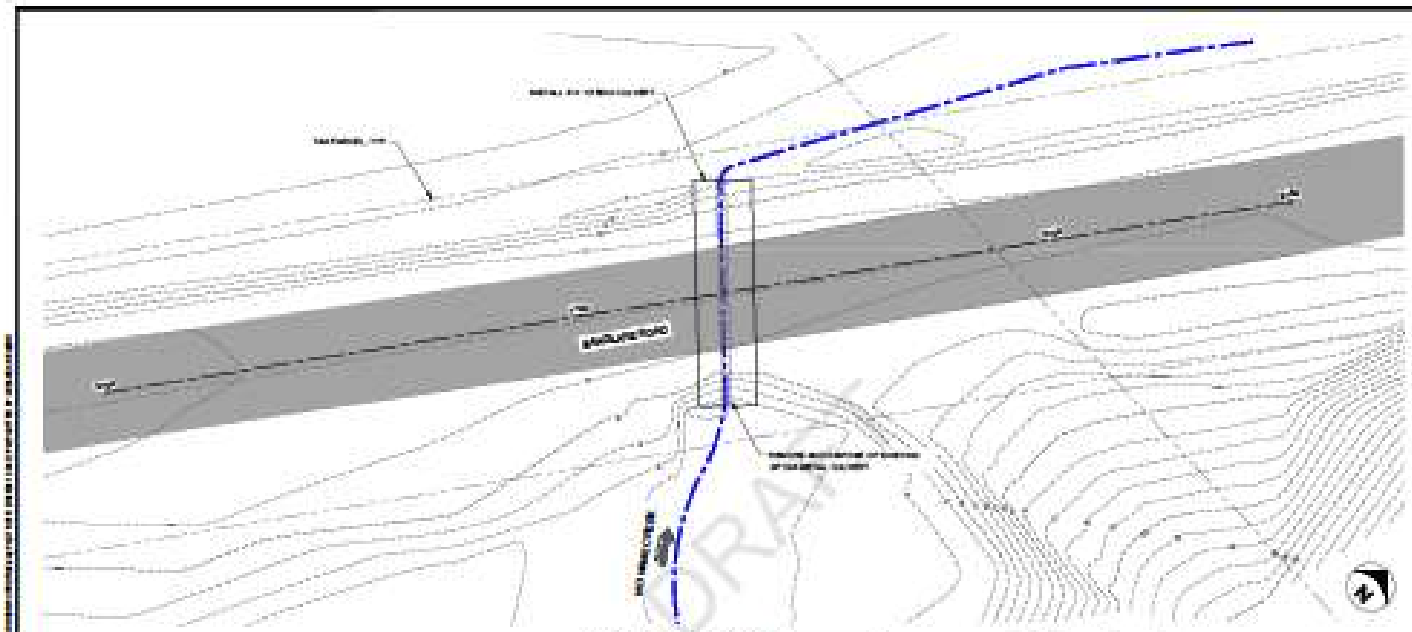
SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT ESA DESIGN

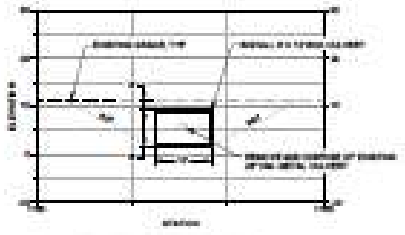
PLANTING DETAILS

L5.1


SANDUMP ROAD
BELTZ CREEK ROADWAY PLAN
BELTZ CREEK ROADWAY PROFILE
ESSA
PRELIMINARY
NOT FOR CONSTRUCTION
BELTZ CREEK STREAM RESTORATION
90% CONSTRUCTION DOCUMENTATION
BELTZ CREEK ROADWAY PROFILE
X.X



NO NAME CREEK ROADWAY PLAN



NO NAME CREEK ROADWAY PROFILE

ESA

PRELIMINARY
NOT FOR CONSTRUCTION

NO NAME CREEK CULVERT REPLACEMENT

ESA CONSTRUCTION DOCUMENTATION

NO NAME CREEK ROADWAY PROFILE

X.X

APPENDIX B: FIELD EXPLORATIONS

Appendix B

Field Explorations

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Figure A7: Log of Boring B-5
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Figure A9: Log of Boring B-7

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B.1 GENERAL

Body Text

Shannon & Wilson explored subsurface conditions at the project site with seven geotechnical borings designated B-1 through B-7. The locations of the explorations were located in the field using a GPS, and points were taken to reference the locations. The approximate exploration locations are shown on the Site and Exploration Plan, Figure 2. This appendix describes the techniques used to perform the explorations and presents logs of the materials encountered.

B.2 GEOTECHNICAL DRILLING

Borings B-3 through B-7 were drilled within the roadway of Sandlake Rd between May 24, 2025, through May 28, 2025. Borings B-1 and B-2 were drilled along trailhead of embankment between March 31, 2025, through April 1, 2025. The borings were drilled using CME-55LCX track rig provided and operated by Western States Soil Conservation Inc. (Western States) of Hubbard, Oregon. A Shannon & Wilson geologist was present during the explorations to locate the borings, observe the drilling, collect soil samples, and log the materials encountered. Below are descriptions of the drilling method used during the drilling.

B.2.1 Open Hole Mud Rotary Drilling

Mud-rotary borings are typically advanced using a tri-cone bit and a string of hollow drill rods (narrower than the bit) through which bentonite drilling mud is pumped. The mud is mixed on site using water and powdered bentonite. The drilling mud serves to cool the bit, keep the hole open, and flush the cuttings to the surface and the borehole is considered "open hole" since the borehole is not cased. Returning drill mud is typically passed through a screen and into a tub over the hole. The screen collects the cuttings, and the tub collects the mud for recirculation back into the hole. If fine-grained, cohesive soils are encountered, other styles of drill bits may also be used with the mud-rotary method, such as scraper or drag bits. The common size mud rotary drill bit is 4 7/8-inch, which produces an approximate 5-inch-diameter hole.

B.3 SOIL SAMPLING

B.3.1 Disturbed Sampling

Disturbed samples were collected in the borings, typically at 2.5- to 5-foot depth intervals, using a standard 2-inch-outside-diameter (O.D.) split spoon sampler in conjunction with Standard Penetration Testing. In a Standard Penetration Test (SPT), ASTM D1586, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance, or N-value. The SPT N-value provides a measure of in situ relative density of cohesionless soils (silt, sand, and gravel), and the consistency of cohesive soils (silt and clay). All disturbed samples were visually identified and described in the field, sealed to retain moisture, and returned to our laboratory for additional examination.

SPT N-values can be significantly affected by several factors, including the efficiency of the hammer used. Automatic hammers generally have higher energy transfer efficiencies than cathead-driven (manual) hammers. For reference, cathead hammers are typically assumed to have an average energy efficiency of 60 percent. Based on information we received from Western States, the energy efficiency of the hammer used on the rubber track mounted CME-55LCX drill rig averaged 85.0 percent. All N-values presented in this report are in blows per foot, as counted in the field. No corrections of any kind have been applied.

An SPT was considered to have met refusal where more than 50 blows were required to drive the sampler 6 inches. If refusal was encountered in the first 6-inch interval (for example, 50 for 1.5"), the count is reported as 50/1st 1.5". If refusal was encountered in the second 6-inch interval (for example, 48, 50 for 1.5"), the count is reported as 50/1.5". If refusal was encountered in the last 6-inch interval (for example, 39, 48, 50 for 1.5"), the count is reported as 98/7.5".

B.3.2 Relatively Undisturbed Sampling

Relatively undisturbed samples were attempted in 3-inch O.D. thin-wall Shelby tubes which were hydraulically pushed into the soil at the bottom of the boreholes. The recovered soils exposed at the ends of the tube were examined and described in the field. After examination, the tubes were sealed to preserve the natural moisture of the samples. The sealed tubes were stored in the upright position, and care was taken to avoid shock and vibration during their transport to and storage in our laboratory.

B.4 MATERIAL DESCRIPTIONS

In the field, soil samples were described and identified visually in general accordance with ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Consistency, color, relative moisture, degree of plasticity, peculiar odors and other distinguishing characteristics of the samples were noted. Once returned to the laboratory, soil samples were re-examined, various laboratory tests were performed, and the field descriptions and identifications were modified as necessary. We refined our visual-manual soil descriptions and identifications based on the results of the laboratory tests, using elements of the Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), ASTM D2487. However, ASTM D2487 was not followed in full, because it requires a suite of tests be performed to classify a single sample. The specific terminology used in the soil classifications is defined on the Soil Description and Log Key, Figure B1. The specific terminology used in the soil classifications is defined on the Rock Classification and Log Key, Figure B2.

B.5 LOGS OF BORINGS

The summary logs of Shannon & Wilson geotechnical borings are presented in Figures B3 through B9. Material descriptions and interfaces on the logs are interpretive, and actual changes may be gradual. The left-hand portion of the boring logs provides descriptions, identifications, and geotechnical unit designations for the materials encountered in the boring. The right-hand portion of the boring logs shows a graphic log, sample locations and designations, backfill details, and a graphical representation of N-values, natural water contents, percent passing the #200 sieve, and sample recovery.

B.6 BOREHOLE ABANDONMENT

Upon completion of drilling the borings were backfilled with bentonite chips or bentonite/cement grout in accordance with Oregon Department of Water Resources regulations. Cold patch asphalt pavement was used to seal the surface of the borings performed within Sandlake Road.

Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT[1]	FINE-GRAINED SOILS (50% or more fines)[1]	COARSE-GRAINED SOILS (less than 50% fines)[1]
Major	*Silt, Lean Clay, Elastic Silt,* or *Fat Clay*[3]	*Sand* or *Gravel*[4]
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: *Sandy* or *Gravelly*[4]	More than 12% fine-grained: *Silty* or *Clayey*[3]
Minor Follows major constituent	15% to 30% coarse-grained: *with Sand* or *with Gravel*[4]	5% to 12% fine-grained: *with Silt* or *with Clay*[3]
	30% or more total coarse-grained and lesser coarse-grained constituent is 15% or more: *with Sand* or *with Gravel*[5]	15% or more of a second coarse-grained constituent: *with Sand* or *with Gravel*[5]

[1] All percentages are by weight of total specimen passing a 3-inch sieve.
[2] The order of terms is: *Modifying Major with Minor*.
[3] Determined based on behavior.
[4] Determined based on which constituent comprises a larger percentage.
[5] Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.

NOTE: Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

PARTICLE SIZE DEFINITIONS

DESCRIPTION	SIEVE NUMBER AND/OR APPROXIMATE SIZE
FINES	< #200 (0.075 mm = 0.003 in.)
SAND Fine Medium Coarse	#200 to #40 (0.075 to 0.4 mm; 0.003 to 0.02 in.) #40 to #10 (0.4 to 2 mm; 0.02 to 0.08 in.) #10 to #4 (2 to 4.75 mm; 0.08 to 0.187 in.)
GRAVEL Fine Coarse	#4 to 3/4 in. (4.75 to 19 mm; 0.187 to 0.75 in.) 3/4 to 3 in. (19 to 76 mm)
COBBLES	3 to 12 in. (76 to 305 mm)
BOULDERS	> 12 in. (305 mm)

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS		COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 4	Very loose	< 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
> 50	Very dense	15 - 30	Very stiff
		> 30	Hard

WELL AND BACKFILL SYMBOLS

- Bentonite Cement Grout
- Bentonite Grout
- Bentonite Chips
- Silica Sand
- Gravel
- Perforated or Screened Casing
- Surface Cement Seal
- Asphalt or Cap
- Slough
- Inclinometer or Non-perforated Casing
- Vibrating Wire Piezometer

PERCENTAGES TERMS [1,2]

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

[1] Gravel, sand, and fines estimated by mass. Other constituents, such as organics, cobbles, and boulders, estimated by volume.

[2] Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

Tillamook Estuary Partners
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

SOIL DESCRIPTION AND LOG KEY

May 2025 112880

SHANNON & WILSON

FIG. B1
Sheet 1 of 3

2013_SD.RING_CLASS1 112880 GEO SWOOTS, BRANYPOX, GLB SWSEN_GDT 5/5/25

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488)

MAJOR DIVISIONS			GROUP SYMBOL	GRAPHIC SYMBOL	TYPICAL IDENTIFICATIONS
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Gravel *(less than 5% fines)*	GW		Well-Graded Gravel; Well-Graded Gravel with Sand
			GP		Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel *(more than 12% fines)*	GM		Silty Gravel; Silty Gravel with Sand
			GC		Clayey Gravel; Clayey Gravel with Sand
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Sand *(less than 5% fines)*	SW		Well-Graded Sand; Well-Graded Sand with Gravel
			SP		Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand *(more than 12% fines)*	SM		Silty Sand; Silty Sand with Gravel
			SC		Clayey Sand; Clayey Sand with Gravel
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML		Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL		Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL		Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH		Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH		Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH		Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT		Peat or other highly organic soils (see ASTM D4427)
FILL	Placed by humans, both engineered and nonengineered. May include various soil materials and debris.				The Fill graphic symbol is combined with the soil graphic that best represents the observed material

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.
2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.
3. The soil graphics above represent the various USCS identifications (i.e., *GP*, *SM*, etc.) and may be augmented with additional symbology to represent differences within USCS designations. *Sandy Silt (ML)*, for example, may be accompanied by the *ML* soil graphic with sand grains added. Non-USCS materials may be represented by other graphic symbols; see log for descriptions.

Tillamook Estuary Partners
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

SOIL DESCRIPTION AND LOG KEY

May 2025 112880

SHANNON & WILSON

FIG. B1
Sheet 2 of 3

DRAFT

2013_SDRWG_CLASS2_112880.GPJ IRS SWOOTS, BRANYPOX.GLB SWWEN.DOT 5/5/25

GRADATION TERMS

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

CEMENTATION TERMS[1]

Weak	Crumbles or breaks with handling or slight finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

PLASTICITY[2]

DESCRIPTION	VISUAL-MANUAL CRITERIA	APPROX. PLASITICTY INDEX RANGE
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.	< 4%
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 to 10%
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 to 20%
High	It take considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	> 20%

ADDITIONAL TERMS

Mottled	Irregular patches of different colors.
Bioturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

PARTICLE ANGULARITY AND SHAPE TERMS[1]

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

ACRONYMS AND ABBREVIATIONS

ATD	At Time of Drilling
approx.	Approximate/Approximately
Diam.	Diameter
Elev.	Elevation
ft.	Feet
FeO	Iron Oxide
gal.	Gallons
Horiz.	Horizontal
HSA	Hollow Stem Auger
I.D.	Inside Diameter
in.	Inches
lbs.	Pounds
MgO	Magnesium Oxide
mm	Millimeter
MnO	Manganese Oxide
NA	Not Applicable or Not Available
NP	Nonplastic
O.D.	Outside Diameter
OW	Observation Well
pcf	Pounds per Cubic Foot
PID	Photo-Ionization Detector
PMT	Pressuremeter Test
ppm	Parts per Million
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
rpm	Rotations per Minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
q_u	Unconfined Compressive Strength
VWP	Vibrating Wire Piezometer
Vert.	Vertical
WOH	Weight of Hammer
WOR	Weight of Rods
Wt.	Weight

STRUCTURE TERMS[1]

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

[1]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.
[2]Adapted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

Tillamook Estuary Partners
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

SOIL DESCRIPTION AND LOG KEY

May 2025 112880

SHANNON & WILSON

FIG. B1
Sheet 3 of 3

2013_SD.PRNG CLASS3 112880 B1 SW2013L ENVIRYPDX.CLB SWANSLGDT 5/5/25

BASED ON INTERNATIONAL SOCIETY FOR ROCK MECHANICS (ISRM) ROCK CLASSIFICATION METHODS[a]

FABRIC TERMS

SEDIMENTARY ROCKS

MASSIVE - Rock without significant structure

BEDDED - Regular layering from sedimentation

FISSILE - Tendency to break along laminations

METAMORPHIC ROCKS

FOLIATED - Parallel arrangement or distribution of minerals

SCHISTOSE - Parallel arrangement of tabular minerals giving a planar fissility

GNEISSOSE - Segregation of minerals into bands

CLEAVAGE - Tendency to split along secondary, planar textures or structures

STRENGTH

GRADE	DESCRIPTION	FIELD IDENTIFICATION	APPROXIMATE RANGE OF UNIAXIAL COMPRESSIVE STRENGTH (MPa)	(psi)
R0	Extremely Weak Rock	Indented by thumbnail	0.25 to 1	36 to 145
R1	Very Weak Rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1 to 5	145 to 700
R2	Weak Rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5 to 25	700 to 3,600
R3	Medium Strong Rock	Cannot be scraped or peeled by a pocket knife, specimen can be fractured with single firm blow of geological hammer	25 to 50	3,600 to 7,200
R4	Strong Rock	Specimen requires more than one blow of geological hammer to fracture it	50 to 100	7,200 to 14,500
R5	Very Strong Rock	Specimen requires many blows of geological hammer to fracture it	100 to 250	14,500 to 36,250
R6	Extremely Strong Rock	Specimen can only be chipped with geological hammer	>250	>36,250

VESCULARITY

Slightly Vesicular	1 to 10%
Moderately Vesicular	10 to 30%
Highly Vesicular	30 to 50%
Scoriaceous	>50%

JOINT ROUGHNESS

SMALL SCALE	INTERMEDIATE SCALE
Rough	Stepped
Smooth	Undulating
Slickensided	Planar

WEATHERING

TERM	DESCRIPTION
Fresh	No visible signs of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.
Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and somewhat weaker than in its fresh condition.
Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
Highly Weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely Weathered	All rock is decomposed and/or disintegrated to soil. The original mass is still largely intact.
Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

DISCONTINUITY TERMS

FRACTURE - Collective term for any natural break excluding shears, shear zones, and faults

JOINT (JT) - Planar break with little or no displacement

FOLIATION JOINT (FJ) or BEDDING JOINT (BJ) - Joint along foliation or bedding

INCIPIENT JOINT (IJ) or INCIPIENT FRACTURE (IF) - Joint or fracture not evident until wetted and dried; breaks along existing surface

RANDOM FRACTURE (RF) - Natural, very irregular fracture that does not belong to a set

BEDDING PLANE SEPARATION or PARTING - A separation along bedding after extraction from stress relief or slaking

FRACTURE ZONE (FZ) - Planar zone of broken rock without gouge

MECHANICAL BREAK (MB) - Breaks due to drilling or handling; drilling break (DB), hammer break (HB)

SHEAR (SH) - Surface of differential movement evident by presence of slickensides, striations, or polishing

SHEAR ZONE (SZ) - Zone of gouge and rock fragments bounded by planar shear surfaces

FAULT (FT) - Shear zone of significant extent; differentiation from shear zone may be site-specific

STRUCTURE SPACING TERMS

STRATIGRAPHIC	SPACING	DISCONTINUITY *
Extremely Thick	> 20 ft. (> 6 m)	Extremely Wide
Very Thick	6 to 20 ft. (2 to 6 m)	Very Wide
Thick	2 to 6 ft. (0.6 to 2 m)	Wide
Medium	8 to 24 in. (0.2 to 0.6 m)	Moderate
Thin	2.5 to 8 in. (60 to 200 mm)	Close
Very Thin	1 to 2.5 in. (20 to 60 mm)	Very Close
Laminated: Thickly	0.25 to 1 in. (6 to 20 mm)	Extremely Close
Laminated: Thinly	<0.25 in. (<6 mm)	Extremely Close

* Refers to apparent spacing along core axis unless measured orthogonal to discontinuity; should then report for each set

[a] *Reference: Brown, E.T., ed., 1981, Rock Characterization Testing and Monitoring ISRM Suggested methods. New York, International Society for Rock Mechanics (ISRM).*

Tillamook Estuary Partners
Sitka Sedge Tidal Wetlands Restoration
Tillamook County, Oregon

ROCK CLASSIFICATION AND LOG KEY

May 2025 112880

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B2

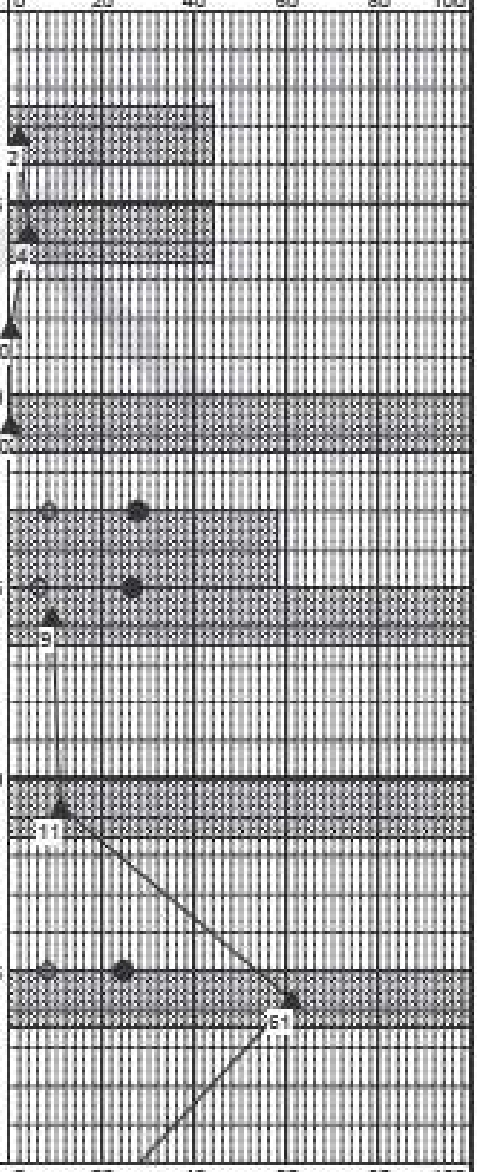
Filename: C:\S & W Tech Forms\Rock Classification - LogKey.dwg Layout: Layout1 Date: 05-12-2008 Login: Seth Sonnier

Total Depth:	76.5 ft.	Northing:	~ 601,333 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,095 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-WF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)
Gravel Pavement	12.5 2.0
Very loose to loose, light brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coarse, subangular gravel; fine to medium sand; low plasticity fines; micaceous; slight iron oxide staining. **EMBANKMENT FILL**	
Very soft, dark gray, *Silty Sand (SM)*; wet; fine to coarse sand; low plasticity fines; trace organics; shell fragments; micaceous.	4.6 10.0
Loose to very dense, gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace fine, subrounded gravel; fine to medium sand; nonplastic fines; micaceous. **FLUVIAL AND ESTUARIAN DEPOSITS**	2.0 12.5

Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



S-1 S-2 S-3 S-4 S-5 S-6 S-7 S-8

CONTINUED NEXT SHEET

LEGEND
- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-1

June 2025 112880

SHANNON & WILSON

FIG. B3
Sheet 1 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHS2013.LIBRARY.PDX.GDT 6/1/629 Log: SCF Rev: SCS Typ: SCF

Total Depth:	78.5 ft.	Northing:	~ 601,333 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,095 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.) | Symbol | Samples | Ground Water | Depth, ft

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*

Continued:

Loose to very dense, gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace fine, subrounded gravel; fine to medium sand; nonplastic fines; micaceous.

-16.0 / 32.5

FLUVIAL AND ESTUARIAN DEPOSITS

Gray, *Elastic Silt (MH)*; wet; fine to medium sand; low plasticity; micaceous.

-21.0 / 35.5

Harder drilling action at 35.5 feet.

Dense, red-brown mottled, *Silty Gravel with Sand (GM)*; moist; fine to coarse, subangular to subrounded; fine to coarse sand; low plasticity fines; distubed texture; heavy iron oxide staining.

-26.5 / 40.0

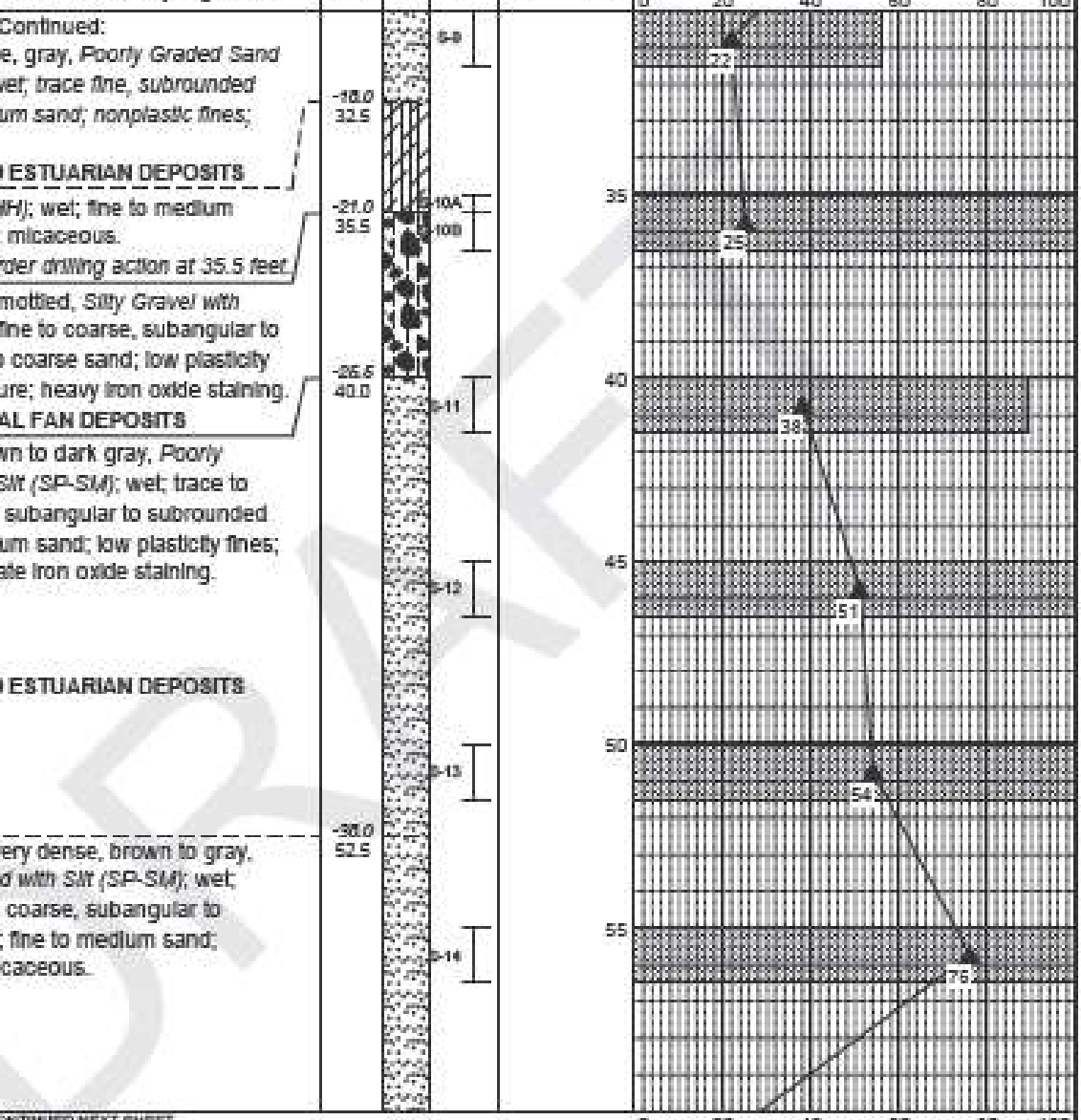
ALLUVIAL FAN DEPOSITS

Dense, Yellow-brown to dark gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coarse, subangular to subrounded gravel; fine to medium sand; low plasticity fines; micaceous; moderate iron oxide staining.

FLUVIAL AND ESTUARIAN DEPOSITS

-36.0 / 52.5

Medium dense to very dense, brown to gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coarse, subangular to subrounded gravel; fine to medium sand; nonplastic fines; micaceous.



CONTINUED NEXT SHEET

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-1

June 2025 | 112880

SHANNON & WILSON | FIG. B3 Sheet 2 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHS2013 LIBRARY.GLB SHANNWIL_PDX.GDT 6/1/25
Log: SCF Rev: SCS Typ: SCF

Total Depth:	76.5 ft.	Northing:	~ 601,333 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,095 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.) | Symbol | Samples | Ground Water Depth, ft.

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*

Continued:

Medium dense to very dense, brown to gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coarse, subangular to subrounded gravel; fine to medium sand; nonplastic fines; micaceous.

Gravel interbeds inferred from drilling action.

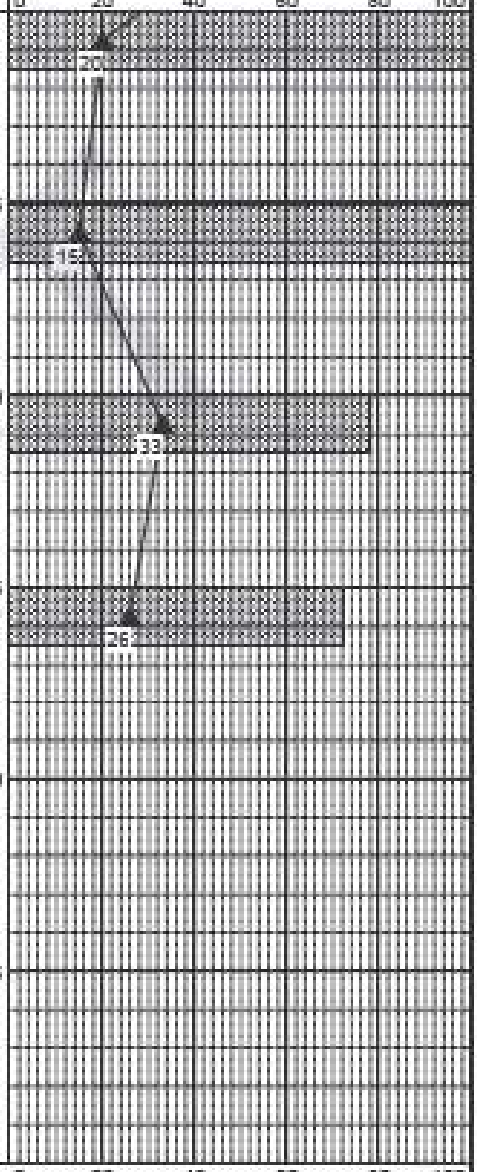
FLUVIAL AND ESTUARIAN DEPOSITS

-63.0 / 67.5

Medium dense to dense, gray to gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; low plasticity fines; stratified with interbeds of *Silty Sand (SM)*; micaceous.

-62.0 / 76.5

Completed: March 31, 2025



LEGEND

- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-1

June 2025 | 112880

SHANNON & WILSON | FIG. B3 Sheet 3 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHS2013 LIBRARY.FDO.GDT 6/1/629 Log: SCF Rev: SCS Typ: SCF

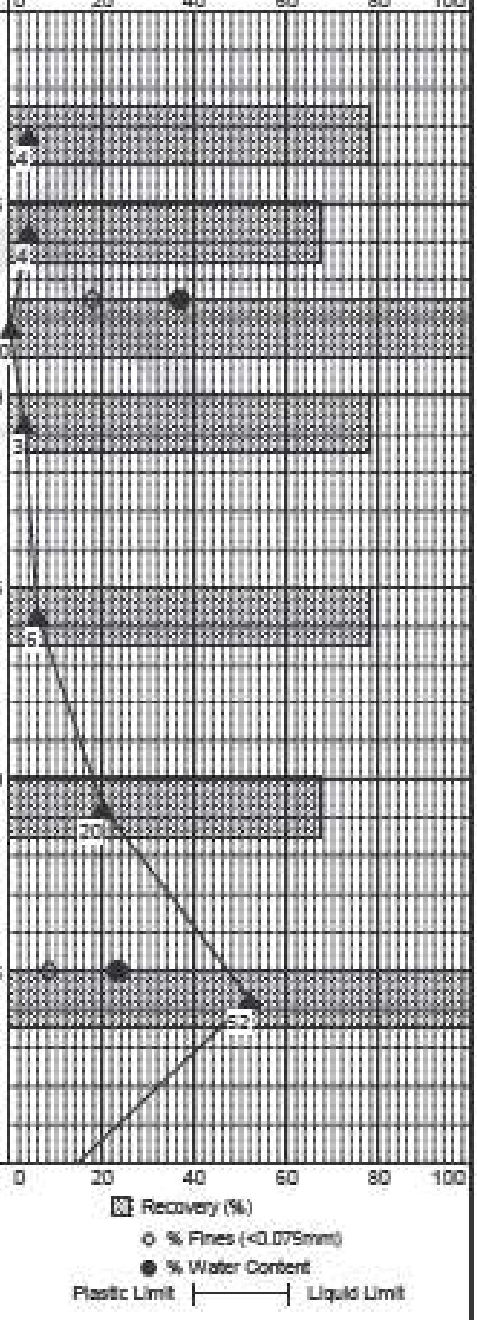
Total Depth:	66.5 ft.	Northing:	~ 601,319 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 14 ft.	Easting:	~ 7,312,217 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-WF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. / Depth (ft.)
Gravel Pavement	
Very loose to loose, brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium; low plasticity fines; micaceous; pockets of *Silty Sand (SM)*. **EMBANKMENT FILL**	11.5 / 2.5
Very loose, gray, *Silty Sand (SM)*; wet; fine to medium; nonplastic fines; micaceous.	6.6 / 7.5
Very loose to loose, gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; micaceous. **FLUVIAL AND ESTUARIAN DEPOSITS**	4.5 / 9.5
Medium dense to very dense, gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coarse, subangular to subrounded gravel; fine to medium sand; nonplastic fines; stratified with interbeds of *Poorly Graded Gravel with Silt and Sand (GP-GM)*; micaceous.	-3.6 / 17.5
Loose, gray to brown, *Silty Sand (SM)*; wet; trace to few, fine to coarse, subangular to subrounded gravel; fine to medium sand.	-13.6 / 27.5

CONTINUED NEXT SHEET

Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND
- Sample Not Recovered
- Standard Penetration Test
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-2

June 2025 | 112880

SHANNON & WILSON | FIG. B4 Sheet 1 of 3

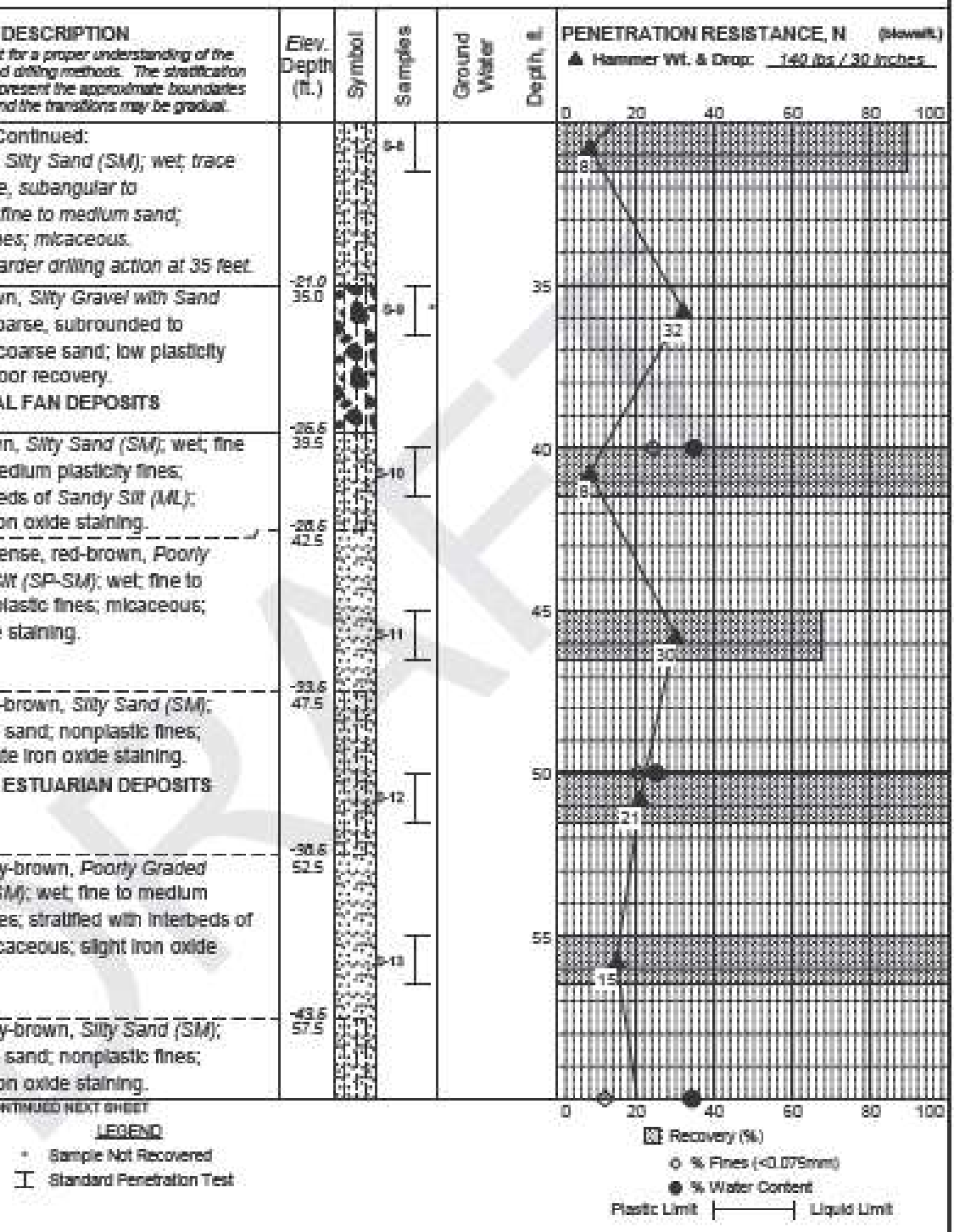
REV 3

MASTER_LOG_E 112880.GPJ SHN2013.LIBRARY.PDX.LOG GINT 6/1/629 Log: SCF Rev: SCS Typ: SCF

Total Depth:	66.5 ft.	Northing:	~ 601,319 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 14 ft.	Easting:	~ 7,312,217 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-WF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)
Continued: Loose, gray-brown, *Silty Sand (SM)*; wet; *trace to few, fine to coarse, subangular to subrounded gravel; fine to medium sand; medium plasticity fines; micaceous.* *Harder drilling action at 35 feet.*	
Dense, gray to brown, *Silty Gravel with Sand (GM)*; wet; fine to coarse, subrounded to subangular; fine to coarse sand; low plasticity fines; micaceous; poor recovery. **ALLUVIAL FAN DEPOSITS**	-21.0 35.0
Loose, gray to brown, *Silty Sand (SM)*; wet; fine to medium sand; medium plasticity fines; stratified with interbeds of *Sandy Silt (ML)*; micaceous; slight iron oxide staining.	-26.5 33.5
Medium dense to dense, red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; micaceous; moderate iron oxide staining.	-28.6 42.5
Medium dense, red-brown, *Silty Sand (SM)*; wet; fine to medium sand; nonplastic fines; micaceous; moderate iron oxide staining. **FLUVIAL AND ESTUARIAN DEPOSITS**	-33.6 47.5
Medium dense, gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; stratified with interbeds of *Silty Sand (SM)*; micaceous; slight iron oxide staining.	-38.6 52.5
Medium dense, gray-brown, *Silty Sand (SM)*; wet; fine to medium sand; nonplastic fines; micaceous; slight iron oxide staining.	-43.6 57.5

CONTINUED NEXT SHEET



LEGEND

* Sample Not Recovered

I Standard Penetration Test

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit ⊢—⊣ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-2

June 2025 112880

SHANNON & WILSON

FIG. B4
Sheet 2 of 3

REV 3

MASTER_LOG_E 112880.GPJ SHN2013.LIBRARY.PDX.LOG GINT 6/1/629
Log: SCF Rev: SCS Typ: SCF
DRAFT

Total Depth:	88.5 ft.	Northing:	~ 601,319 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 14 ft.	Easting:	~ 7,312,217 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)	Symbol	Samples	Ground Water Depth, ft.	PENETRATION RESISTANCE, N (blows/ft.) ▲ Hammer Wt. & Drop: 140 lbs / 30 Inches 0 20 40 60 80 100
Continued: Medium dense, gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; micaceous; slight iron oxide staining.	-48.6 62.5		S-14		21
Medium dense, gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; stratified with interbeds of *Silty Sand (SM)*; micaceous; slight iron oxide staining.			S-15		65 21
FLUVIAL AND ESTUARIAN DEPOSITS			S-16		70 25
			S-17		75 24
Dense, gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; micaceous; moderate iron oxide staining.	-68.6 82.5				80
			S-18		85 33
Completed: April 1, 2025	-72.6 86.5				

0 20 40 60 80 100

LEGEND
- Sample Not Recovered
- Standard Penetration Test
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-2

June 2025 112880

SHANNON & WILSON

FIG. B4
Sheet 3 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHS2013 LIBRARY/PDX_GLB SHANNWIL_PDX.GDT 6/1/629
Log: SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	– 600,349 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	– 15 ft.	Easting:	– 7,312,476 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	–	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-WF	Offset:	–	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION	Elev. Depth (ft.)	Symbol	Samples	Ground Water Depth, ft.	PENETRATION RESISTANCE, N (blows/ft.)
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.					▲ Hammer Wt. & Drop: 140 lbs / 30 inches
Asphalt: 4 inches thick	15.0				
Base Aggregate	0.3 13.8				
Very soft, dark brown, *Organic Soil (OH)*; wet; trace, fine, subrounded to rounded gravel; trace to few, fine to coarse sand; high plasticity; few to little organics.	1.5		S-1		
MARSH DEPOSITS			S-2		
	7.8				
Very loose, yellow-brown, *Silty Gravel (GM)*; wet; fine to coarse, subrounded to rounded gravel; fine to coarse sand; low to medium plasticity fines; micaceous.	7.5 6.8		S-3		
	6.5		S-4		
Gray, *Elastic Silt (MH)*; wet; trace, fine to coarse sand; medium to high plasticity; slight iron oxide staining.	4.3 11.0				
Loose to medium dense, gray to brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; micaceous.			S-5		
FLUVIAL AND ESTUARIAN DEPOSITS					
	4.7				
Medium dense to very dense, brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace, fine to coarse, subrounded to rounded gravel; fine to coarse sand; medium plasticity fines; micaceous.	20.0		S-6		
			S-7		

CONTINUED NEXT SHEET

LEGEND

I Standard Penetration Test

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit ⊢—⊣ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-3

June 2025 112880

SHANNON & WILSON

FIG. B5
Sheet 1 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2013.LIBRARY.FOLIO.GLB SHANNAL_PDX.GDT 6/1/629
Log: SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	~ 600,349 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,476 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-WF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.)

Symbol

Samples

Ground Water

Depth, ft.

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

0 20 40 60 80 100

Continued:

Medium dense to very dense, brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse sand; medium plasticity fines; micaceous.

S-8

S-9

S-10

S-11

S-12

S-13

35 40 45 50 55

56 57 32 16 20 13

FLUVIAL AND ESTUARIAN DEPOSITS

-37.2
52.5

Dense, brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine, subrounded to rounded gravel; fine to coarse sand; nonplastic fines; micaceous; heavy iron oxide staining.

CONTINUED NEXT SHEET

0 20 40 60 80 100

LEGEND

I Standard Penetration Test

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit ⊢—⊣ Liquid Limit

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-3

June 2025 112880

SHANNON & WILSON

FIG. B5
Sheet 2 of 3

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHG2013.LIBRARY.FOLIO.GLB SHANNAL_PDX.GDT 6/1/629
Log: SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	~ 600,349 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,476 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-WF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION	Elev. Depth (ft.)	Symbol	Samples	Ground Water Depth, ft.	PENETRATION RESISTANCE, N (blows/ft.)
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.					▲ Hammer Wt. & Drop: 140 lbs / 30 inches
Continued: Dense, brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few fine, subroudned to rounded gravel; fine to coarse; nonplastic fines; micaceous; heavy iron oxide staining.			S-14		39
FLUVIAL AND ESTUARIAN DEPOSITS	-48.7 64.0				
Harder drilling action at 64 feet.			S-15		50/1st 3"
SILTSTONE/SANDSTONE: very weak to weak (R1-R2); moderately weathered; joint structure indiscernible in SPT sample.					
ALSEA FORMATION	-54.0 70.1		S-16		50/1st 1"
Completed: March 24, 2025					

0 20 40 60 80 100

65 70 75 80 85

0 20 40 60 80 100

LEGEND

I Standard Penetration Test

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit ⊢—⊣ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-3

June 2025 112880

SHANNON & WILSON

FIG. B5
Sheet 3 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHANNWIL_PDX.GDT 6/1/25
Log: SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	~ 600,357 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,606 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)	Symbol	Samples	Ground Water Depth, ft.	PENETRATION RESISTANCE, N (blows/ft.) ▲ Hammer Wt. & Drop: 140 lbs / 30 inches
Asphalt Concrete	14.7 0.3				
Base Aggregate	13.5 1.5				
Very soft, dark brown, *Organic Soil (OH)*; wet; fine to coarse sand; high plasticity; few to little organics. **MARSH DEPOSITS**			S-1 S-2 S-3		1 5 2 0
Very soft to soft, light gray to gray-brown, *Elastic Silt (MH)*; wet; trace fine sand; medium to high plasticity; micaceous.	6.0 10.0		S-4		10 2
Very loose, blue-gray, *Silty Sand (SM)*; wet; fine to medium sand; high plasticity fines; micaceous.	3.0 12.0		S-5		15 2
Medium dense, yellow-brown to red-brown, *Silty Sand with Gravel (SM)*; wet; fine to coarse, subangular to subrounded; gravel; fine to coarse sand; low plasticity fines; micaceous; slight iron oxide staining. **FLUVIAL AND ESTUARIAN DEPOSITS**	-1.3 16.3		S-6		20 21
Dense, yellow-brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse sand; nonplastic fines; heavy iron oxide staining.	-10.0 25.0		S-7		25 33
CONTINUED NEXT SHEET					0 20 40 60 80 100

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-4

June 2025 112880

SHANNON & WILSON

FIG. B6
Sheet 1 of 3

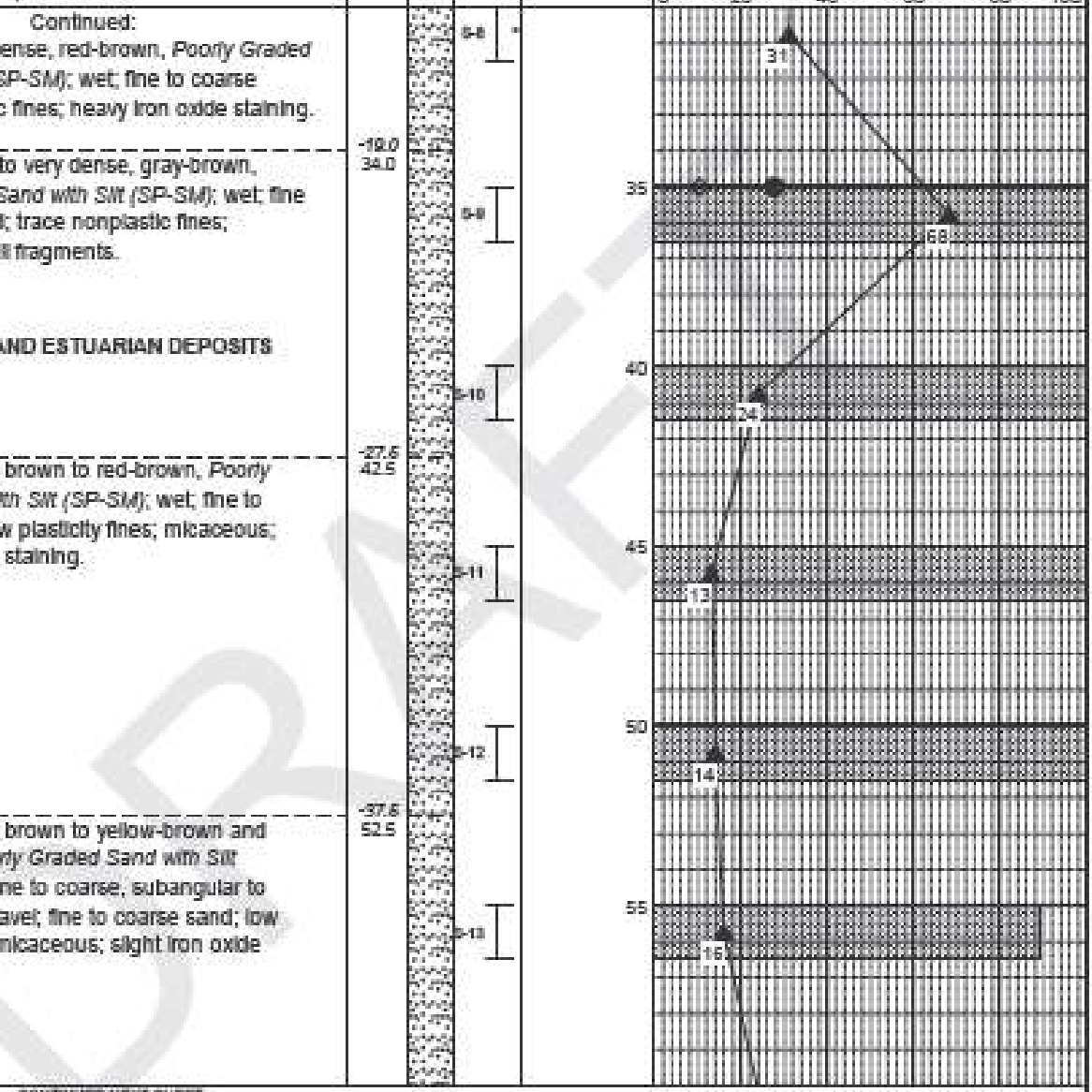
REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHANNWIL_PDX.GDT 6/1/25
Low SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	~ 600,357 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,606 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)
Continued: Dense to very dense, red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse sand; nonplastic fines; heavy iron oxide staining.	
Medium dense to very dense, gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; trace nonplastic fines; micaceous, shell fragments.	-19.0 34.0
FLUVIAL AND ESTUARIAN DEPOSITS	
Medium dense, brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse sand; low plasticity fines; micaceous; slight iron oxide staining.	-27.6 42.5
Medium dense, brown to yellow-brown and red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse, subangular to subrounded, gravel; fine to coarse sand; low plasticity fines; micaceous; slight iron oxide staining.	-37.6 52.5



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered

I Standard Penetration Test

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-4

June 2025 112880

SHANNON & WILSON

FIG. B6
Sheet 2 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2013.LIBRARY.FOLIO.GLB SHANNAL_FOLIO.GDT 6/1/829
Log: SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	~ 600,357 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,606 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-WF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.) | Symbol | Samples | Ground Water | Depth, ft

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

Continued:

Medium dense, red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse, subangular to subrounded, gravel; fine to coarse sand; low plasticity fines; micaceous; slight iron oxide staining.

~49.0 / 64.0

FLUVIAL AND ESTUARIAN DEPOSITS

Harder drilling action at 64 feet.

SILTSTONE/SANDSTONE: extremely weak to weak (R0-R2); moderately weathered; joint structure indiscernible in SPT sample.

ALSEA FORMATION

~56.1 / 70.1

Completed: March 25, 2025

S-14: 25

S-15: 50/3"

S-16: 50/1st 1"

0 20 40 60 80 100

65 70 75 80 85

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-4

June 2025 | 112880

SHANNON & WILSON | FIG. B6 Sheet 3 of 3

REV 3

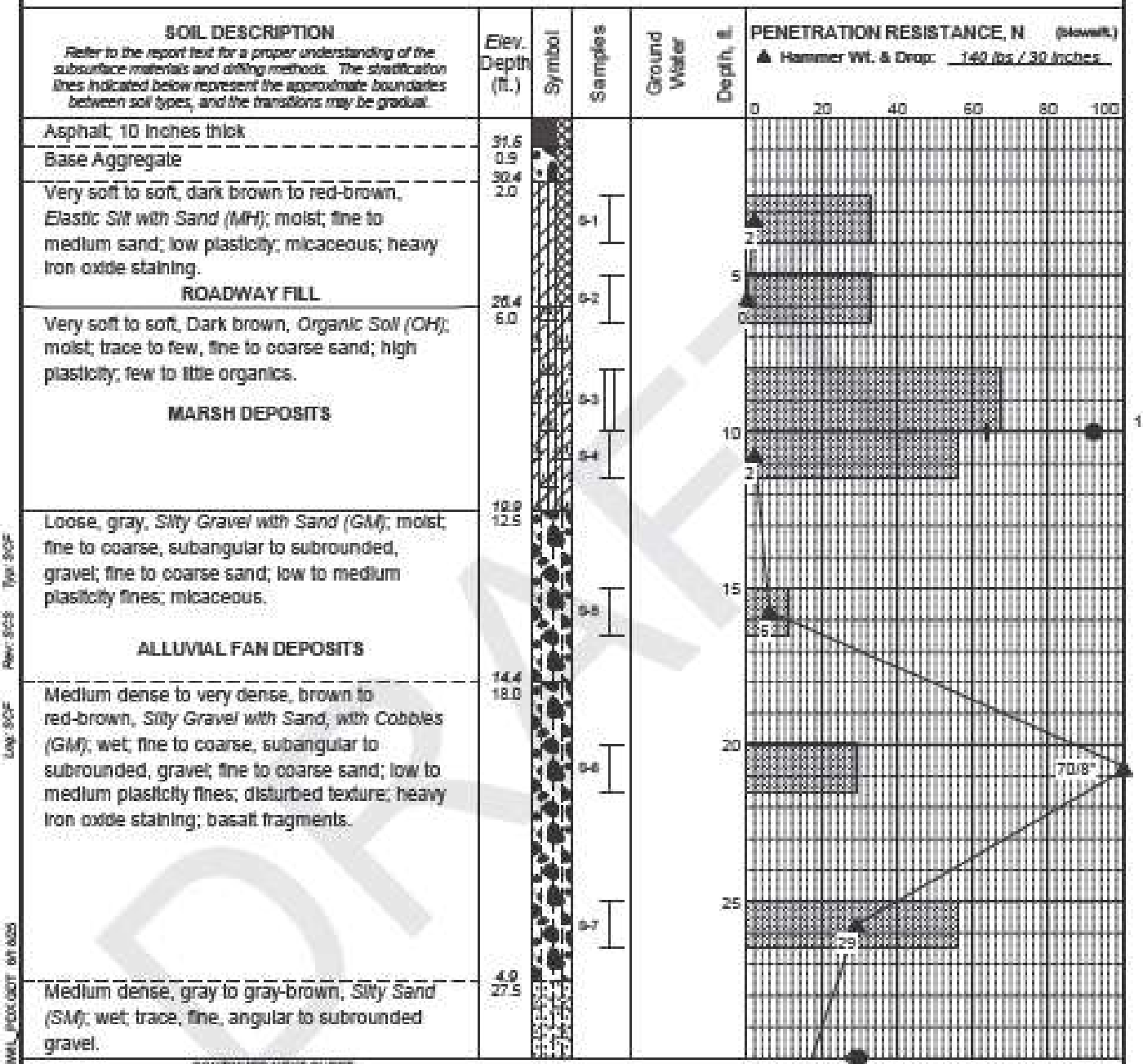
DRAFT

MASTER_LOG_E 112880.GPJ SHS2013.LIBRARY.ROLOGS SHANNAL_ROLOGDT 6/1/629
Log: SCF Rev: SCS Typ: SCF

Total Depth:	74.1 ft.	Northing:	– 699,629 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	– 32 ft.	Easting:	– 7,311,955 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	–	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	–	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)
Asphalt; 10 inches thick	31.6
Base Aggregate	0.9 30.4
Very soft to soft, dark brown to red-brown, *Elastic Silt with Sand (MH)*; moist; fine to medium sand; low plasticity; micaceous; heavy iron oxide staining. **ROADWAY FILL**	2.0 26.4
Very soft to soft, Dark brown, *Organic Soil (OH)*; moist; trace to few, fine to coarse sand; high plasticity; few to little organics. **MARSH DEPOSITS**	6.0
Loose, gray, *Silty Gravel with Sand (GM)*; moist; fine to coarse, subangular to subrounded, gravel; fine to coarse sand; low to medium plasticity fines; micaceous. **ALLUVIAL FAN DEPOSITS**	12.9 12.5
Medium dense to very dense, brown to red-brown, *Silty Gravel with Sand, with Cobbles (GM)*; wet; fine to coarse, subangular to subrounded, gravel; fine to coarse sand; low to medium plasticity fines; disturbed texture; heavy iron oxide staining; basalt fragments.	14.4 18.0
Medium dense, gray to gray-brown, *Silty Sand (SM)*; wet; trace, fine, angular to subrounded gravel.	4.9 27.5

CONTINUED NEXT SHEET



LEGEND

- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-5

June 2025 112880

SHANNON & WILSON

FIG. B7
Sheet 1 of 3

REV 3

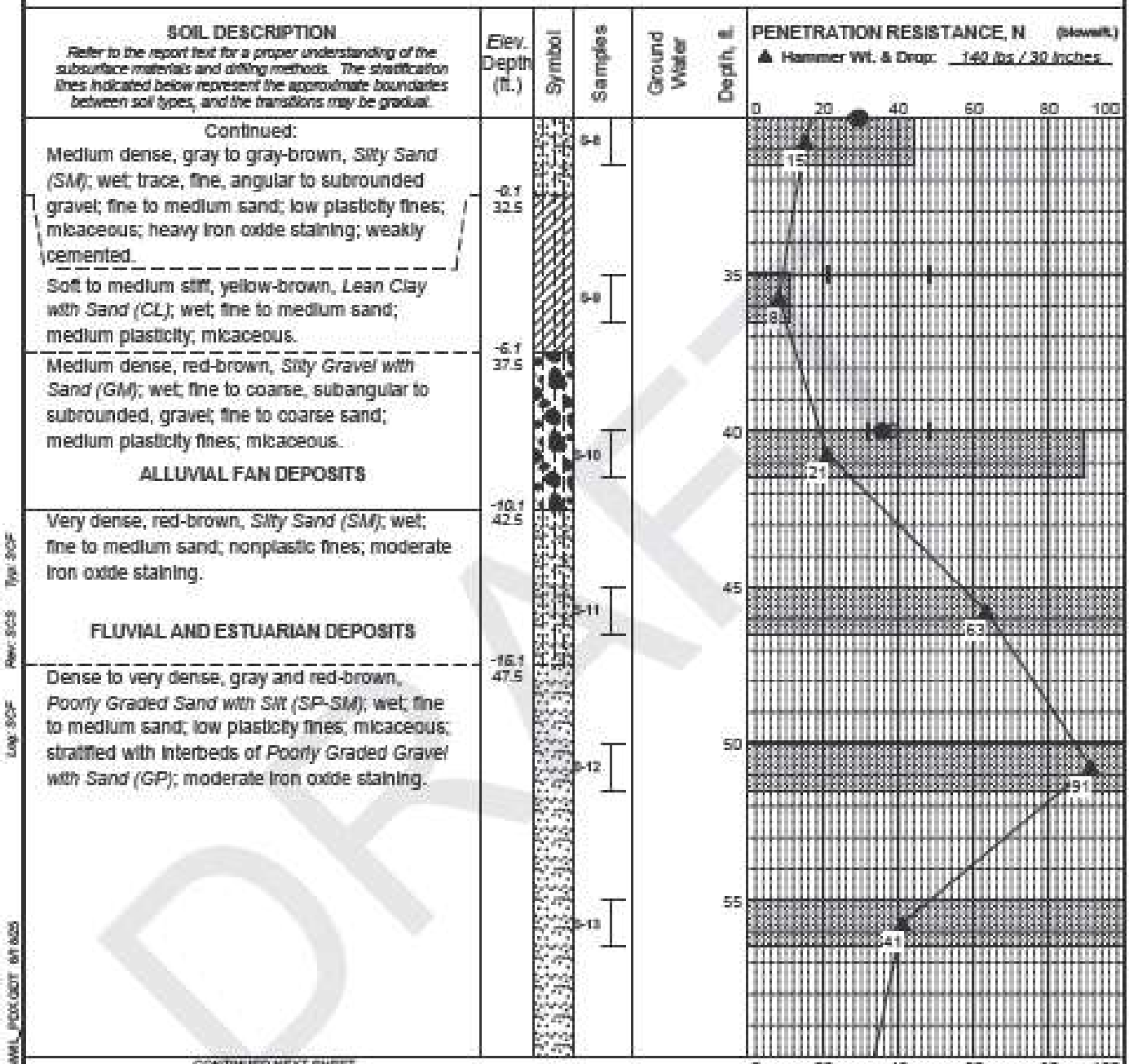
DRAFT

MASTER_LOG_E 112880.GPJ SHS2013.LIBRARY.PDX.GDT 6/1/629 Low SCF Rev. SCS Typ. SCF

Total Depth:	74.1 ft.	Northing:	– 699,629 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	– 32 ft.	Easting:	– 7,311,955 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	–	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	–	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)
Continued: Medium dense, gray to gray-brown, *Silty Sand (SM)*; wet; trace, fine, angular to subrounded gravel; fine to medium sand; low plasticity fines; micaceous; heavy iron oxide staining; weakly cemented.	-0.1 32.5
Soft to medium stiff, yellow-brown, *Lean Clay with Sand (CL)*; wet; fine to medium sand; medium plasticity; micaceous.	
Medium dense, red-brown, *Silty Gravel with Sand (GM)*; wet; fine to coarse, subangular to subrounded, gravel; fine to coarse sand; medium plasticity fines; micaceous. **ALLUVIAL FAN DEPOSITS**	-6.1 37.5
Very dense, red-brown, *Silty Sand (SM)*; wet; fine to medium sand; nonplastic fines; moderate iron oxide staining. **FLUVIAL AND ESTUARIAN DEPOSITS**	-10.1 42.5
Dense to very dense, gray and red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; low plasticity fines; micaceous; stratified with interbeds of *Poorly Graded Gravel with Sand (GP)*; moderate iron oxide staining.	-16.1 47.5

CONTINUED NEXT SHEET



LEGEND

- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-5

June 2025 112880

SHANNON & WILSON

FIG. B7
Sheet 2 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHS2013.LIBRARY.PDX.GDT 6/1/629 Log: SCF Rev: SCS Typ: SCF

Total Depth:	74.1 ft.	Northing:	~ 699,629 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 32 ft.	Easting:	~ 7,311,955 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.) | Symbol | Samples | Ground Water | Depth, ft

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*

Continued:

Dense to very dense, gray and red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium; low plasticity fines; micaceous; stratified with interbeds of *Poorly Graded Gravel with Sand (GP)*; moderate iron oxide staining.

FLUVIAL AND ESTUARIAN DEPOSITS

Harder drilling action at 67 feet.

-34.6 / 67.0

SILTSTONE/SANDSTONE: very weak to weak (R1-R2); slightly to moderately weathered; joint structure indiscernible in SPT sample.

ALSEA FORMATION

-41.7 / 74.1

Completed: March 28, 2025

S-14 32

S-15 40

S-16 50/1st 3"

S-17 50/1st 1"

65 70 75 80 85

0 20 40 60 80 100

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit ⊢—⊣ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-5

June 2025 112880

SHANNON & WILSON

FIG. B7
Sheet 3 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHS2013.LIBRARY.ROLOGS SHANNAL_ROLOGDT 6/1/629
Log: SCF Rev: SCS Typ: SCF

Total Depth:	90.1 ft.	Northing:	– 699,209 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	– 25 ft.	Easting:	– 7,311,451 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	–	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	–	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)	Symbol	Samples	Ground Water Depth, ft.	PENETRATION RESISTANCE, N (blows/ft.) ▲ Hammer Wt. & Drop: 140 lbs / 30 inches
Asphalt Concrete; 10 inches thick	24.1				
Base Aggregate	0.9				
Loose to medium dense, red-brown to gray-brown, *Silty Gravel with Sand (GM)*; wet; fine to coarse, subangular to subrounded gravel; fine to coarse sand; low plasticity fines. **ROADWAY FILL** *Lost drilling fluid circulation at approximately 5 feet. Hole sloughing from 5 to 9 feet. Skipped samples at 5.0 and 7.5 feet due to sloughing.*	22.5 2.5		S-1		
Stiff, gray to yellow-brown, *Elastic Silt with Sand (MH)*; wet; trace, fine to coarse, subangular to subrounded gravel; fine to medium sand; high plasticity; decomposed gravel clasts.	16.6 8.5		S-2		
Medium dense, red-brown mottled, *Silty Gravel with Sand (GM)*; moist; fine, subangular to subrounded gravel; fine to coarse sand; medium plasticity fines; micaceous; moderate iron oxide staining. **ALLUVIAL FAN DEPOSITS** *Softer drilling action at 18.5 feet.*	13.0 12.0		S-3 S-4		
Stiff, yellow-brown, *Elastic Silt with Sand (MH)*; wet; fine to coarse sand; medium plasticity; micaceous; slight iron oxide staining.	6.5 18.5		S-5		
Medium dense, red-brown to gray-brown, *Silty Sand with Gravel (SM)*; moist.	2.5 22.5		S-6 S-7		

CONTINUED NEXT SHEET

LEGEND
- * Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-6

June 2025 112880

SHANNON & WILSON

FIG. B8
Sheet 1 of 4

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHS2013 LIBRARY.FDO.GDT 6/1/629 Log: SCF Rev: SCS Typ: SCF

Total Depth:	90.1 ft.	Northing:	~ 699,209 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 25 ft.	Easting:	~ 7,311,461 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-WF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.) | Symbol | Samples | Ground Water Depth, ft.

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

0 20 40 60 80 100

Continued:

Medium dense, red-brown to gray-brown, *Silty Sand with Gravel (SM)*; moist; interbedded with gray-brown sandy SILT (ML); moist; medium plasticity fines; sand fine to medium; fine, subangular to subrounded gravel; fine to coarse; low to medium plasticity; micaceous; disturbed texture; heavy iron oxide staining.

-2.6 27.5

ALLUVIAL FAN DEPOSITS

Dense to very dense, gray-brown to red-brown, *Silty Sand (SM)*; wet; fine to medium sand; medium plasticity fines; micaceous.

-7.6 32.5

Medium dense to very dense, gray-brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; moist to wet; fine to coarse sand; nonplastic fines; micaceous; slight to moderate iron oxide staining.

FLUVIAL AND ESTUARIAN DEPOSITS

S-8 S-9 S-10 S-11

30 35 40 45

33 50 59 55

0 20 40 60 80 100

CONTINUED NEXT SHEET

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Silka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-6

June 2025 112880

SHANNON & WILSON

FIG. B8
Sheet 2 of 4

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2013.LIBRARY.PDX.GDT 6/1/629 Log: SCF Rev: SCS Typ: SCF

Total Depth:	90.1 ft.	Northing:	~ 699,209 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 25 ft.	Easting:	~ 7,311,461 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-WF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.) | Symbol | Samples | Ground Water | Depth, ft

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

0 20 40 60 80 100

Continued:

Medium dense to very dense, gray-brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse; nonplastic fines; micaceous; moderate iron oxide staining.

S-12 — 21

FLUVIAL AND ESTUARIAN DEPOSITS

S-13 — 22

55

Harder drilling action at 60 feet.

-35.0
60.0

S-14 — 50/4"

60

SILTSTONE: very weak to weak (R1-R2); slightly to moderately weathered; joint structure indiscernible in SPT sample.

ALSEA FORMATION

S-15 — 50/1st 2"

65

S-16 — 50/1st 1"

70

CONTINUED NEXT SHEET

0 20 40 60 80 100

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit ⊢—⊣ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-6

June 2025 112880

SHANNON & WILSON

FIG. B8
Sheet 3 of 4

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHS2013 LIBRARY.GLB SHANNWIL_PDX.GDT 6/1/629
Log: SCF Rev: SCS Typ: SCF

Total Depth:	90.1 ft.	Northing:	~ 699,209 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 25 ft.	Easting:	~ 7,311,461 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. / Depth (ft.) | Symbol | Samples | Ground Water | Depth, ft

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

0 20 40 60 80 100

Continued:

SILTSTONE: very weak to weak (R1-R2); slightly to moderately weathered; joint structure indiscernible in SPT sample.

ALSEA FORMATION

S-17 50/1st 2"

80

S-18 50/1st 2"

85

S-19 50/1st 2"

-65.1

90

S-20 50/1st 1"

90.1

Completed: March 27, 2025

95

0 20 40 60 80 100

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-6

June 2025 | 112880

SHANNON & WILSON | FIG. B8 Sheet 4 of 4

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHS2013 LIBRARY.GLB SHANNWL_PDX.GDT 6/1/25
Log: SCF Rev: SCS Typ: SCF

Total Depth:	41.5 ft.	Northing:	~ 698,700 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 11 ft.	Easting:	~ 7,311,100 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-WF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Asphalt Concrete; 9 inches thick — 9.0

Base Aggregate — 0.8 — 8.6 — 2.0

Very soft to soft, dark brown, *Organic Soil (OH)*; wet; trace, fine to medium sand; high plasticity; trace to few organics; stratified with interbeds of peat.

MARSH DEPOSITS

-0.6 — 11.2

Loose, gray, *Silty Sand (SM)*; wet; fine to medium sand; low plasticity fines; micaceous.

FLUVIAL AND ESTUARIAN DEPOSITS

-3.6 — 14.0

Loose, brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines.

-7.0 — 17.5

Medium dense, yellow-brown to brown, *Silty Sand (SM)*; wet; trace to few, fine to coarse, subrounded gravel; fine to coarse sand; nonplastic fines; micaceous; slight iron oxide staining.

-12.0 — 22.5

Medium dense, yellow-brown to brown, *Poorly Graded Sand with Silt (SP-SM)*; wet.

CONTINUED NEXT SHEET

Elev. Depth (ft.) | Symbol | Samples | Ground Water Depth, ft.

S-1 | S-2 | S-3 | S-4 | S-5 | S-6

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

0 20 40 60 80 100

5: 1, 2 — 10: 2, 2 — 15: 7 — 20: 21 — 183

0 20 40 60 80 100

LEGEND

- 3" O.D. Shelby Tube
- Standard Penetration Test
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit ⊢—⊣ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-7

June 2025 — 112880

SHANNON & WILSON

FIG. B9
Sheet 1 of 2

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2013.LIBRARY.FOX.GDT 6/1/629 Log: SCF Rev: SCS Typ: SCF

Total Depth:	41.5 ft.	Northing:	~ 698,700 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 11 ft.	Easting:	~ 7,311,100 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.) | Symbol | Samples | Ground Water Depth, ft.

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

Continued:

Medium dense, yellow-brown to brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coase, subrounded gravel; fine to coarse sand; nonplastic fines; micaceous; slight iron oxide staining; stratified with interbeds of *Poorly Graded Gravel with Silt and Sand (GP-GM)*.

FLUVIAL AND ESTUARIAN DEPOSITS

S-7

S-8

S-9

S-10

10

15

14

18

30

35

40

45

0 20 40 60 80 100

-31.0
41.5

Completed: March 25, 2025

LEGEND

3" O.D. Shelby Tube

Standard Penetration Test

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit ⊢—⊣ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-7

June 2025 112880

SHANNON & WILSON

FIG. B9
Sheet 2 of 2

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHS2013 LIBRARY/EOL.GLB SHANNWIL_PDX.GDT 6/1/25
Log: SCF Rev: SCS Typ: SCF

APPENDIX C: LABORATORY TEST RESULTS

Appendix C

Laboratory Test Results

CONTENTS

C.1 Introduction 1
C.2 Soil Testing 1
C.2.1 Moisture (Natural Water) Content 1
C.2.2 Atterberg Limits 2
C.2.3 Particle Size Analysis 2
C.2.4 Corrosivity Testing 2

Figure

Figure C1: Atterberg Limits Results
Figure C2: Particle Size Analysis

Attachment

Cooper Testing Laboratory, Inc., Corrosivity Tests Summary, dated September 5, 2024

DRAFT

APPENDIX C: LABORATORY TEST RESULTS

C.1 INTRODUCTION

Soil samples obtained during the field explorations were described and identified in the field in general accordance with the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), ASTM D2488. The samples were reviewed in the Shannon & Wilson laboratory. The physical characteristics of the samples were noted, and the field descriptions and identifications were modified where necessary in accordance with the terminology presented in Appendix A, Figure A1. Select samples were chosen for various laboratory tests. We refined our visual-manual soil descriptions and identifications based on the results of the laboratory tests, using elements of the Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), ASTM D2487. The refined descriptions and identifications were then incorporated into the Logs of Borings, presented in Appendix A. Note that ASTM D2487 was not followed in full because it requires that a suite of tests be performed to fully classify a single sample.

The soil testing program included natural moisture content, Atterberg Limits, particle size analyses, and corrosivity testing. Laboratory testing was performed by Shannon & Wilson in our in-house laboratory in Lake Oswego, Oregon, and Cooper Testing Laboratory of Palo Alto, California. All test procedures were performed in accordance with applicable ASTM International standards.

Analytical testing was performed on a composite sample of investigation-derived waste from the upper 10-feet of the borings for disposal. Analytical testing was performed by Pace Analytical National (Pace) of Mount Juliet, Tennessee in accordance with applicable Environmental Protection Agency (EPA) standards.

C.2 SOIL TESTING

C.2.1 Moisture (Natural Water) Content

Natural moisture content determinations were performed in accordance with ASTM D2216, on selected soil samples. The natural moisture content is a measure of the amount of moisture in the soil at the time of exploration. It is defined as the ratio of the weight of water to the dry weight of the soil, expressed as a percentage. The results of moisture content determinations are presented on the Logs of Borings in Appendix A.

C.2.2 Atterberg Limits

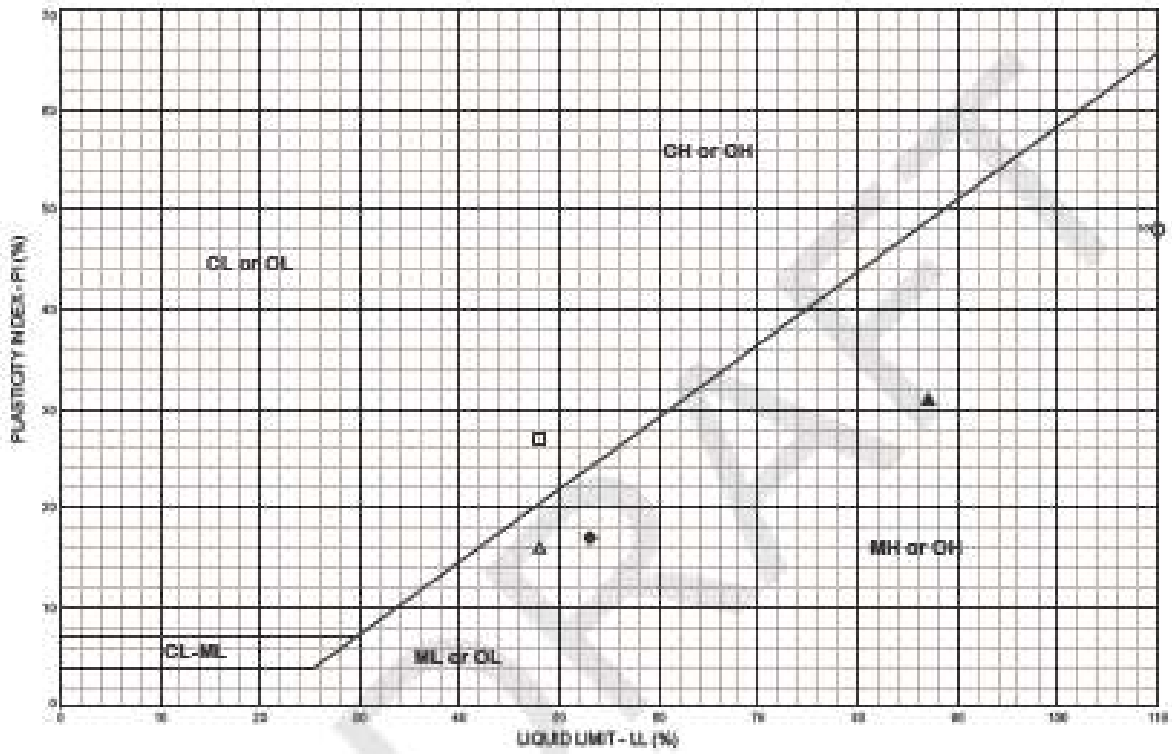
Atterberg Limits were determined for select samples in accordance with ASTM D4318. This analysis yields index parameters of the soil that are useful in soil identification, as well as in a number of analyses, including liquefaction analysis. An Atterberg Limits test determines a soil's liquid limit (LL) and plastic limit (PL). These are the maximum and minimum moisture contents at which the soil exhibits plastic behavior. A soil's plasticity index (PI) can be determined by subtracting PL from LL. The LL, PL, and PI of tested samples are presented in Figure C1, Atterberg Limits Results. The results of the Atterberg Limits tests are also presented in the Logs of Borings in Appendix A. For the purposes of soil description, Shannon & Wilson uses the term nonplastic to refer to soils with a PI less than 4, low plasticity for soils with a PI range of 4 to 10, medium plasticity for soils with a PI range of 10 to 20, and high plasticity for soils with a PI greater than 20. Oven-dried liquid limits were performed on samples B-3 S-2 at 5.0 feet, and B-7 S-3 at 7.5 feet.

C.2.3 Particle Size Analysis

Particle-size analyses were conducted on select samples to determine their grain-size distributions. Grain size distributions were determined in accordance with ASTM D6913 and D1140, as applicable. For the samples tested, a wet sieve analysis was performed to determine the percentage (by weight) of each sample passing the No. 200 (0.075 mm) was determined (ASTM D1140). Results of all particle-size analyses are presented in Figure C2, Grain Size Distribution. The percentage of each sample passing the No. 200 sieve is presented on the Logs of Borings in Appendix A.

C.2.4 Corrosivity Testing

Corrosivity testing was performed on a select specimen to evaluate the corrosivity potential of the soil at the site. The corrosivity test suite included redox potential, soil conductivity (resistivity), soluble chloride and sulfate concentration, sulfide tests, and soil pH.
A discussion of the corrosivity results is included in the main report text in Section 5.3.
Analytical testing was performed by Cooper Testing Laboratory and results of the corrosivity testing are attached to the end of this appendix.



NOTES

1) Atterberg limits tests were performed in general accordance with ASTM D4318 unless otherwise noted in the report.

2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

3) Plasticity adjectives used in sample descriptions correspond to plasticity index as follows:
- Nonplastic (NP) (< 4%)
- Low Plasticity (4 to 10%)
- Medium Plasticity (10 to 20%)
- High Plasticity (> 20%)

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL*	GROUP NAME*	LL %	PL %	PI %	NAT. W.C. %	FINES %
B-1, S-5	13.0	SP-SM	*Poorly Graded Sand with Silt*	NP	NP	NP	20	8
B-2, S-3	7.5	SM	*Silty Sand*	NP	NP	NP	37	18
▲ B-3, S-2	5.0	OH	*Organic Soil (Oven Dried LL = 47)*	87	56	31	83	
◆ B-4, S-4	10.0	MH	*Elastic Silt*	53	36	17	50	
○ B-5, S-4	10.0	OH	*Organic Soil with Gravel*	112	64	48	82	
□ B-5, S-9	35.0	CL	*Lean Clay with Sand*	48	21	27		
△ B-5, S-10	40.0	GM	*Silty Gravel with Sand*	48	32	16	36	38

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

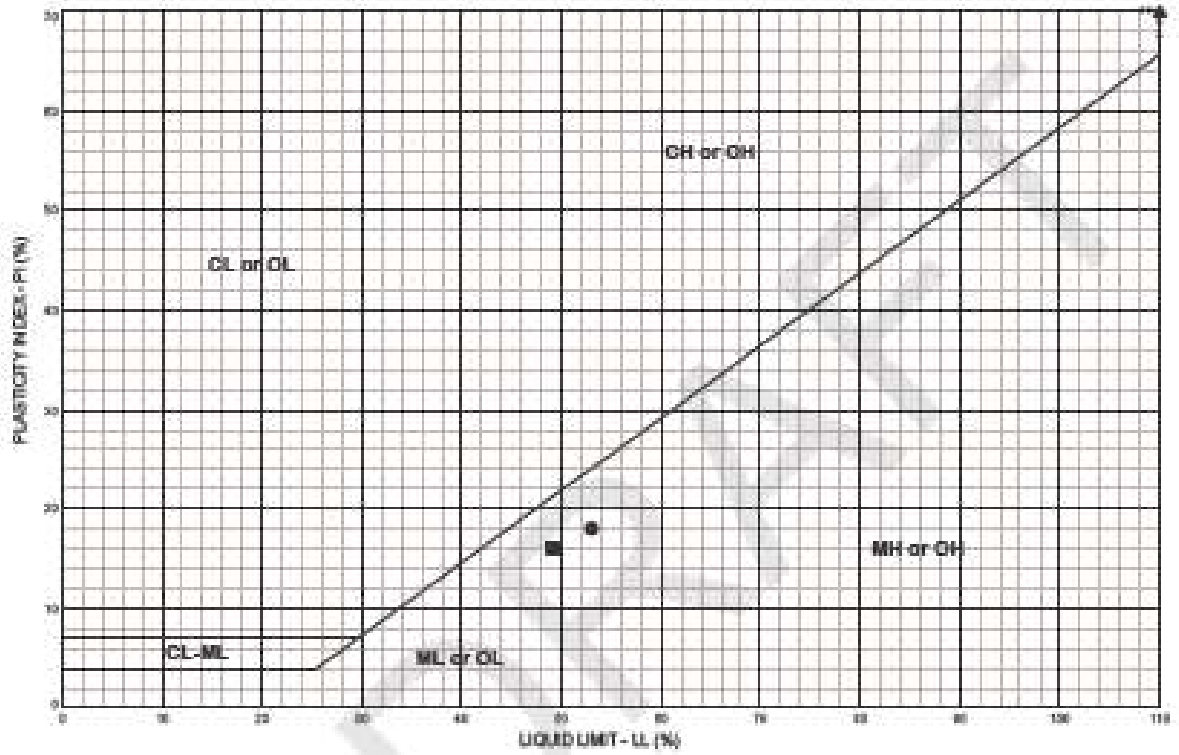
ATTERBERG LIMITS RESULTS

June 2025 112880

SHANNON & WILSON

FIG. C1
Sheet 1 of 2

FIG. C1



NOTES

1) Atterberg limits tests were performed in general accordance with ASTM D4318 unless otherwise noted in the report.

2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

3) Plasticity adjectives used in sample descriptions correspond to plasticity index as follows:
- Nonplastic (NP) (< 4%)
- Low Plasticity (4 to 10%)
- Medium Plasticity (10 to 20%)
- High Plasticity (> 20%)

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL*	GROUP NAME*	LL %	PL %	PI %	NAT. W.C. %	FINES %
● B-6, S-2	10.0	MH	*Elastic Silt with Sand*	53	35	18	30	
■ B-6, S-5	20.0	MH	*Elastic Silt with Sand*	49	33	16	42	38
▲ B-7, S-3	7.5	OH	*Organic Soil (Oven Dried LL = 58)*	153	83	80	109	69

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Tillamook County, Oregon

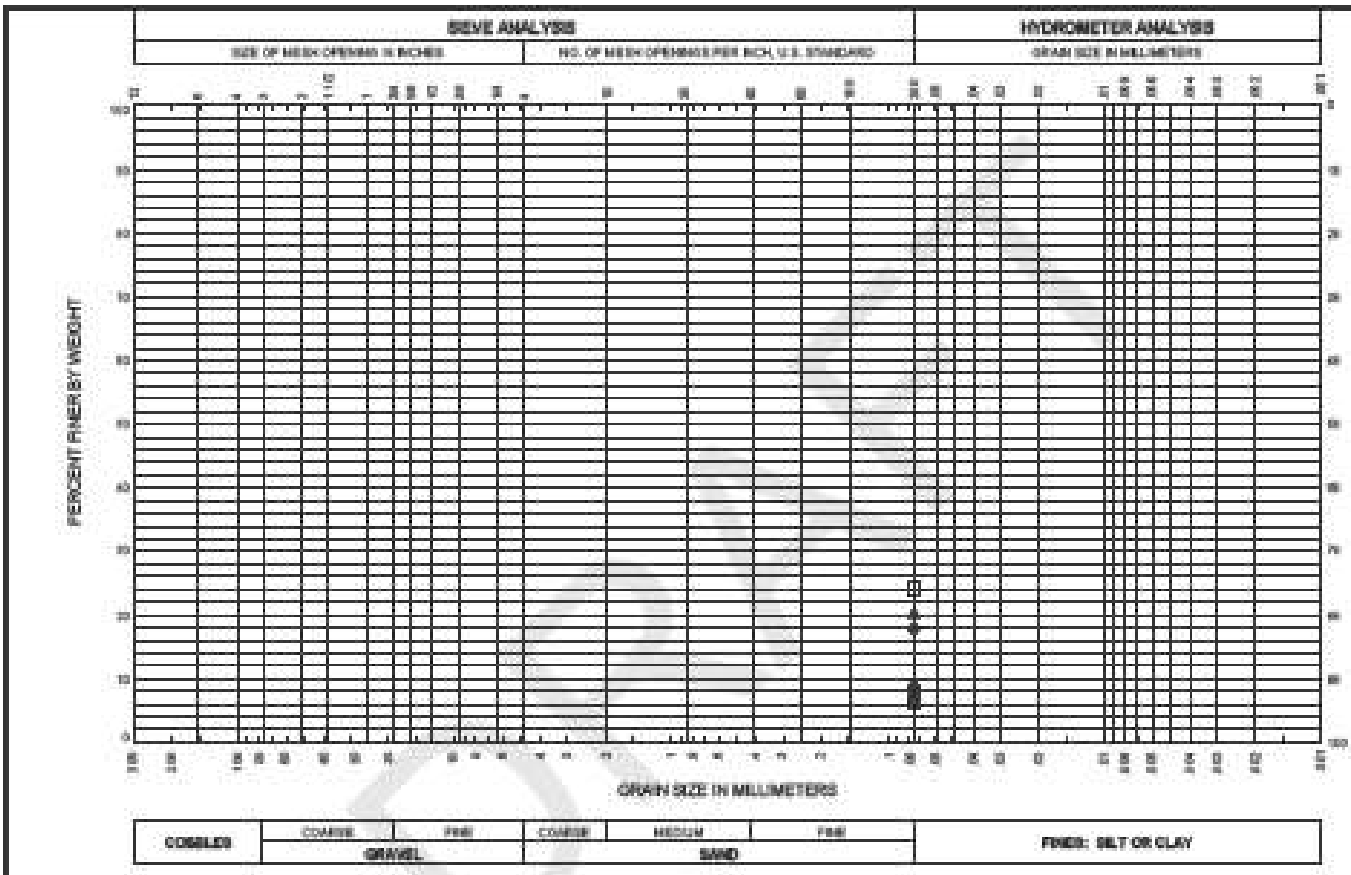
ATTERBERG LIMITS RESULTS

June 2025 112880

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FIG. C1
Sheet 2 of 2

FIG. C1



NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913. sieve with hydrometer analyses were performed in general accordance with ASTM D422, and unless otherwise noted in the report.
2) Group Name and Group Symbol were determined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[1]	GROUP NAME[2]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-1, S-5	13.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	6	29	
■ B-1, S-8	18.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	6	27	
▲ B-1, S-9	20.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	6	28	
◆ B-2, S-3	7.5	SM	*Silty Sand*	-	-	19	37	
○ B-2, S-7	28.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	6	23	
□ B-2, S-10	43.0	SM	*Silty Sand*	-	-	24	35	
△ B-2, S-12	53.0	SM	*Silty Sand*	-	-	30	28	

Silka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

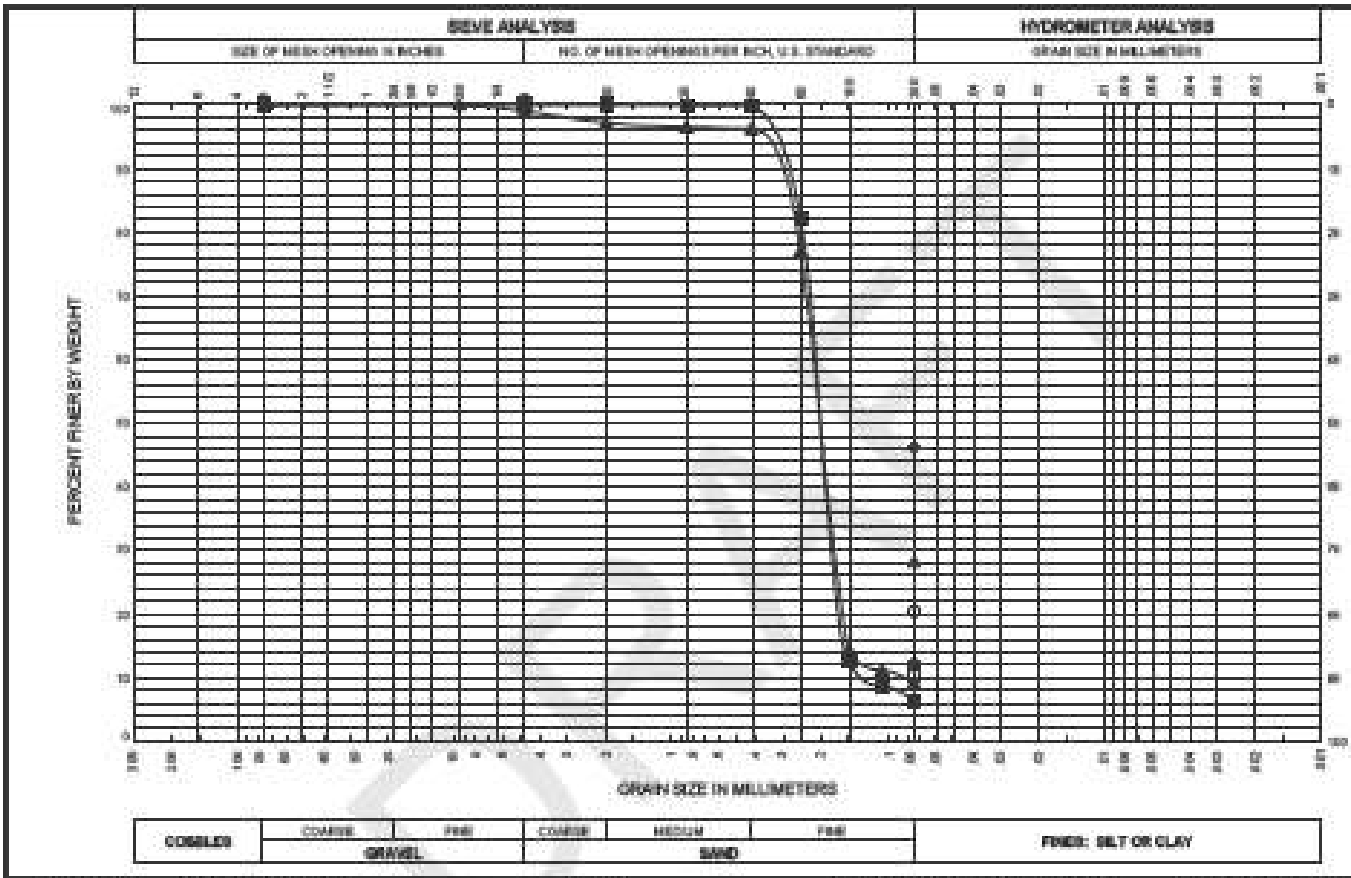
GRAIN SIZE DISTRIBUTION

June 2025 112880

SHANNON & WILSON

FIG. C2
Sheet 1 of 3

FIG. C2



NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913. sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount that pass #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Name and Group Symbol are in accordance with ASTM D2487 where appropriate laboratory tests are performed.

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[1]	GROUP NAME[2]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
B-2, S-14	63.0	SM	*Silty Sand*	-	-	13	34	
B-3, S-9	15.0	SP-SM	*Poorly Graded Sand with Silt*	0	94	6	27	
B-3, S-7	25.0	SP-SM	*Poorly Graded Sand with Silt*	1	90	9	27	
B-4, S-5	15.0	SM	*Silty Sand*	-	-	45	39	
B-4, S-8	33.0	SM	*Silty Sand with Gravel*	-	-	33	30	
B-4, S-9	35.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	10	29	
B-5, S-8	33.0	SM	*Silty Sand*	-	-	28	30	

Silka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

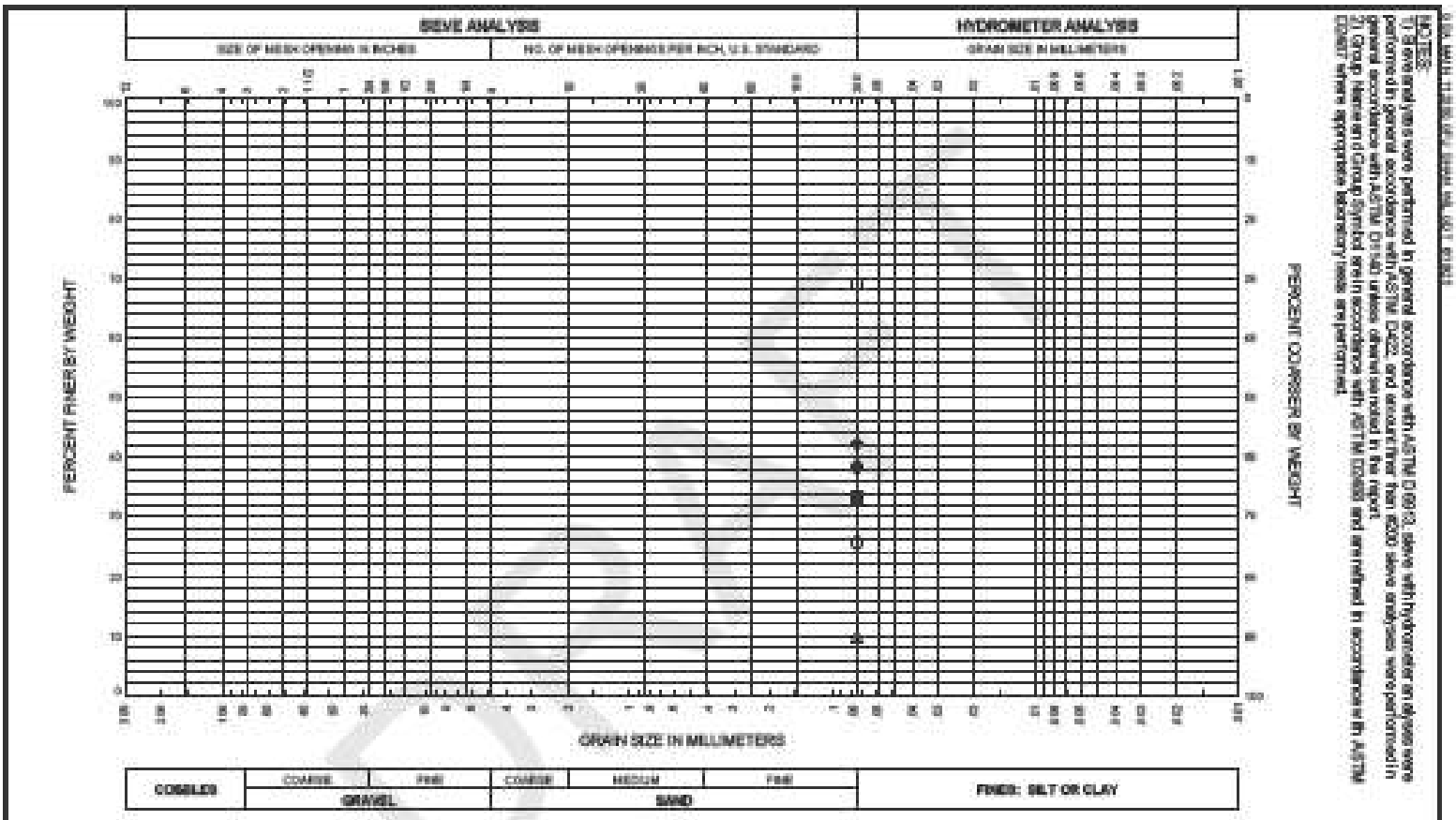
GRAIN SIZE DISTRIBUTION

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FIG. C2
Sheet 2 of 3

FIG. C2



NOTES:
1) Sieve analysis were performed in general accordance with ASTM D6913. sieve with hydrometer analysis were performed in general accordance with ASTM D422, and amount finer than #200 sieve analysis were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Name and Group Symbol were in accordance with ASTM D2487 where appropriate laboratory tests are performed.

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[1]	GROUP NAME[2]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-6, S-10	43.0	GM	*Silty Gravel with Sand*	-	-	39	36	
■ B-6, S-4	15.0	GM	*Silty Gravel with Sand*	-	-	33	38	
▲ B-6, S-5	23.0	MH	*Elastic Silt with Sand*	-	-	58	42	
◆ B-6, S-7	33.0	SM	*Silty Sand*	-	-	42	39	
○ B-6, S-8	38.0	SM	*Silty Sand*	-	-	38	33	
□ B-7, S-3	7.5	OH	*Organic Silt (Oven Dried LL = 58)*	-	-	69	109	
△ B-7, S-5	15.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	10	26	

Silka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

GRAIN SIZE DISTRIBUTION

June 2025 112880

SHANNON & WILSON

FIG. C2
Sheet 3 of 3

FIG. C2

COOPER

Corrosivity Tests Summary

CTL #: 240-229 | Date: 5/2/2025 | Tested By: PJ | Checked: PJ

Client: Shannon & Wilson | Project: Sitka Sedge | Proj. No: 112000

Remarks:

Sample Location or ID			Resistivity @ 15.6 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP (Redox)		Sulfide Qualitative by Lead Acetate Paper	Moisture At Test % ASTM D2216	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg Dry Wt.	mg/kg Dry Wt.	% Dry Wt.		E_H (mV) ASTM D200	At Test Temp °C			
Boring	Sample. No.	Depth, ft.	ASTM G57	Calc'd	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D50	ASTM D200	Temp °C	Acetate Paper	ASTM D2216	
B-1	-	10	183	-	-	3,847	2,070	0.2070	7.1	457	20	-	37.5	Gray Clayey SAND w/ organics
B-3	-	2.5	13,813	-	-	39	155	0.0155	6.6	305	20	-	61.4	Brown Sandy CLAY w/ Gravel

DRAFT

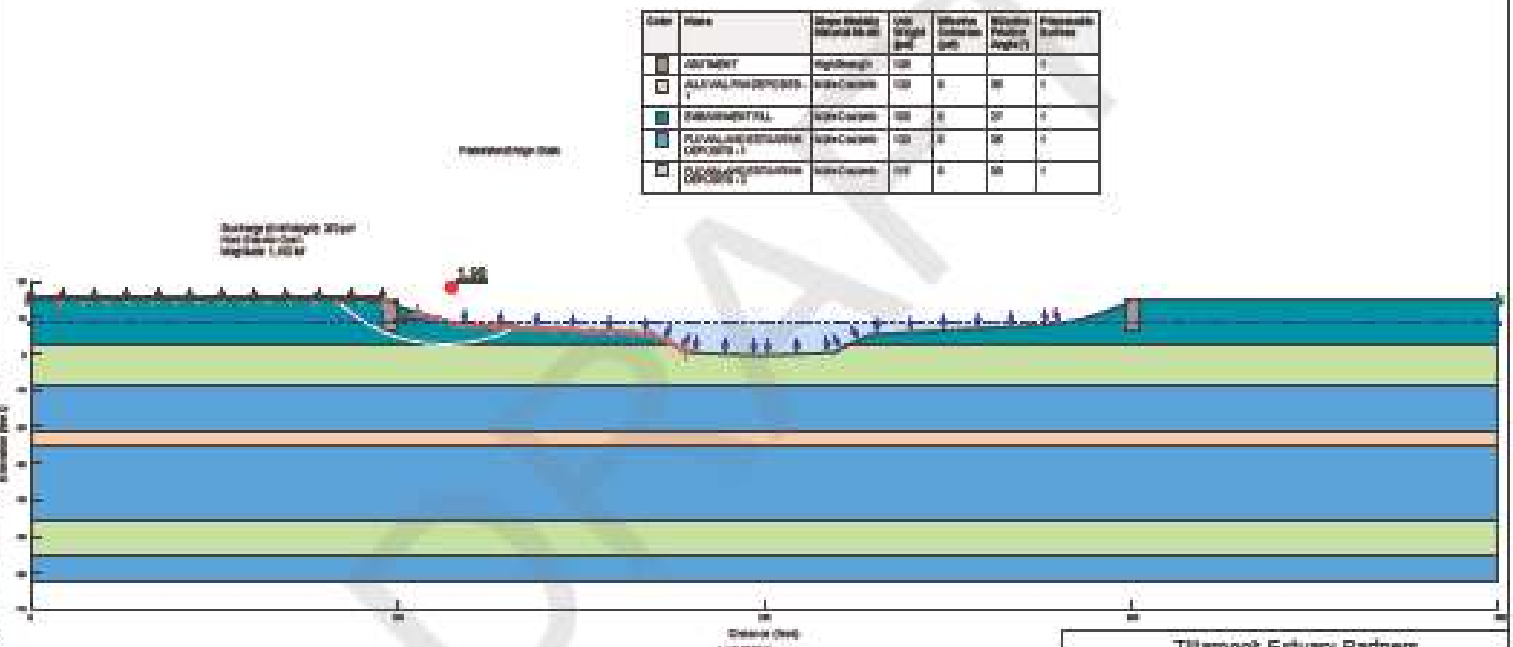
Appendix D

Slope Stability Analysis

Figures

Figure D1: Global Stability Analysis Pedestrian Bridge Static Conditions
Figure D2: Global Stability Analysis Pedestrian Bridge Seismic Conditions
Figure D3: Global Stability Analysis Pedestrian Bridge Post-Seismic Conditions
Figure D4: Global Stability Analysis Pedestrian Bridge Seismic Conditions with Additional Lateral Loads
Figure D5: Global Stability Analysis Pedestrian Bridge Post-Seismic Conditions with Additional Lateral Loads
Figure D6: Global Stability Analysis Reneke Creek Bridge Static Conditions
Figure D7: Global Stability Analysis Reneke Creek Bridge Seismic Conditions
Figure D8: Global Stability Analysis Reneke Creek Bridge Post-Seismic Conditions
Figure D9: Global Stability Analysis Beltz Creek Bridge With 4-foot Thick Riprap Static Conditions
Figure D10: Global Stability Analysis Beltz Creek Bridge With 4-foot Thick Riprap Seismic Conditions
Figure D11: Global Stability Analysis Beltz Creek Bridge With 4-foot Thick Riprap Post-Seismic Conditions
Figure D12: Global Stability Analysis Beltz Creek Bridge With 4-foot Thick Riprap Seismic Conditions with Additional Lateral Loads
Figure D13: Global Stability Analysis Existing Trailhead Embankment Static Conditions

DRAFT



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions. No seismic forces are included.
3. Analysis assumes 250 psf surcharge loading.

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Tillamook County, Oregon

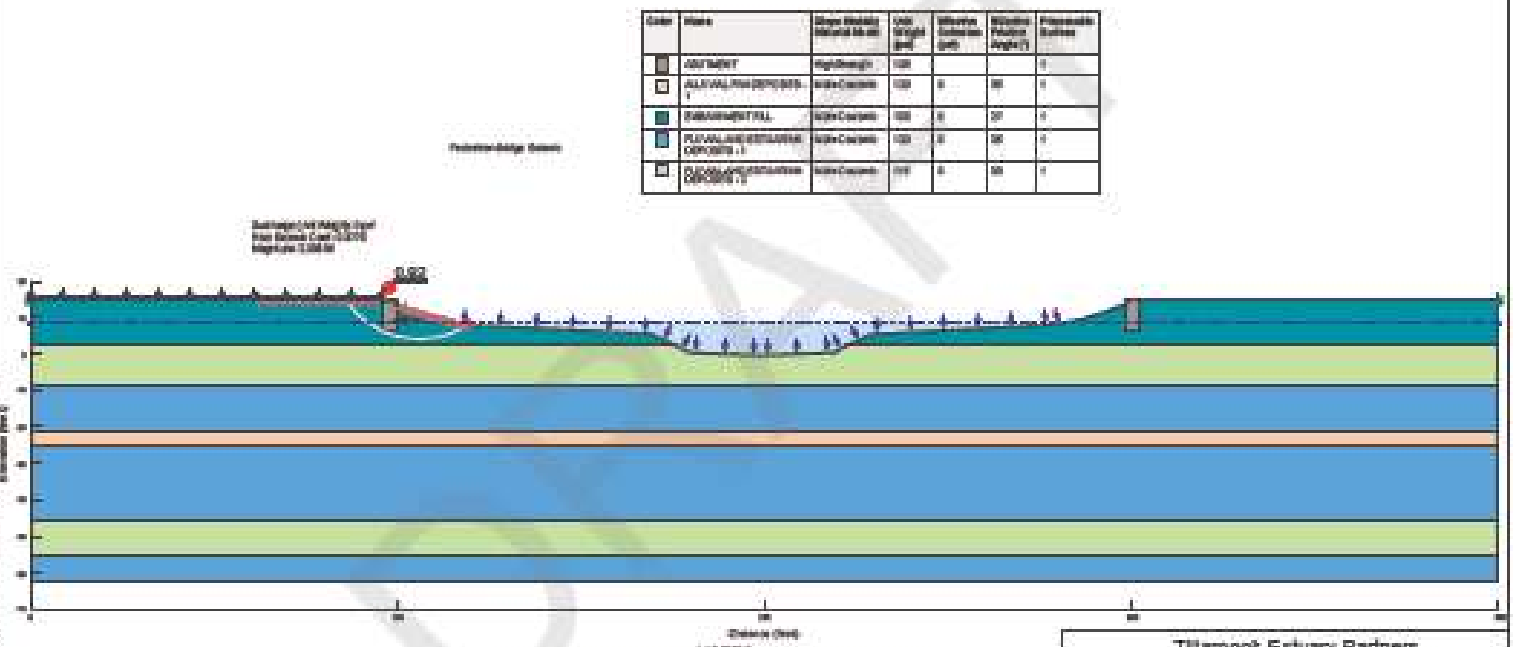
GLOBAL STABILITY ANALYSIS PEDESTRIAN BRIDGE STATIC CONDITIONS

October 2025 | 112880

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Geotechnical and Environmental Consultants

FIG. D1

DRAFT



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions using a horizontal seismic coefficient equal to one-half of PGA.
3. Analysis assumes no surcharge loading.

Tillamook Estuary Partners
Silka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

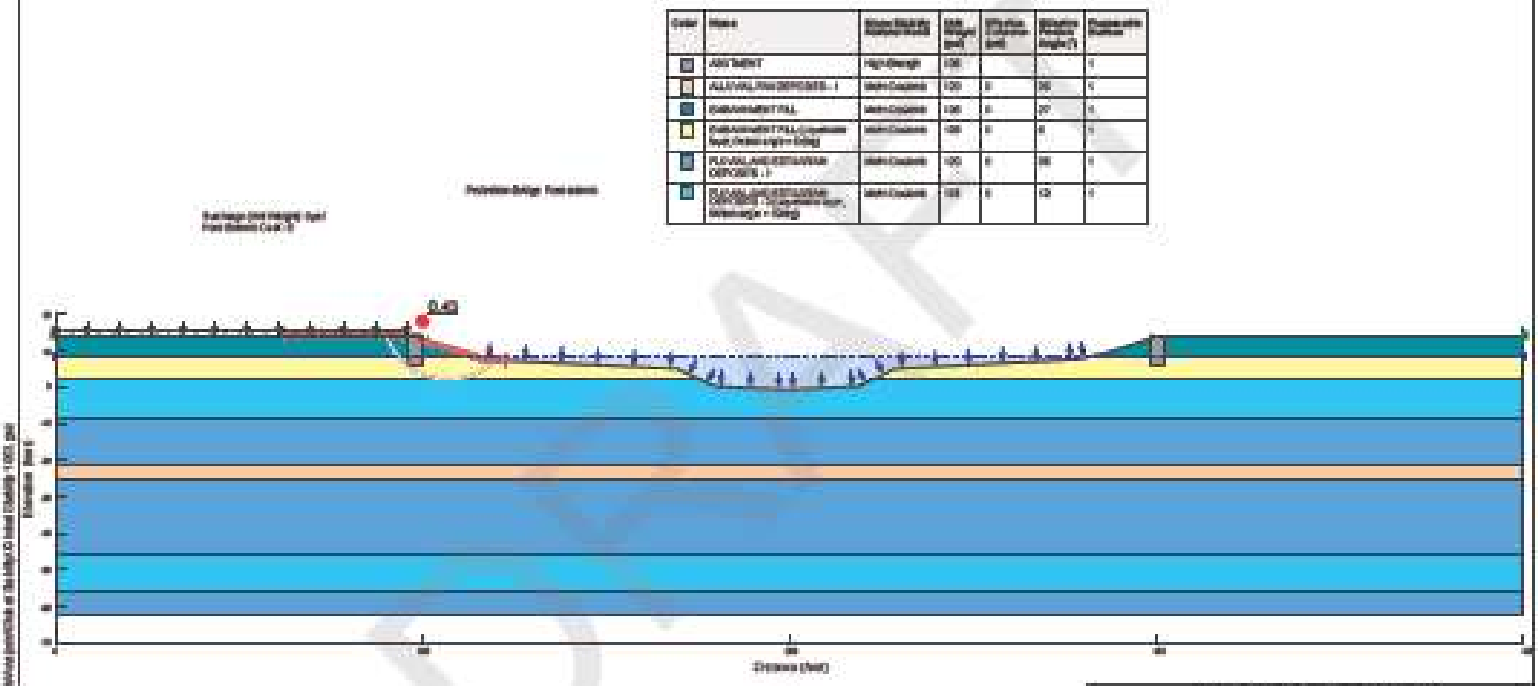
GLOBAL STABILITY ANALYSIS PEDESTRIAN BRIDGE SEISMIC CONDITIONS

October 2025 | 112880

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FIG. D2

DRAFT



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions. No seismic forces are included.
3. Analysis assumes no surcharge loading.

Tillamook Estuary Partners
Silka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

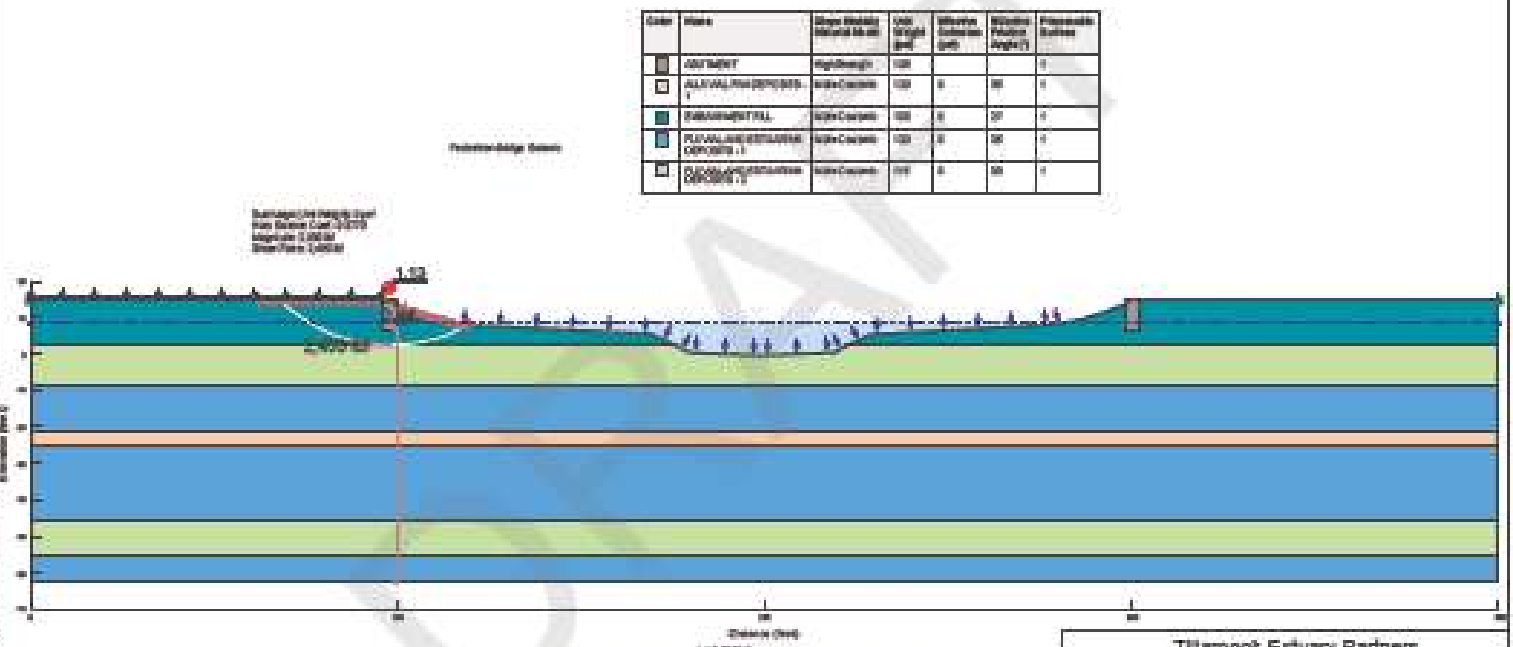
GLOBAL STABILITY ANALYSIS PEDESTRIAN BRIDGE POST-SEISMIC CONDITIONS

October 2025 112880

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FIG. D3

DRAFT



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions using a horizontal seismic coefficient equal to one-half of PGA.
3. Analysis assumes no surcharge loading.

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Tillamook County, Oregon

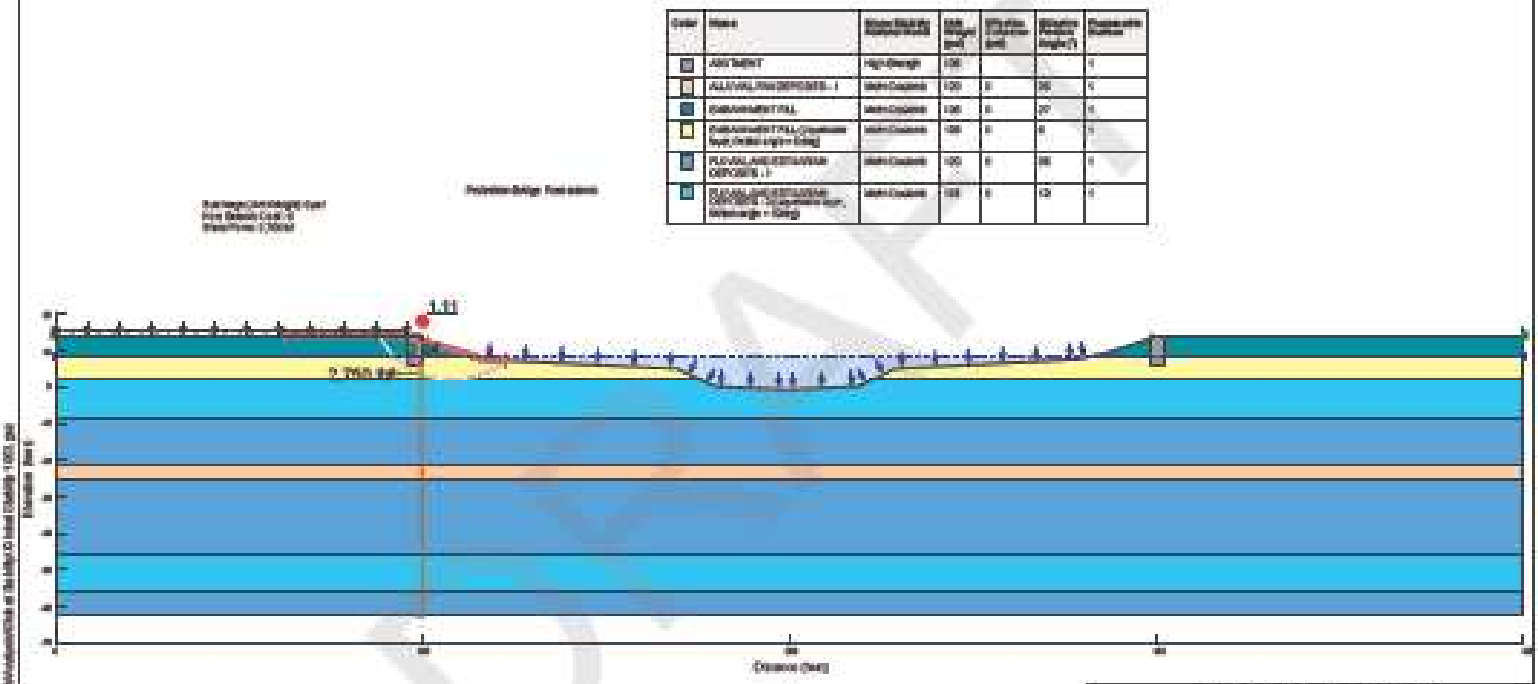
GLOBAL STABILITY ANALYSIS PEDESTRIAN BRIDGE SEISMIC W ADDITIONAL LOAD

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FIG. D4

DRAFT



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions. No seismic forces are included.
3. Analysis assumes no surcharge loading.

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Tillamook County, Oregon

GLOBAL STABILITY ANALYSIS PEDESTRIAN BRIDGE POST-SEISMIC W/ ADDITIONAL LOAD

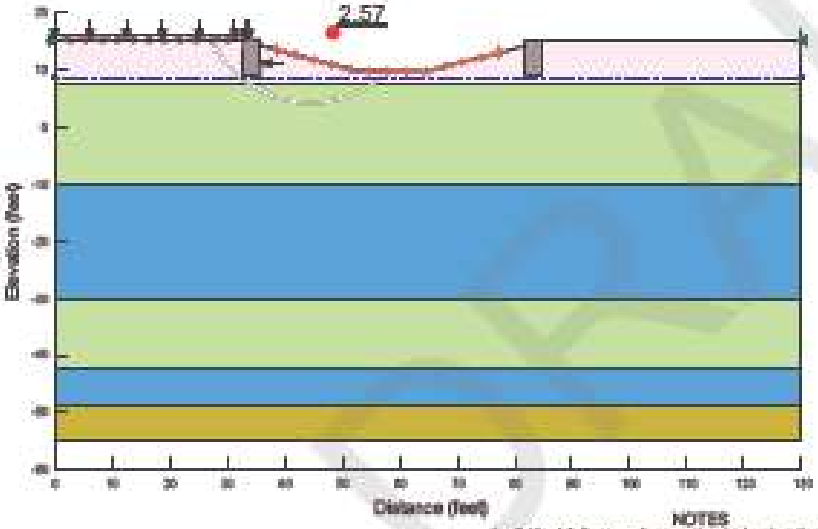
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FIG. D5

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	ABUTMENT	High Strength	135			1
	ALSEA FORMATION	Mohr-Coulomb	135	0	40	1
	FLUVIAL/AND ESTUARINE DEPOSITS - 1	Mohr-Coulomb	120	0	35	1
	FLUVIAL/AND ESTUARINE DEPOSITS - 2	Mohr-Coulomb	115	0	33	1
	MARSH DEPOSITS	Mohr-Coulomb	100	50	28	1

Reneke Bridge: Static
Kh=0 Seismic Coef.:
Magnitude: 6.48 Ibf
Surcharge (Unit Weight): 250 pcf



NOTES:
1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions. No seismic forces are included.
3. Analysis assumes 250 psf surcharge loading.

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GLOBAL STABILITY ANALYSIS RENEKE BRIDGE STATIC CONDITIONS

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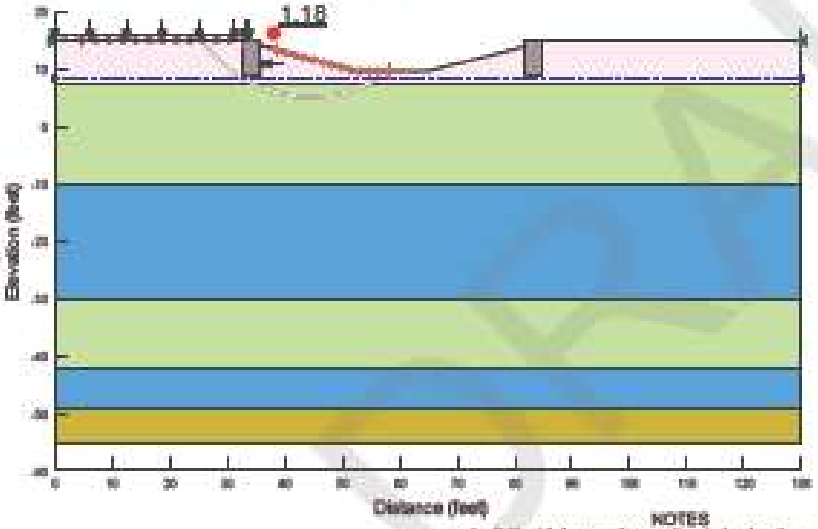
FIG. D6

DRAFT

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	ABUTMENT	High Strength	135			1
	ALSEA FORMATION	Mohr-Coulomb	135	0	40	1
	FLUVIAL AND ESTUARINE DEPOSITS - 1	Mohr-Coulomb	120	0	35	1
	FLUVIAL AND ESTUARINE DEPOSITS - 2	Mohr-Coulomb	115	0	33	1
	MARSH DEPOSITS	Mohr-Coulomb	100	50	28	1

Reneke Bridge: Seismic

Horz Seismic Coef.: 0.3775
Magnitude: 1.314 Bf
Surcharge (Unit Weight): 0 pcf



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions using a horizontal seismic coefficient equal to one-half of PGA.
3. Analysis assumes no surcharge loading.

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GLOBAL STABILITY ANALYSIS RENEKE BRIDGE SEISMIC CONDITIONS

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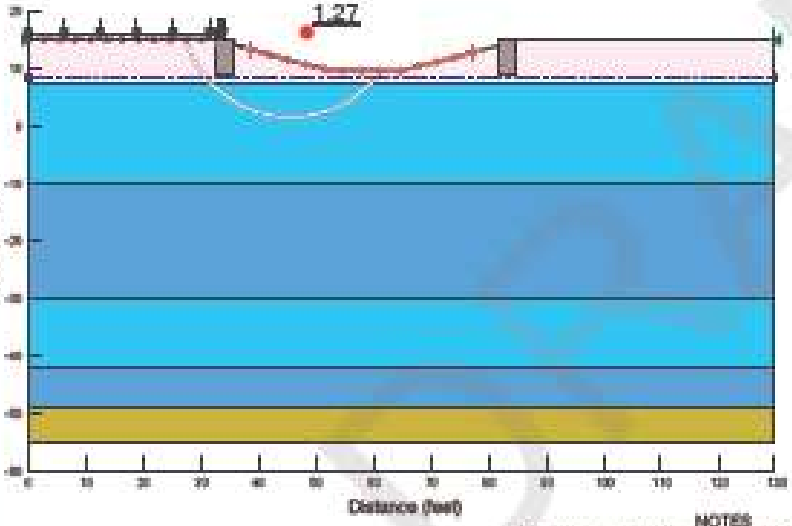
FIG. D7

DRAFT

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	ABUTMENT	High Strength	135			1
	ALSEA FORMATION	Mohr-Coulomb	130	0	40	1
	FLUVIAL/ESTUARINE DEPOSITS - 1	Mohr-Coulomb	120	0	35	1
	FLUVIAL/ESTUARINE DEPOSITS - 2 (Liquefiable layer, friction angle = 10deg)	Mohr-Coulomb	115	0	10	1
	MARSH DEPOSITS	Mohr-Coulomb	103	50	28	1

Reneke Bridge: Post-seismic

Horz Seismic Coef.: 0
Surcharge (Unit Weight): 0 pcf



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions. No seismic forces are included.
3. Analysis assumes no surcharge loading.

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GLOBAL STABILITY ANALYSIS RENEKE BRIDGE POST-SEISMIC CONDITIONS

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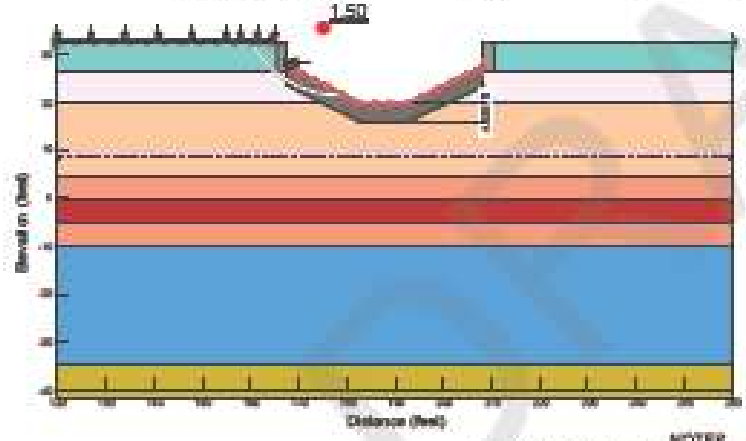
FIG. D8

DRAFT

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	ABUTMENT	High Strength	120			1
	ALLUVIAL FAN DEPOSITS - 1	Mohr-Coulomb	120	0	35	1
	ALLUVIAL FAN DEPOSITS - 2	Mohr-Coulomb	115	0	30	1
	ALLUVIAL FAN DEPOSITS - 3	Mohr-Coulomb	120	0	36	1
	ALSEA FORMATION	Mohr-Coulomb	135	0	40	1
	FLUVIAL AND ESTUARINE DEPOSITS - 1	Mohr-Coulomb	120	0	30	1
	MARSH DEPOSITS	Mohr-Coulomb	100	50	26	1
	RIPRAP	Mohr-Coulomb	140	0	42	1
	ROADWAY FILL	Mohr-Coulomb	130	50	28	1

Beltz Bridge - Static

Magnitude: 650 lbf
Hold Sides? Cost: 0
Surcharge (Unit Weight): 250 psf



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions. No seismic forces are included.
3. Analysis assumes 250 psf surcharge loading.

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GLOBAL STABILITY ANALYSIS BELTZ BRIDGE STATIC CONDITIONS

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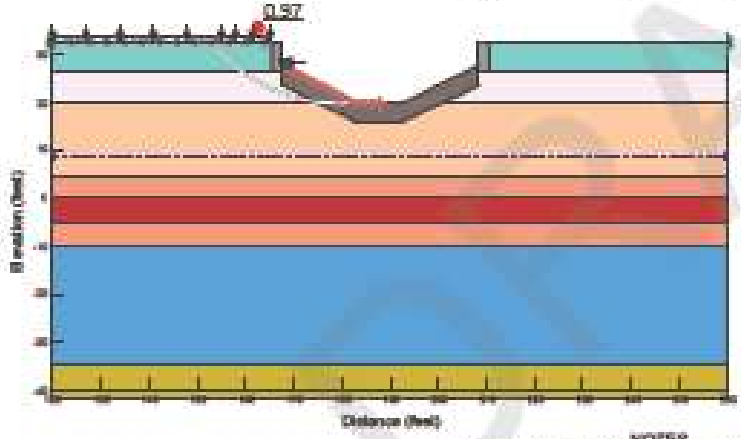
FIG. D9

DRAFT

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	ABUTMENT	High Strength	120			1
	ALLUVIAL FAN DEPOSIT 0 - 1	Mohr-Coulomb	120	0	35	1
	ALLUVIAL FAN DEPOSIT 0 - 2	Mohr-Coulomb	115	0	30	1
	ALLUVIAL FAN DEPOSIT 0 - 3	Mohr-Coulomb	130	0	38	1
	BEDAFORMATION	Mohr-Coulomb	135	0	40	1
	FLUVIAL AND ESTUARINE DEPOSIT 0 - 1	Mohr-Coulomb	120	0	35	1
	MARSH DEPOSITS	Mohr-Coulomb	100	50	26	1
	RIP RAP	Mohr-Coulomb	140	0	42	1
	ROADWAY FILL	Mohr-Coulomb	130	50	28	1

Beltz Bridge: Seismic

Magnitude: 1,200 lbf
Horiz Seismic Coef.: 0.2770
Surcharge Load Weight: 0 psf



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions using a horizontal seismic coefficient equal to one-half of PGA.
3. Analysis assumes no surcharge loading.

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GLOBAL STABILITY ANALYSIS BELTZ BRIDGE SEISMIC CONDITIONS

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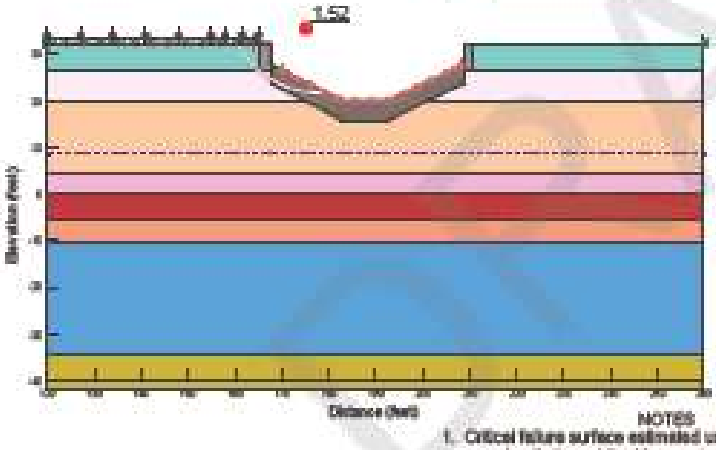
FIG. D10

DRAFT

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	ABUTMENT	High Strength	150			1
	ALLUVIAL FAN DEPOSITS - 1	Mohr-Coulomb	120	0	35	1
	ALLUVIAL FAN DEPOSITS - 2	Mohr-Coulomb	115	0	34	1
	ALLUVIAL FAN DEPOSITS - 2 (Liquefied Layer)	Mohr-Coulomb	115	0	17	1
	ALLUVIAL FAN DEPOSITS - 3	Mohr-Coulomb	130	0	38	1
	ASE FORMATION	Mohr-Coulomb	135	0	40	1
	FLUVIAL AND ESTUARINE DEPOSITS - 1	Mohr-Coulomb	120	0	35	1
	MARSH DEPOSITS	Mohr-Coulomb	100	50	28	1
	RIP RAP	Mohr-Coulomb	140	0	42	1
	ROADWAY FILL	Mohr-Coulomb	130	50	28	1

Beltz Bridge Post-seismic

Surcharge (Unit Weight): 0 pcf
Water Surface: Custom



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions. No seismic forces are included.
3. Analysis assumes no surcharge loading.

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GLOBAL STABILITY ANALYSIS BELTZ BRIDGE POST-SEISMIC W/ ADDITIONAL LOAD

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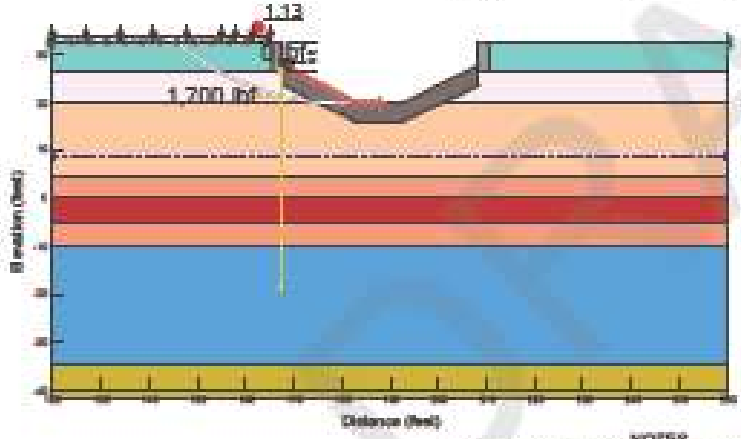
FIG. D11

DRAFT

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	ABUTMENT	High Strength	120			1
	ALLUVIAL FAN DEPOSIT 0 - 1	Mohr-Coulomb	120	0	35	1
	ALLUVIAL FAN DEPOSIT 0 - 2	Mohr-Coulomb	115	0	30	1
	ALLUVIAL FAN DEPOSIT 0 - 3	Mohr-Coulomb	130	0	38	1
	BEDROCK FORMATION	Mohr-Coulomb	135	0	40	1
	FLUVIAL AND ESTUARINE DEPOSIT 0 - 1	Mohr-Coulomb	120	0	30	1
	MARSH DEPOSITS	Mohr-Coulomb	100	50	26	1
	RIP RAP	Mohr-Coulomb	140	0	42	1
	ROADWAY FILL	Mohr-Coulomb	130	50	28	1

Beltz Bridge: Seismic

Shear Force: 1,200 lbf
Magnitude: 1,200 lbf
Horiz Seismic Coef.: 0.2778
Surcharge Load Weight: 0 pcf



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions using a horizontal seismic coefficient equal to one-half of PGA.
3. Analysis assumes no surcharge loading.

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GLOBAL STABILITY ANALYSIS BELTZ BRIDGE SEISMIC W ADDITIONAL LOAD

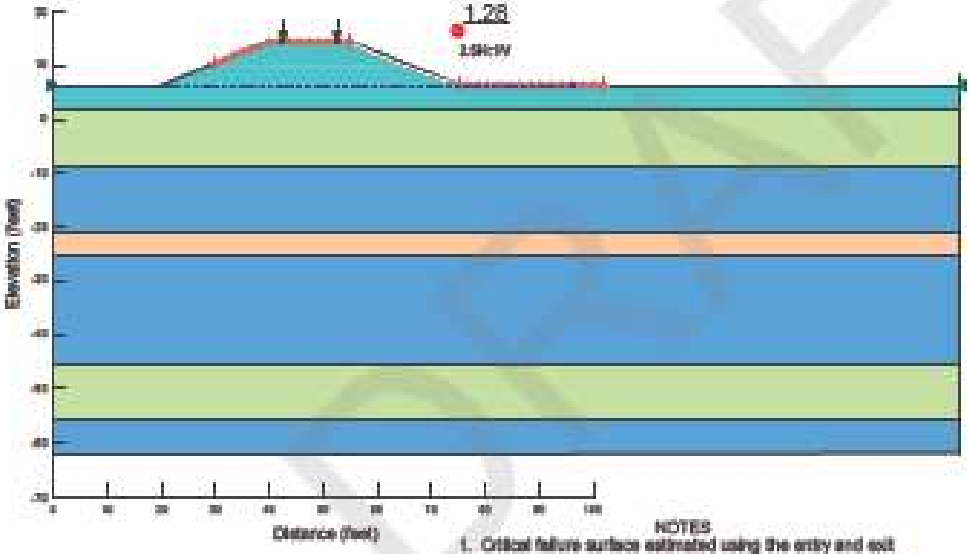
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FIG. D12

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Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Surface
	ALLUVIAL FAN DEPOSITS - 1	Mohr-Coulomb	120	0	35	1
	EMBANKMENT FILL (3)	Mohr-Coulomb	100	0	27	1
	FLUVIALAND ESTUARINE DEPOSITS - 1	Mohr-Coulomb	120	0	35	1
	FLUVIALAND ESTUARINE DEPOSITS - 2	Mohr-Coulomb	115	0	33	1



NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern-Price analysis method.
2. Analysis only assumes static conditions. No seismic forces are included.

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GLOBAL STABILITY ANALYSIS EXISTING TRAILHEAD EMBANKMENT STATIC CONDITIONS

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FIG. D13

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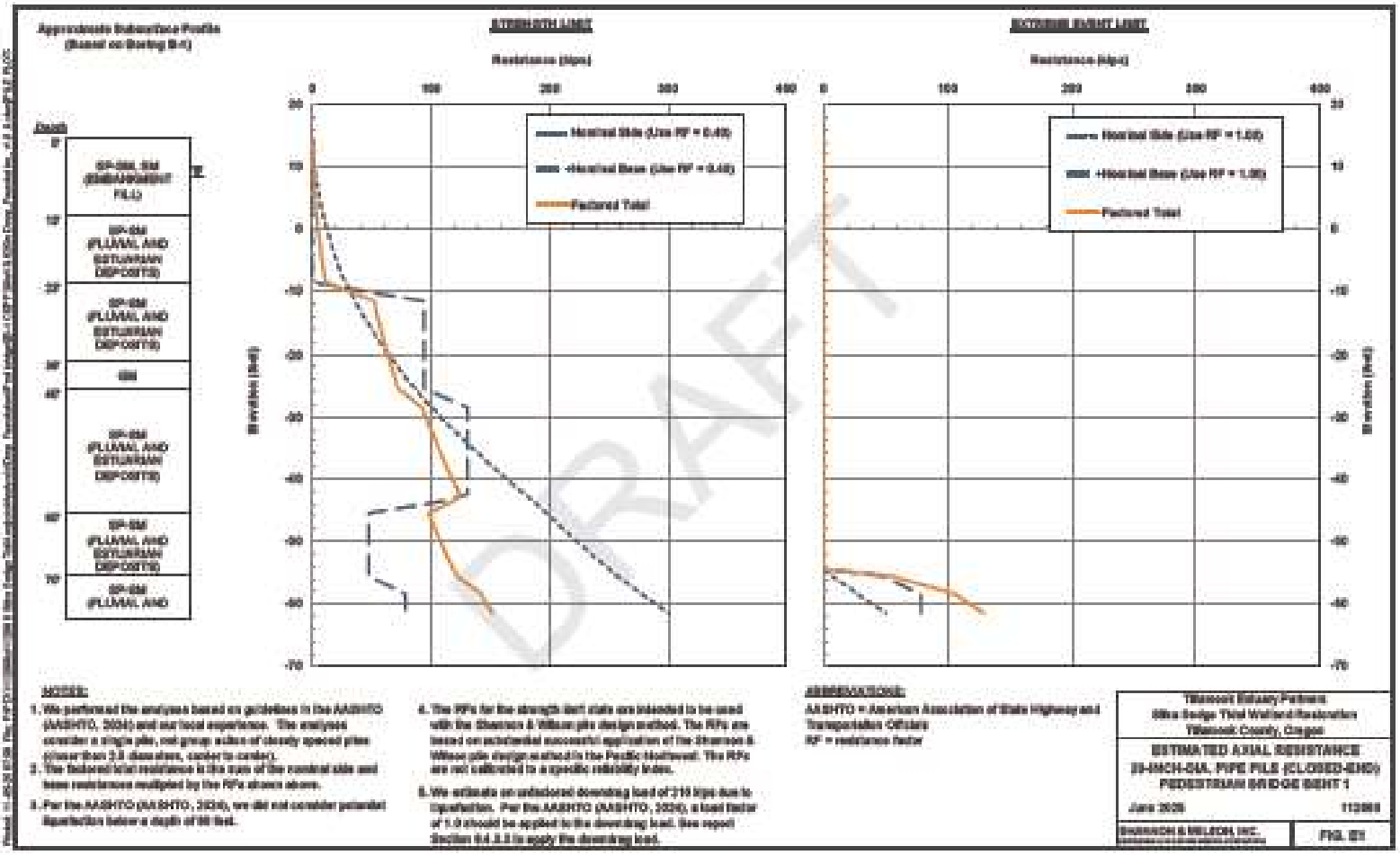
Appendix E

Slope Stability Analysis

Figures

Figure E1: Estimated Axial Resistance 20-Inch-Diameter Pipe Pile (Closed-End) Pedestrian Bridge Bent 1
Figure E2: Estimated Axial Resistance 20-Inch-Diameter Pipe Pile (Closed-End) Pedestrian Bridge Bent 2
Figure E3: Estimated Axial Resistance HP 16 x 141 Reneke Creek Bent 1
Figure E4: Estimated Axial Resistance HP 16 x 141 Reneke Creek Bent 2
Figure E5: Estimated Axial Resistance HP 14 x 141 Reneke Creek Abutment
Figure E6: Estimated Axial Resistance HP 16 x 141 Reneke Creek Abutment

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NOTES:

1. We performed the analyses based on guidelines in the AASHTO (AASHTO, 2024) and our local experience. The analyses consider a single pile, not group action of closely spaced piles (closer than 3.0 diameters, center to center).
2. The factored total resistance is the sum of the nominal side and base resistances multiplied by the RFs shown above.
3. Per the AASHTO (AASHTO , 2024), we did not consider potential liquefaction below a depth of 60 feet.
4. The RFs for the strength limit state are intended to be used with the Shannon & Wilson pile design method. The RFs are based on accidental successful application of the Shannon & Wilson pile design method in the Pacific Northwest. The RFs are not calibrated to a specific reliability index.
5. We estimate an unfactored downdrag load of 218 kips due to liquefaction. Per the AASHTO (AASHTO, 2024), a load factor of 1.0 should be applied to the downdrag load. See report Section 9.4.3.3 to apply the downdrag load.

ABBREVIATIONS:

AASHTO = American Association of State Highway and Transportation Officials
RF = resistance factor

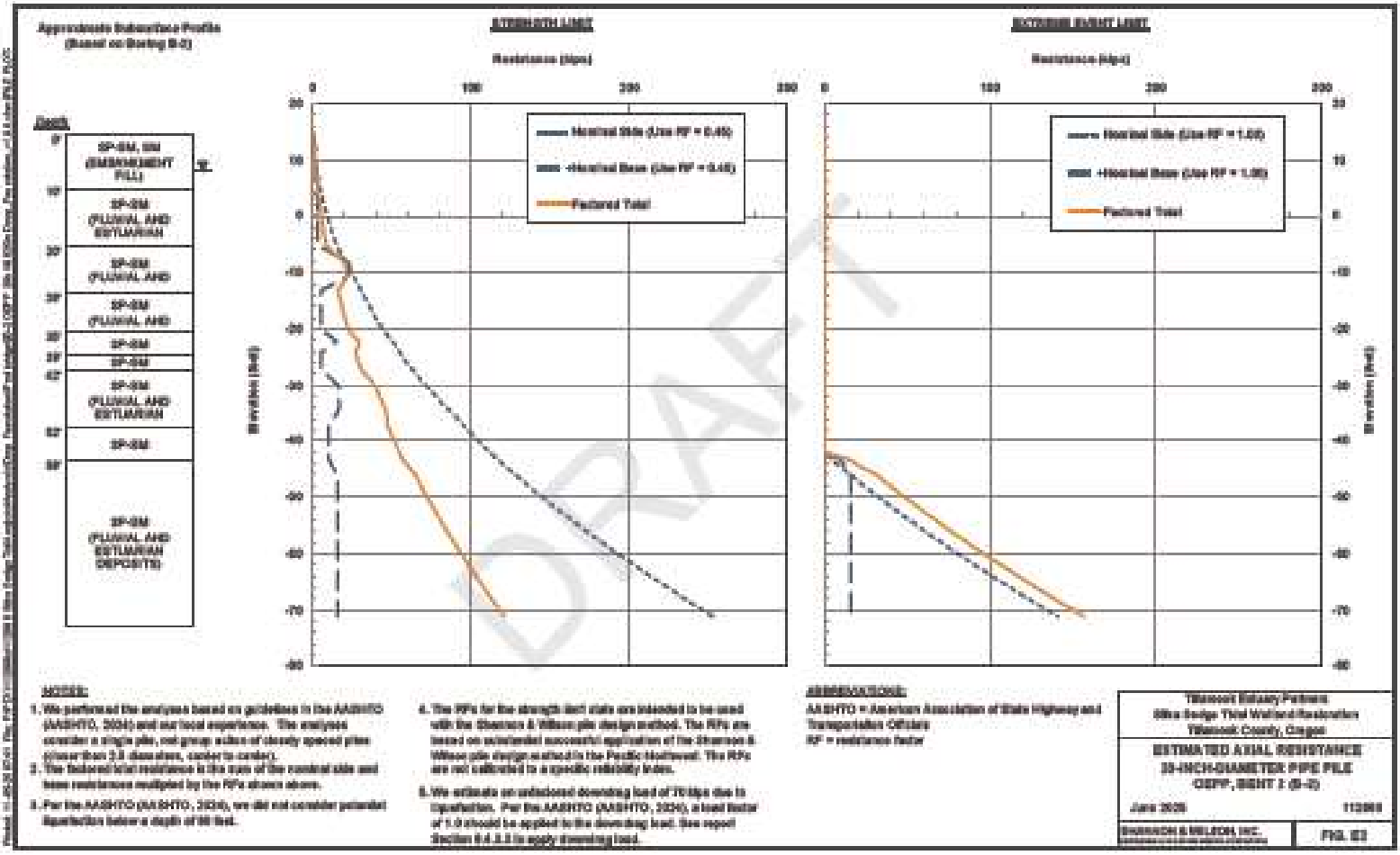
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Office Sedge Thiel Wetland Restoration
Tillamook County, Oregon

ESTIMATED AXIAL RESISTANCE
30-INCH-DIA. PIPE PILE (CLOSED-END)
PEDESTRIAN BRIDGE BENT 1

June 2026 113069

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FIG. E1



NOTES:

1. We performed the analyses based on guidelines in the AASHTO (AASHTO, 2024) and our local experience. The analyses consider a single pile, not group action of closely spaced piles (closer than 3.0 diameters, center to center).
2. The factored total resistance is the sum of the nominal side and base resistances multiplied by the RFs shown above.
3. Per the AASHTO (AASHTO , 2024), we did not consider potential liquefaction below a depth of 60 feet.
4. The RFs for the strength limit state are intended to be used with the Shannon & Wilson pile design method. The RFs are based on extended successful application of the Shannon & Wilson pile design method in the Pacific Northwest. The RFs are not calibrated to a specific reliability index.
5. We estimate an unfactored downdrag load of 70 kips due to liquefaction. Per the AASHTO (AASHTO, 2024), a load factor of 1.0 should be applied to the downdrag load. See report Section 9.4.3.3 to apply downdrag load.

ABBREVIATIONS:

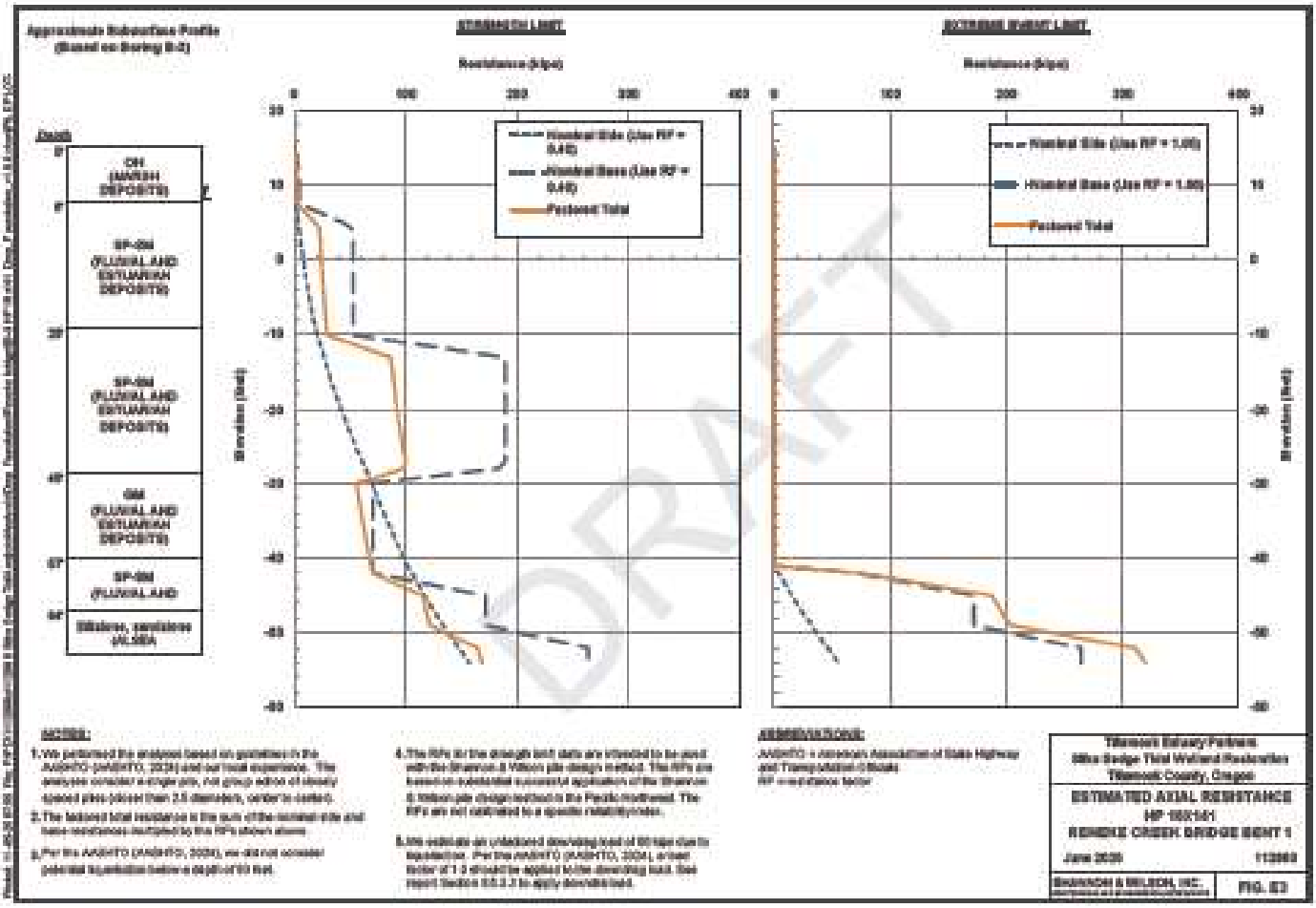
AASHTO = American Association of State Highway and Transportation Officials
RF = resistance factor

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Tillamook County, Oregon

ESTIMATED AXIAL RESISTANCE
30-INCH-DIAMETER PIPE PILE
OEPP, BENT 2 (B-2)

June 2026 | 113969

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NOTES:

1. We performed the analyses based on guidelines in the AASHTO (AASHTO, 2024) and our local experience. The analyses consider a single pile, not group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. The factored total resistance is the sum of the nominal side and base resistances multiplied by the RFs shown above.
3. Per the AASHTO (AASHTO, 2024), we did not consider potential liquefaction below a depth of 93 feet.
4. The RFs for the strength limit state are intended to be used with the Shannon & Wilson pile design method. The RFs are based on substantial successful application of the Shannon & Wilson pile design method in the Pacific Northwest. The RFs are not calibrated to a specific reliability index.
5. We estimate an unfactored downdrag load of 60 kips due to liquefaction. Per the AASHTO (AASHTO, 2024), a load factor of 1.2 should be applied to the downdrag load. See report Section 6.5.3.1 to apply downdrag load.

ABBREVIATIONS:

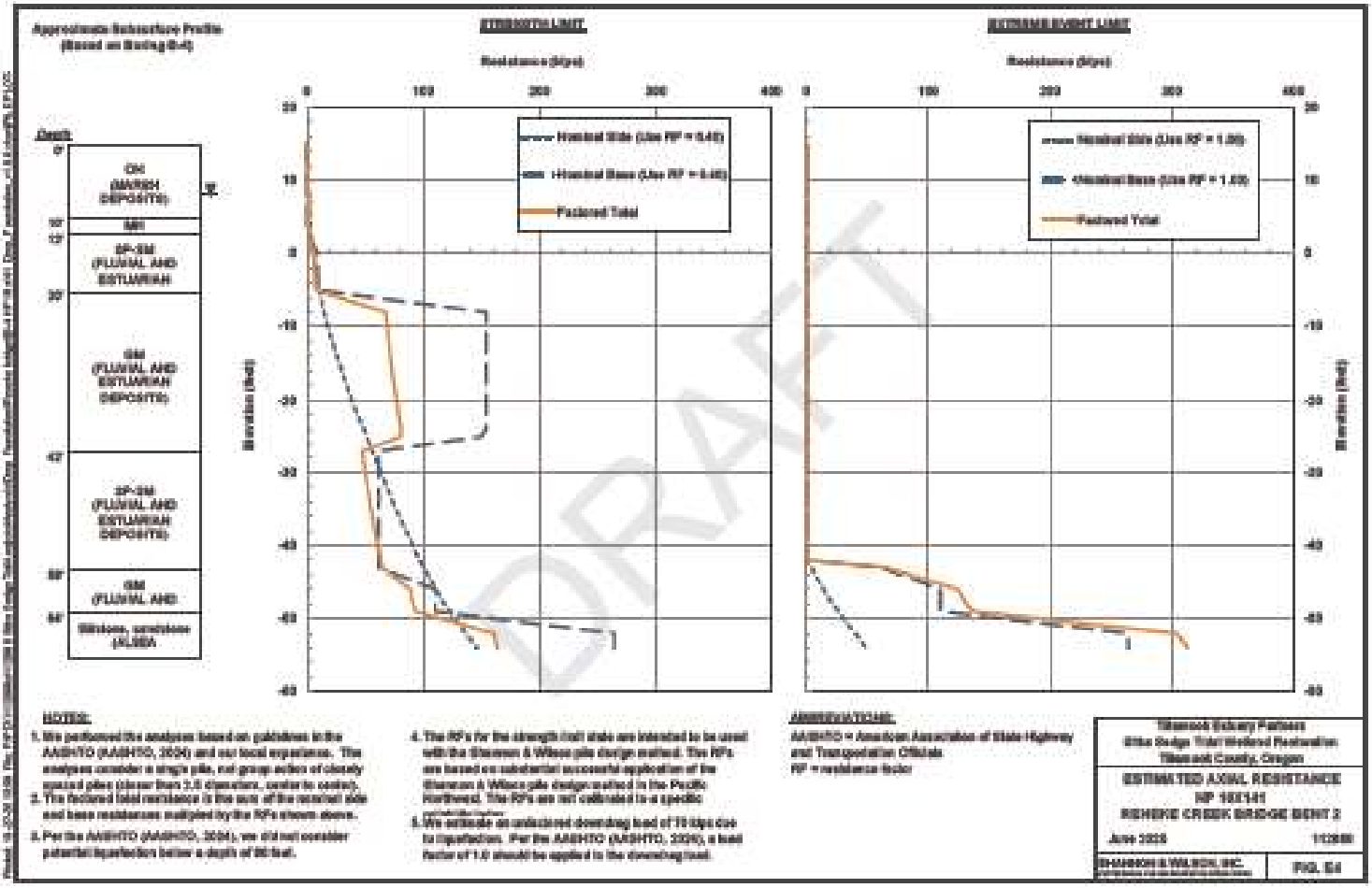
AASHTO = American Association of State Highway and Transportation Officials
RF = resistance factor

Tillamook Estuary Partners
Bike Sledge Tidal Wetland Restoration
Tillamook County, Oregon

ESTIMATED AXIAL RESISTANCE
HP 10X101
REHBEIN CREEK BRIDGE BENT 1

June 2026 113883

SHANNON & WILSON, INC. FIG. E3



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NOTES:

1. We performed the analyses based on guidelines in the AASHTO (AASHTO, 2024) and our local experience. The analyses consider a single pile, not group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. The factored total resistance is the sum of the nominal side and base resistances multiplied by the RFs shown above.
3. Per the AASHTO (AASHTO, 2024), we did not consider potential liquefaction below a depth of 80 feet.
4. The RF's for the strength limit state are intended to be used with the Shannon & Wilson pile design method. The RFs are based on substantial successful application of the Shannon & Wilson pile design method in the Pacific Northwest. The RFs are not calibrated to a specific
5. We estimate an unfactored downdrag load of 70 kips due to liquefaction. Per the AASHTO (AASHTO, 2020), a load factor of 1.0 should be applied to the downdrag load.

ABBREVIATIONS:

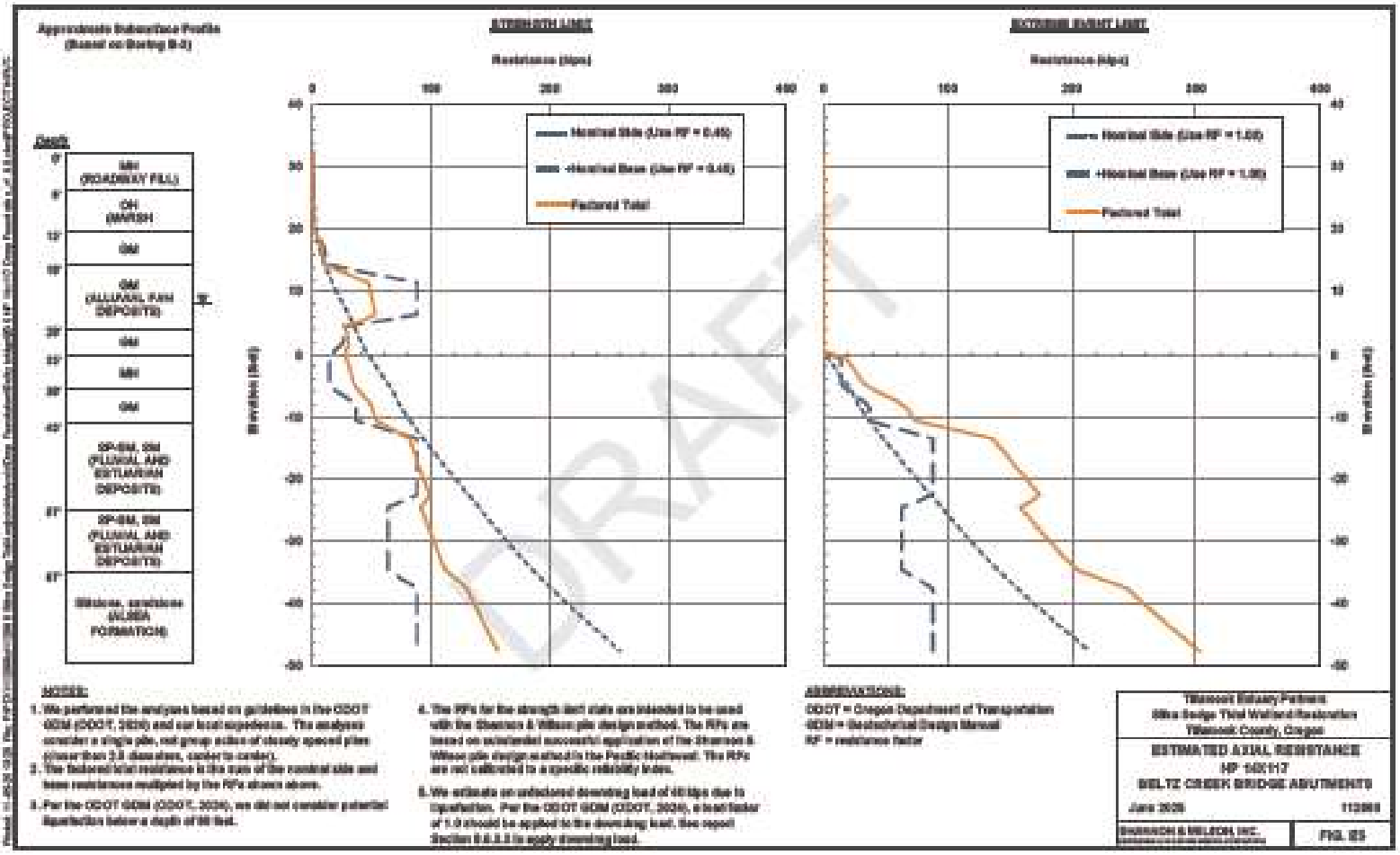
AASHTO = American Association of State Highway and Transportation Officials
RF = resistance factor

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Bike Bridge Trail Wetland Restoration
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ESTIMATED AXIAL RESISTANCE
RF 102141
RISHEIKE CREEK BRIDGE BENT 2

June 2025 | 112880

SHANNON & WILSON, INC. | FIG. E4



NOTES:

1. We performed the analyses based on guidelines in the ODOT GDM (ODOT, 2024) and our local experience. The analyses consider a single pile, not group action of closely spaced piles (closer than 3.0 diameters, center to center).
2. The factored total resistance is the sum of the nominal side and base resistances multiplied by the RFs shown above.
3. Per the ODOT GDM (ODOT, 2024), we did not consider potential liquefaction below a depth of 90 feet.
4. The RFs for the strength limit state are intended to be used with the Shannon & Wilson pile design method. The RFs are based on extended successful application of the Shannon & Wilson pile design method in the Pacific Northwest. The RFs are not calibrated to a specific reliability index.
5. We estimate an unfactored downdrag load of 40 kips due to liquefaction. Per the ODOT GDM (ODOT, 2024), a load factor of 1.0 should be applied to the downdrag load. See report Section 8.6.3.3 to apply downdrag load.

ABBREVIATIONS:

ODOT = Oregon Department of Transportation
GDM = Geotechnical Design Manual
RF = resistance factor

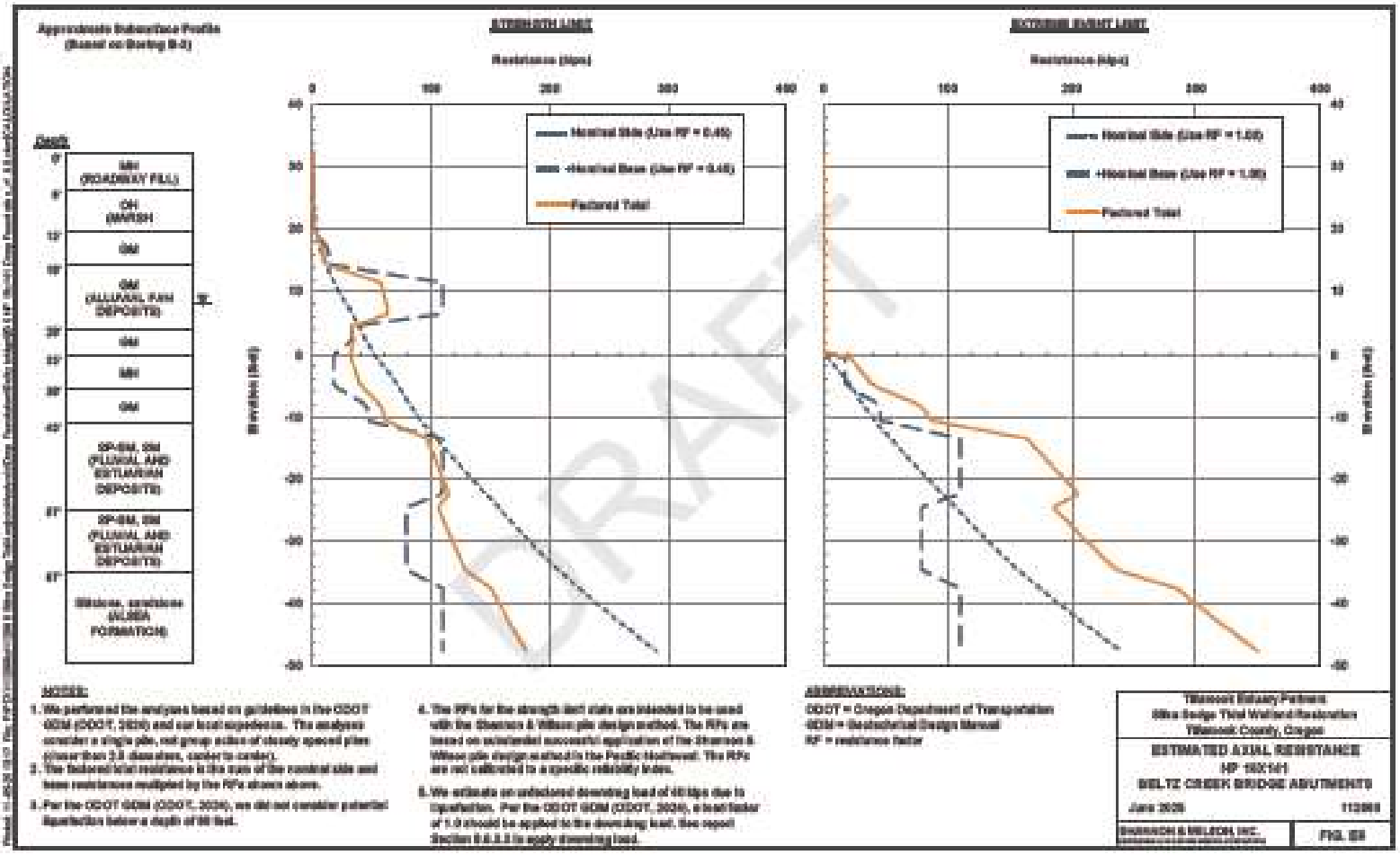
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Tillamook County, Oregon

ESTIMATED AXIAL RESISTANCE
HP 14X117
BELTZ CREEK BRIDGE ABUTMENTS

June 2026 113069

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FIG. E5



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NOTES:

1. We performed the analyses based on guidelines in the ODOT GDM (ODOT, 2024) and our local experience. The analyses consider a single pile, not group action of closely spaced piles (closer than 3.0 diameters, center to center).
2. The factored total resistance is the sum of the nominal side and base resistances multiplied by the RFs shown above.
3. Per the ODOT GDM (ODOT, 2024), we did not consider potential liquefaction below a depth of 90 feet.
4. The RFs for the strength limit state are intended to be used with the Shannon & Wilson pile design method. The RFs are based on extended successful application of the Shannon & Wilson pile design method in the Pacific Northwest. The RFs are not calibrated to a specific reliability index.
5. We estimate an unfactored downdrag load of 40 kips due to liquefaction. Per the ODOT GDM (ODOT, 2024), a load factor of 1.0 should be applied to the downdrag load. See report Section 6.6.3.3 to apply downdrag load.

ABBREVIATIONS:

ODOT = Oregon Department of Transportation
GDM = Geotechnical Design Manual
RF = resistance factor

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ESTIMATED AXIAL RESISTANCE
HP 10X101
BELTZ CREEK BRIDGE ABUTMENTS

June 2026 11388

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FIG. E9

Important Information

About Your Geotechnical/Environmental Report

IMPORTANT INFORMATION

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IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the Geoprofessional Business Association (https://www.geoprofessional.org)

Appendix G.
Sitka Sedge Tidal Wetland Restoration (Dike Setback) Preliminary Geotechnical Engineering Report



SUBMITTED TO:
Environmental Science Associates
1615 SE 3rd Avenue, Suite 400
Portland, OR 97214



BY:
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DRAFT

PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

Sitka Sedge Tidal Wetland Restoration (Dike Setback)

TILLAMOOK COUNTY, OREGON





SHANNON & WILSON

April 2026

Shannon & Wilson No: 112880

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Submitted To: Environmental Science Associates
1615 SE 3rd Avenue, Suite 400
Portland, OR 97214
Attn: Hunter White

Subject: DRAFT PRELIMINARY GEOTECHNICAL ENGINEERING REPORT, SITKA SEDGE TIDAL WETLAND RESTORATION (DIKE SETBACK), TILLAMOOK COUNTY, OREGON

Shannon & Wilson prepared this Preliminary Geotechnical Engineering Report (Dike Setback) and participated in this project as a subconsultant to Environmental Science Associates (ESA). Our scope of services was specified in Amendment 1, dated February 5, 2026, of the Subcontractor Agreement with ESA dated September 23, 2024. This report presents the findings of our geotechnical investigation for the Bridges and Culvert phase of this project, the results of our geotechnical analysis and assumptions for the Dike Setback design phase, and our recommendations for the geotechnical elements of the Dike Setback phase of the project. This report was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or if we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON

Travis Nguyen, PE, GE
Senior Associate

Eric Paslack, PE
Senior Associate

JMM:SCS:TTN:ECP:BSR/mmb:myw

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Appendix B: Nearby Field Explorations
Appendix C: Laboratory Test Results
Appendix D: Seepage and Slope Stability Analysis
Important Information

ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
bgs	below ground surface
FS	Factor of Safety
LRFD	Load and Resistance Factor Design
MTR	Muted Tidal Regulator
NAVD88	North American Vertical Datum of 1988
ODOT	Oregon Department of Transportation
SPT	Standard Penetration Test
USACE	U.S. Army Corps of Engineers
USCS	Unified Soil Classification System

1 INTRODUCTION

1.1 Project Overview

This preliminary report presents the results of our preliminary geotechnical design recommendations for a proposed dike setback in Sand Lake Estuary as part of the overall Sitka Sedge Tidal Wetland Restoration Project (the Project) in Tillamook County, Oregon. As a consultant to Environmental Science Associates (ESA), Shannon & Wilson is providing geotechnical engineering services, including field explorations, laboratory testing, and preliminary engineering analysis and recommendations for the proposed dike setback. The location of the project site is shown on the Vicinity Map, Figure 1.

A final report will be prepared for the dike setback after field explorations (anticipated in June 2026) are performed to confirm the assumed subsurface conditions. Preliminary geotechnical design recommendations for the bridges and culvert design elements were provided in a separate report titled "Sitka Sedge Tidal Wetland Restoration (Bridges and Culvert)" (Shannon & Wilson, 2025).

Elevations in this report are referenced to North American Vertical Datum 1988 (NAVD88).

1.2 Project Background

Based on our discussions with ESA, we understand that the proposed construction of the dike setback will begin in June 2026. We also understand that the permit applications to access the site and to perform geotechnical field explorations in the proposed dike setback area are currently being reviewed by the respective permitting agencies. The duration of the geotechnical field explorations and subsequent design may take several months to complete, and therefore, was anticipated to not meet the proposed project construction schedule. Therefore, ESA has requested that Shannon & Wilson design the dike setback without new field explorations and extrapolate information from the existing, previous borings completed for the bridges and culvert phase of the overall project for this dike setback design phase.

The geotechnical field explorations for the dike setback will be performed prior to construction to confirm the assumed subsurface conditions along the dike setback alignment and for our design. The proposed field exploration program for the dike setback consists of three (3) borings extending to a depth of 60 feet, as shown on the attached Site and Exploration Plan, Figure 2. We have discussed the risks of encountering different subsurface conditions from the assumed subsurface conditions with ESA. ESA understands

the risks associated with this approach, which may include geotechnical re-design, and updating plans, specifications, and construction costs. For the design of the dike setback, we have primarily relied on information from borings B-1 and B-2 for the water side, boring B-6 for the dike tie in to high ground on Sandlake Road, and boring B-7 for the landward side. The existing boring locations are shown on the attached Site and Exploration Plan, Figure 2.

1.3 Scope of Services

Shannon & Wilson's services were conducted in accordance with Amendment 1, dated February 5, 2026, of the Subcontractor Agreement with ESA dated September 23, 2024. The completed geotechnical design services for the Project consisted of the following tasks:

- Interpretation of subsurface conditions.
- Development of engineering material properties for geologic units.
- Design recommendations for the new tide gate culvert.
- Up to three filter designs for interfaces of dissimilar materials (such as under riprap or around tide gate culvert).
- Design recommendations for wing walls around the tide gate culvert, including lateral earth pressures under static conditions, hydrostatic pressures, and lateral resistance.
- Seepage and slope stability for the dike prism for up to two cross-sections following USACE design guidelines. Provide mitigation options to meet design factors of safety for slope stability, if required.
- Evaluate hydraulic gradients (i.e., underseepage) behind the dike prism. Provide mitigation options to meet the design factor of safety for underseepage, if required.
- Estimate settlement at the end of 3 months, 15 months, and the dike's design life (assume 50 years). Provide overbuild recommendations so that the dike will maintain its design crest elevation over time.
- Estimate the potential settlement of Sandlake Road due to the proposed dike.
- Provide recommendations for suitable dike embankment fill materials.
- Provide other relevant geotechnical-related construction considerations.
- Prepare this Preliminary Geotechnical Engineering Report summarizing the tasks listed above.

Our scope of services does not include scour analysis and seismic design or the risk evaluation associated with a seismic event occurring at or near the site.

2 PROJECT UNDERSTANDING

2.1 Site Description

The Project site is located within the Sitka Sedge State Natural Area (Sitka Sedge) and along Sandlake Road between the entrance to Sitka Sedge Parking and Trailhead and Roma Avenue in Tillamook County, Oregon. The Sitka Sedge is generally located within the Sand Lake Estuary, with a separate area of the Beltz Marsh located south of the Beltz Dike. The area was used for cattle grazing and dairy farming throughout the early to mid-1900s. A dike embankment (Beltz Dike) runs east to west across the marsh area and was constructed in the 1930s to extend the limits of grazing land (Wilsey and Ham, Inc., 1974). A culvert and tidal gate were installed within the central portion of the dike to control tidal flows within the grazing lands. Ground surface elevations generally range from between 7 to 9 feet within the Beltz Marsh, approximately 12 to 15 feet along Beltz Dike, and approximately 17 to 32 feet along the stretch of Sandlake Road within the Project area. The site is occupied by grass, shrubs, wetland vegetation, and a small channel that crosses the proposed dike setback alignment. The existing ground surface elevation at the proposed dike setback alignment ranges from approximately 5 to 10 feet. Exhibits 2-1 through 2-7 present site photographs showing several views of the site and existing features.



Exhibit 2-1: Looking West from the Trail on Beltz Dike Near the Proposed Pedestrian Bridge



Exhibit 2-2: Looking Southeast over Beltz Marsh and Hills East of Sandlake Road



Exhibit 2-3: Looking South at Beltz Marsh South of Beltz Dike



Exhibit 2-4: Looking West at the Beltz Dike Embankment Slope



Exhibit 2-5: Looking East at the Proposed Dike Setback Area in Sand Lake Estuary



Exhibit 2-6: Looking East at the Existing Small Channel Crossing the Proposed Dike Setback Area in Sand Lake Estuary



Exhibit 2-7: Looking East at Existing Vegetation within the Proposed Dike Setback Area in Sand Lake Estuary

2.2 Project Description

Based on 60 percent design drawings (ESA, 2025), which are attached in Appendix A, the proposed dike setback alignment traverses east to west across the Beltz Marsh area, and ties into high ground at the loop trail on the west side and Sandlake Road on the east side. The proposed dike setback is approximately 1,000 feet long and between 7 to 14 feet high, with a crest elevation of approximately 17.5 feet. A tide gate is planned through the setback dike at a location to the west of the existing channel. The proposed tide gate structure is a 12' x 10' box culvert with concrete headwalls and Muted Tidal Regulator (MTR) system, on a shallow-mat foundation of layered aggregate and geogrid. Based on our meeting with ESA on March 19, 2026, we understand that the proposed dike embankment prism core will have side slopes of 2 Horizontal to 1 Vertical (2H:1V) on both sides. An outer shell (i.e., planting berm) of material on the dike will extend the side slopes to 5H:1V on both sides for plantings, including forbs and shrubs, and to mitigate against erosion. The prism core and outer shell will be the same material with topsoil added to the outer shell for plantings. The proposed dike setback location is shown on the Site and Exploration Plan, Figure 2.

3 GEOLOGIC SETTING

3.1 Regional Geology

The project site lies within the Coast Range physiographic province near the western edge of the mountains of the Coast Range and in the estuary and wetland areas east of the shoreline of the Pacific Ocean. Basement rocks within the Coast Range consist of volcanic and sedimentary rocks of the Siletz Terrane. The Siletz Terrane is an exotic crustal block made up of rocks from an ocean island arc within the Pacific, which was accreted to the western edge of the North American continent approximately 51 to 49 million years ago (Wells and others, 1994). Overlying the rocks of the Siletz Terrane is a series of sedimentary rocks deposited in shallow seas within the western continental shelf of North America throughout the late Eocene (55.8 to 33.9 million years ago), Oligocene (55.8 to 23 million years ago), and into the Miocene (23 to 5.2 million years ago) (Wells and others, 1994).

Tectonic uplift of the Coast Range from the Miocene into the Pleistocene (2.3 million to 11,700 years ago), combined with erosion, created the rugged coastline and bay inlets seen today as the rivers and streams flowing out of the Coast Range mountains carved large valleys and canyons, which were inundated as sea levels rose after the last ice age. Sediments transported from inland, as well as from ocean currents and wind, accumulated within these inundated bay areas creating wetlands and estuaries.

3.2 Site Geology

The coastal area around Sand Lake is located within an estuary located between Pacific City to the south and Cape Lookout to the north. The estuary of Sand Lake was created within a wide ancient river channel where multiple creek valleys converge within an area of low elevation before draining into the Pacific Ocean. The estuary area is bounded and protected due to long-shore transport of beach sediment which developed sand bars along the shoreline that partially shielded Sand Lake from the ocean (Wilsey and Ham, Inc. 1974). Large amounts of sediment have been deposited within this estuary through a combination of sources, including stream alluvium deposited from creeks which drain from the Coast Range highlands, oceanic and tidal sediments deposited from long-shore currents and tidal activity, and eolian sediments deposited by the wind (Wilsey and Ham, Inc. 1974; Wells and others, 1994). The surface geology of the greater Sand Lake area, including the project site, is generally mapped by Wells and others (1994) as quaternary sediment, including fluvial and estuarine deposits of unconsolidated clay, silt, sand, and gravel deposited by rivers and streams, as well as tidal flat mud, sand, and peat. Wells and others (1994) mapped the uplands to the east as Alsea Formation, consisting of Oligocene and Miocene marine sedimentary rocks, including tuffaceous siltstone and sandstone. The Alsea formation is locally intruded by irregular bodies of 6 to 17 million-year-old Grande Ronde Basalt (Wells and others, 1994).

Landslides and debris flows are common within the slopes of the mountains and foothills of the Coast Range (DOGAMI, 2024). According to the Statewide Landslide Information Database for Oregon (SLIDO), alluvial fan deposits are mapped within the alignment and near borings B-3, B-4, and B-5. A small landslide is mapped between B-6 and B-7. The landslide deposits near B-7 are designated as historic (<150 years old), and the other alluvial fan and landslide deposits are designated as prehistoric (>150 years old) (DOGAMI, 2024). Exhibit 3-1 presents an image from SLIDO of the mapped landslide located upslope from Sandlake Road.

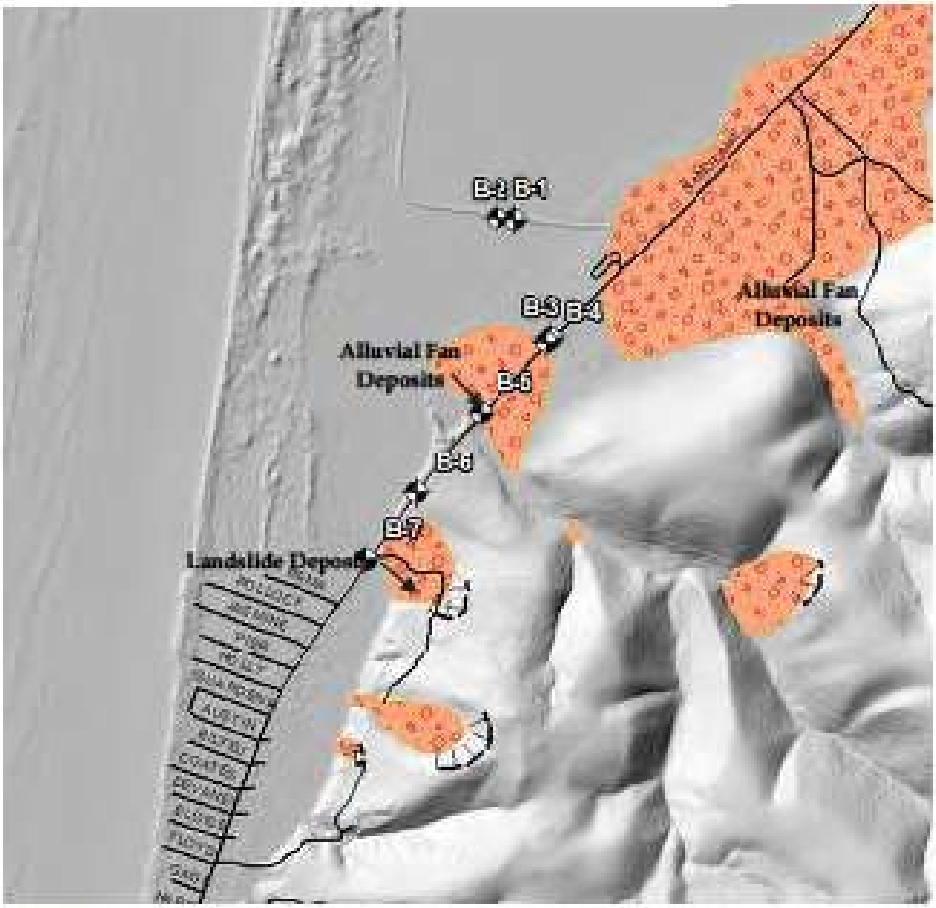


Exhibit 3-1: SLIDO data showing mapped landslides and alluvial fan deposits within the project area (DOGAMI, 2024).

In recent history, the landscape of the Sand Lake Estuary has been further altered by human development, including changes to waterways, grading, and placement of fill material. Three dike structures have altered the tidal flow of the estuary. These dike structures include part of the Tillamook County access road to Whalen Island, the Beltz Dike, and a flood control dike located on the north shore of the Sandlake Road between Galloway Road and Sand Lake east of the recreation area. These dikes have constricted tidal flows and created significant increases in marsh areas within the bay (Wilsey and Ham, Inc., 1974).

4 FIELD EXPLORATIONS FOR BRIDGES AND CULVERT

Field explorations for the proposed dike setback have not yet been performed. The geotechnical field explorations for the dike setback will be performed prior to construction to confirm the subsurface conditions assumed for the dike setback design. For

completeness, this report includes information from the borings performed for the bridges and culvert design phase of the project, which have informed our assumptions for the preliminary design of the dike setback.

Shannon & Wilson explored subsurface conditions for the bridges and culvert design phase of the project with seven (7) geotechnical borings designated B-1 through B-7. Boring exploration depths and locations are summarized in Exhibit 4-1. Geotechnical borings B-3 through B-7 were drilled within the roadway of Sandlake Road between March 24, 2025, through March 28, 2025. Geotechnical borings B-1 and B-2 were drilled along the trail embankment between March 31, 2025, through April 1, 2025. The borings were drilled using a CME-55LC track rig provided and operated by Western States Soil Conservation Inc. (Western States) of Hubbard, Oregon. The boring locations were measured by handheld GPS in the field, and the approximate boring locations are shown on the Site and Exploration Plan, Figure 2.

Exhibit 4-1: Summary of Field Explorations

Location	Geotechnical Boring Designation	Approximate Exploration Depth (feet)
Pedestrian Bridge Bent 1	B-1	76.5
Pedestrian Bridge Bent 2	B-2	86.5
Reneke Creek Bridge Bent 1	B-3	70.1
Reneke Creek Bridge Bent 2	B-4	70.1
Beltz Creek Bridge South Abutment	B-5	74.1
Dike Setback Tie into Sandlake Road	B-6	90.1
No Name Creek Culvert	B-7	41.5

A Shannon & Wilson geologist was on-site throughout the exploration program to locate the borings, observe the drilling, collect soil samples, and log the materials encountered. Details of the exploration program, including descriptions of the techniques used to advance and sample the borings, logs of the materials encountered, and backfill details, are presented in Appendix B, Nearby Field Explorations.

5 LABORATORY TESTING

Laboratory tests were performed on selected samples from the explorations for the bridges and culvert to determine the basic index and engineering properties of the soils encountered. The samples we obtained during our field explorations were transported to our laboratory for further examination. The initial soil testing program for bridge and culvert design included moisture content tests, Atterberg Limit tests, oven-dried liquid limit

tests, particle-size analyses, and a corrosivity suite. In addition to the laboratory testing performed for the bridges and culvert, laboratory testing was performed on select samples remaining in storage for the preliminary dike setback design, including particle-size analyses and consolidation testing. Testing was performed by Shannon & Wilson at our in-house laboratories in Lake Oswego, Oregon and Seattle, Washington, and by Cooper Testing, Inc. of Palo Alto, California. Laboratory tests were performed in accordance with applicable American Society for Testing and Materials (ASTM) International standards. The results of the laboratory tests and brief descriptions of the test procedures are presented in Appendix C, Laboratory Test Results.

6 SUMMARY OF ASSUMED SUBSURFACE CONDITIONS

6.1 General

The explorations and initial laboratory testing were performed to evaluate geotechnical soil and groundwater conditions for the proposed bridge and culvert structures. This section describes the general soil units encountered in our previous subsurface exploration program, which informed the assumed conditions for the dike setback.

Our observations presented in this report are specific to the locations, depths, and times noted on the logs and may not be applicable to all areas of the site. No amount of explorations or testing can precisely predict the characteristics, quality, or distribution of subsurface and site conditions. Potential variation includes, but is not limited to, the following:

- The conditions between and below explorations may be different.
- The passage of time or intervening causes (natural and manmade) may result in changes to site and subsurface conditions.
- Groundwater levels and flow directions may fluctuate due to seasonal, irrigation-related, and recharge source variations.

If conditions different from those described herein are encountered during construction, we should review our description of the subsurface conditions and reconsider our conclusions and recommendations.

6.2 Geotechnical Soil Units

Shannon & Wilson grouped the materials encountered in our field explorations into six (6) geotechnical units, as described below. This interpretation of the subsurface conditions is

based on the explorations and regional geologic information from published sources. The geotechnical units are as follows:

- **Roadway Fill**: includes asphalt pavement, roadway base aggregate, and Sandlake Road fill consisting of loose to medium dense, Silty Gravel with Sand (GM) and very soft to soft, Elastic Silt with Sand (MH);
- **Embankment Fill**: includes Beltz Dike gravel surfacing and dike fill consisting of very loose to loose, Poorly Graded Sand with Silt (SP-SM);
- **Marsh Deposits**: very soft to soft, Organic Soil (OH);
- **Alluvial Fan Deposits**: loose to very dense, Silty Gravel to Silty Gravel with Sand (GM); soft to stiff, Elastic Silt to Elastic Silt with Sand (MH); medium dense, Silty Sand to Silty Sand with Gravel (SM);
- **Fluvial and Estuarian Deposits**: very loose to very dense, Poorly Graded Sand with Silt (SP-SP); very loose to very dense, Silty Sand to Silty Sand with Gravel (SM); very loose, Silty Gravel (GM); very soft to soft, Elastic Silt (MH); and
- **Alsea Formation**: extremely weak to weak (R0-R2), slightly to moderately weathered, Siltstone/Sandstone; very weak to weak (R1-R2), slightly to moderately weathered, Siltstone.

The geotechnical units listed above were grouped based on their engineering properties, geologic origins, and their distribution in the subsurface. Contacts between the units may be more gradational than shown in the Drill Logs in Appendix B and the modeled cross sections in Appendix D. The Standard Penetration Test (SPT) N-values shown on the Drill Logs and discussed below are in blows per foot (bpf) as counted in the field (uncorrected). SPT blow counts in gravelly soils may not be representative of actual soil density due to the influence of gravel and cobble clasts on the sampler. The following sections describe the geotechnical unit characteristics in greater detail.

6.2.1 Roadway Fill

Roadway Fill was encountered in borings B-3 through B-7 and extends to depths ranging between 6 to 8.5 feet. Fill used to construct Sandlake Road was encountered below the base aggregate in Borings B-5 and B-6 and generally consisted of loose to medium dense, Silty Gravel with Sand (GM), and very soft to soft, Elastic Silt with Sand (MH). SPT N-values ranged between 2 and 10 bpf and averaged 6 bpf. Asphalt pavement and base aggregate are included within the Roadway Fill. The asphalt pavement and base aggregate thickness varied between about 18 to 30 inches.

6.2.2 Embankment Fill

Embankment Fill was characterized as material used to construct the Beltz Dike in the locations of borings B-1 and B-2. The Embankment Fill was encountered from the ground surface to depths of 12.5 and 9.5 feet, respectively, and generally consisted of very loose to loose, Poorly Graded Sand with Silt (SP-SM). Pockets of Silty Sand (SM) were observed in some samples. SPT N-values in the unit ranged from 0 to 4 bpf and averaged 2 bpf.

6.2.3 Marsh Deposits

Marsh Deposits were encountered in borings B-3, B-4, B-5, and B-7, and extended to depths ranging between 7.5 and 12.5 feet. The Marsh Deposits generally consisted of high plasticity Organic Soil (OH). Few to little organic wood and debris material was observed within the samples, including what appeared to be interbeds of peat in boring B-7. Natural moisture contents ranged from 83 to 112 percent. Plasticity indices ranged between 31 and 93, with oven-dried liquid limits ranging between 47 and 56 percent. Particle-size analysis on one sample indicated a fines content of 64 percent. A Unified Soil Classification System (USCS) designation of OH was assigned after assessment of the natural moisture content and Atterberg Limits testing data. SPT N-values ranged from 0 to 2 bpf and averaged 1 bpf.

6.2.4 Alluvial Fan Deposits

Alluvial Fan Deposits were encountered in borings B-1, B-2, B-5, and B-6 between depths of 2.5 to 35.0 feet bgs and ranged in thickness between approximately 4.5 and 16.0 feet. The unit generally consisted of loose to very dense, Silty Gravel to Silty Gravel with Sand (GM); soft to stiff, Elastic Silt to Elastic Silt with Sand (MH); and medium dense, Silty Sand to Silty Sand with Gravel (SM). The Alluvial Fan Deposits are interbedded within the Fluvial and Estuarian Deposits and interpreted to be ancient and modern alluvial fans which were created as fine- and coarse-grained sediment was transported and deposited at the mouth of the stream channels into the bay as they exited the mountains of the Coast Range. Some of the alluvial fans are interpreted to have formed before the Holocene Epoch during the ice age, when sea levels were lower, and then as sea levels rose, these alluvial fans were overlain and buried under younger Fluvial and Estuarian Deposits.

Within the unit, the fines ranged from low to medium plasticity, and an Atterberg Limits test was performed on a single selected sample which indicated a plasticity index of 16 and a USCS designation of MH. Disturbed textures were observed with moderate to heavy iron oxide staining. The unit generally ranged in color from red-brown to gray and brown. Natural moisture contents ranged from 30 to 42 percent. Particle-size analyses indicated fines ranging between 35 and 76 percent. SPT N-values ranged from 6 to 32 bpf and

averaged 19 bpf, with one SPT meeting refusal, where over 50 blows were required to advance the sampler 6 inches.

6.2.5 Fluvial and Estuarian Deposits

Fluvial and Estuarian Deposits were encountered in all borings, underlying Embankment Fill and Marsh Deposits, and were interbedded with Alluvial Fan Deposits. The unit generally consists of very loose to very dense, Poorly Graded Sand with Silt (SP-SP); very loose to very dense, Silty Sand to Silty Sand with Gravel (SM); very loose, Silty Gravel (GM); and very soft to soft, Elastic Silt (MH). Fines in the unit generally ranged from nonplastic to high plasticity. Slight to moderate iron oxide staining was observed in some samples, and the material was typically micaceous. The sand constituent was generally fine to coarse-grained. Natural moisture contents ranged from 20 to 50 percent. Particle-size analyses indicated fines ranging between 7 and 46 percent. One Atterberg Limits test was performed on a single selected sample which indicated a plasticity index of 17 and a USCS designation of MH. SPT N-values ranged from 2 to 91 bpf and averaged 28 bpf.

6.2.6 Alsea Formation

The Alsea Formation was encountered in borings B-3, B-4, B-5, and B-6 at depths ranging between 60 and 67 feet, and the borings were terminated within the unit. The Alsea Formation generally consisted of extremely weak to weak (R0-R2), slightly to moderately weathered, Siltstone/Sandstone; and very weak to weak (R1-R2), slightly to moderately weathered, Siltstone.

6.3 Groundwater

The borings were drilled using mud rotary drilling techniques, which introduce fluid into the boring during drilling, making it difficult to discern groundwater levels. For the dike setback design, we assumed groundwater levels at the ground surface, with surface water elevations provided by ESA and described later.

6.4 Soil Corrosivity

The proposed dike setback will be subjected to a marine environment. Analytical testing was performed to evaluate the corrosivity potential of the soil at the project site. The corrosivity testing suite included soil resistivity, pH, chloride content, sulfate content, sulfide content, and oxidation-reduction potential. Analytical testing was performed by Cooper Testing Laboratory, Inc., in Palo Alto, California. Corrosivity test results are presented in Appendix C.

Soil corrosivity potential was evaluated based on the guidelines in Section 10.7.5 of the AASHTO LRFD (AASHTO, 2024). Soil corrosivity tests were performed on a soil sample at a depth of 10 feet from boring B-1 and a soil sample at a depth of 2.5 feet from boring B-3. The soil sample at a depth of 10 feet from boring B-1 is considered to be corrosive based on test results. Therefore, structures within the dike setback (e.g., culverts, tide gates, flap gates, sheet piles, etc.) should be designed for corrosion.

7 GEOTECHNICAL ENGINEERING ANALYSIS

7.1 General

Where applicable, analysis methods, design parameters, and design criteria were adopted from the current USACE design and analysis guidance. Since dike setback explorations were not available at the time of the analysis, we assumed the following possible subsurface conditions for our analyses:

- Assumed Conditions – Based on our understanding of the geology of the site, we expect that the subsurface conditions at the setback dike location to be similar to those encountered in borings B-1 and B-2, primarily consisting of coarse-grained fluvial and estuarian deposits (i.e., free-draining sands). Our models primarily considered layers of Silty Sand (SM) and Poorly Graded Sand with Silt (SP-SM), with minor layers (less than 5-feet thick) of Silty Gravel (GM) and Elastic Silt (MH) at an approximate depth of 30 feet, as encountered in boring B-1. The bedrock depth was assumed to be approximately 60 feet, based on boring B-6. These assumed conditions are supported by the two shallow (i.e., 1-foot-deep) hand-augers performed by Pacific Groundwater Group, which did not find low-permeability soils that could act as a barrier to groundwater exchange with surface water (Pacific Groundwater Group, 2019). Hand auger logs are not available in the Pacific Groundwater Group report.
- Sensitivity Analysis – While not expected, it is possible that a surficial low-permeability, compressible, confining layer may be present at the dike setback location, or on the landward side of the proposed dike setback. This would be a critical condition to consider for settlement, seepage, and stability analyses. Therefore, we performed sensitivity analyses assuming primarily sands on the water side (as described above and found in borings B-1 and B-2), with a surficial layer of Marsh Deposits (OH) at the dike location and on the landward side. The Marsh Deposits were assumed from the ground surface to an elevation of 0 feet (with an approximate thickness of 5 feet), as found in boring B-7. By inspection, the assumed conditions are similar to the fine-grained Alluvial Fan Deposits (MH) found in boring B-6. This possibility is also supported by the clay layer found at an approximate depth of 8 feet by Pacific Groundwater Group in their boring PGG-1i on Roma Ave (Pacific Groundwater Group, 2019). Boring PGG-1i is included in Appendix B, for reference.

7.2 Settlement Analysis

7.2.1 General

Constructing the dike setback through placement of fill will result in settlement of the dike embankment, nearby ground surface, and nearby structures. To estimate the magnitude and extent of these settlements, we modeled the dike construction using the software program Settle3 (Rocscience, 2024). The input parameters and analysis methods were in general agreement with EM 1110-1-1904 (USACE, 1990) and EM 1110-2-1913 (USACE, 2000).

Soil layers in the settlement models were based on the assumed conditions and sensitivity analyses as described in Section 7.1. Due to the lack of soil information, and for simplicity, settlement models were assumed to have flat (horizontal) soil layers. Conservatively, and for simplicity, coarse-grained Fluvial and Estuarian Deposits (SM, SP-SM) and coarse-grained Alluvial Fan Deposits (GM) were given material properties for SM, which is assumed to have the smallest elastic modulus of the coarse-grained soils. The settlement models (both Assumed Conditions and Sensitivity Analyses) included thin (2- to 3-foot-thick) fine-grained Alluvial Fan Deposits (MH) layers at depths of approximately 8 feet and 25 feet based on borings PGG-1i and B-1.

7.2.2 Settlement Models

One settlement model was created for each subsurface condition evaluated (Assumed Conditions and Sensitivity Analysis), as described in Section 7.1. The proposed dike setback was grouped into similar areas based on proposed height and crest width to model the load variation across the dike length.

The 60 percent design plans include 2 feet of overbuild (ESA, 2026), which will be constructed during the initial construction phase. We understand that a second phase of construction will occur after about one year of settlement and will include constructing the tide gate and re-grading the overbuild to 2 feet above the long-term design crest elevation.

7.2.3 Soil Parameters for Settlement Analysis

Information from the available borings (for the bridges and culvert), associated laboratory testing, typical values from the literature, and our previous experience on similar projects was used to determine appropriate immediate (elastic) settlement and consolidation parameters for the various soils likely to be present below the dike setback. Soil parameters used in our analysis are provided in Exhibit 7-1.

Exhibit 7-1: Soil Parameters for Settlement Analysis

Material	Es (ksf)	Cc	Cr	eo	OCR	Cv (ft²/y)	Ca
Marsh Deposits (OH)	36	0.5	0.02	2.02	1	33	0.03
Fluvial and Estuarian Deposits (SM)	340	-	-	-	-	-	-
Alluvial Fan Deposits (MH)	122	0.5	0.02	2.02	1	33	0.01

NOTES: Es = Elastic Modulus, Cc = Compression Index, Cr = Recompression Index, e0 = initial void ratio, OCR = Over-Consolidation Ratio, Cv = Coefficient of Consolidation, Ca = Secondary Compression Index
ksf = kips per square foot, ft2/y = square feet per year

7.2.4 Ground Surface and Embankment Loads

The existing ground surface varies between about elevation (El.) 5 and 10 feet along the proposed dike centerline. The design elevation of the proposed dike setback, including the overbuild considered in the 60 percent plans, varies from El. 17 feet (typical) to 18 feet (at the tide gate) and 22 feet (near the tie-in at Sandlake Road). The amount of fill proposed along the dike setback varies from about 0 to 14 feet, depending on location. Due to limitations within Settle3, in which loads may only be applied on a horizontal surface, the ground surface was assumed to be uniform at El. 7 feet, which is considered generally representative for the majority of the dike length.

The proposed dike setback was modeled with multiple embankment loads, each with a unit weight of 115 pounds per cubic foot, side slopes of 5H:1V, and a base width, crest width, and height corresponding to the drawings. The assumed unit weight of 115 pcf is based on assumed import dike fill recommended in Section 8.6. The loads were increased after the first year, to add a height corresponding to the consolidation settlement to-date after construction, therefore representing the re-grading that will take place. The tide gate was modeled as being constructed after the first year (preloading phase), by replacing that portion of the embankment with a rectangular load corresponding to the estimated weight of the concrete and cover soil as shown in the drawings.

7.2.5 Groundwater Assumptions

The groundwater table was assumed to be at the ground surface for all settlement models; small changes in groundwater elevation do not significantly change the results of this settlement analysis.

7.2.6 Settlement Analyses Results

The estimated elastic settlement, consolidation settlement, and secondary settlement are presented in Exhibit 7-2. Settlement will vary along the dike alignment. Greater settlements

are expected where the dike embankment is tallest; settlements would be expected to be less where the embankment is shorter. The values presented in Exhibit 7-2 represent a location of anticipated maximum settlement for the type of settlement, subsurface condition, and time-period shown. Consolidation settlement and secondary settlement are presented after 1 year (preloading phase), and after 50 years (Project design life). Values are also presented for "Post-Regrading Settlement" after 50 years, representing the design lifetime settlement that is expected after regrading is performed in the second construction phase.

Exhibit 7-2: Settlement Results

Result	Assumed Conditions (Sand)	Sensitivity Analysis (Marsh Deposits)
Elastic Settlement during Initial Construction (inches)	4	7
Consolidation Settlement after 1 year (inches)	4	20
Secondary Settlement after 1 year (inches)	<0.5	<0.5
Consolidation Settlement after 50 years (inches)	4	22
Secondary Settlement after 50 years (inches)	0.5	2
Post-Regrading Consolidation Settlement after 50 years (inches)	<0.5	4
Post-Regrading Secondary Settlement after 50 years (inches)	0.5	2

The majority of settlement is expected to happen between the first and second phase of construction. The time for consolidation settlement to occur is a function of the thickness of the fine-grained soil layer (e.g., OH, MH, CL) squared. For the estimated Coefficient of Consolidation and an assumed Marsh Deposit thickness of 7 feet, the theoretical time to 90% consolidation is between 1 and 2 years. However, the estimated time would increase for increased thickness of Marsh Deposits. Thin horizontal sand layers, which can be common in fluvial environments and difficult to capture in geotechnical investigations, would decrease the time for consolidation settlement. Exhibit 7-3 shows the estimated total settlement (elastic settlement, consolidation settlement, and secondary settlement) over time for the tallest proposed embankment section (approximately Station 9+50).

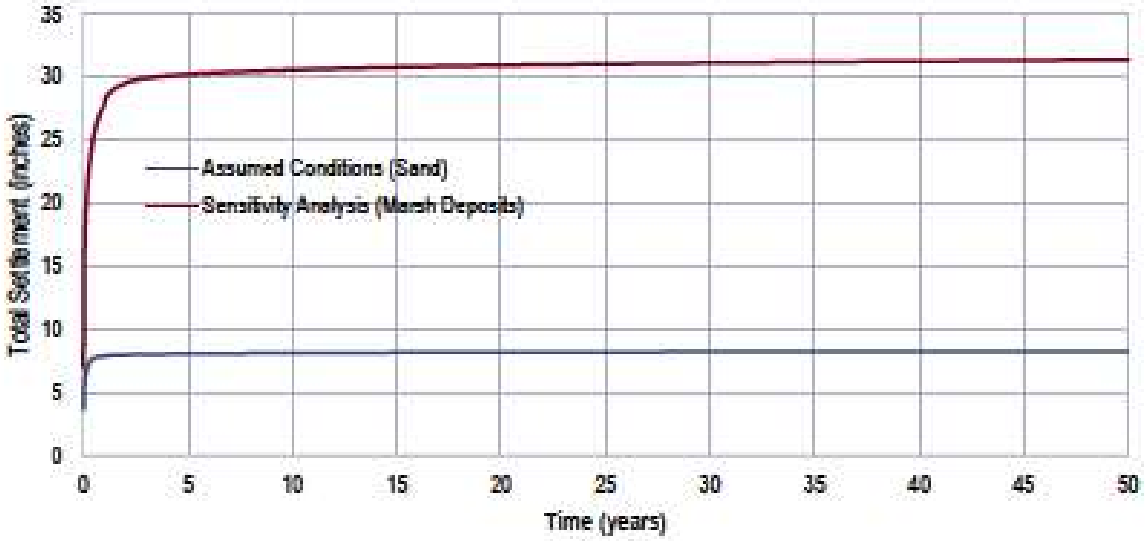


Exhibit 7-3: Time versus Total Settlement, Tallest Embankment Section

Based on the results of our settlement analyses, and the estimated post-regrading settlement, we anticipate that the proposed construction sequence and including 2 feet of overbuild in the 60 percent plans is sufficient to maintain the long-term design crest height for the design life of 50 years.

Exhibit 7-4 provides settlement at the proposed tide gate and for Sandlake Road where the dike setback connects to the road. Total settlement for the tide gate includes elastic, consolidation, and secondary settlement from tide gate construction (which will occur in the second phase of construction after preloading) to the design life of 50 years. The Sandlake Road settlement presented is total settlement (elastic, consolidation, and secondary settlement) due to dike setback loads over the design life of 50 years.

Exhibit 7-4: Settlement of Structures and Pavement

Result	Assumed Conditions (Sand)	Sensitivity Analysis (Marsh Deposits)
Total Settlement at Tide Gate[1] (inches)	0.5	2
Differential Settlement at Tide Gate[2] (inches)	<0.5	0.5
Sandlake Road Settlement (inches)	0.5	1

NOTES:

1 Maximum tide gate settlement is expected at the centerline of the dike.

2 Differential settlement at the tide gate is measured perpendicular to the dike alignment, from the center of the tide gate (i.e., approximate dike centerline) to the upstream or downstream edge of slab (approximate distance of 30 feet).

We recommend installing settlement plates and monitoring the settlement over time so that we can compare the actual settlements to the estimated settlement results, as described in Section 8.

7.3 Seepage and Stability Analyses

7.3.1 General

Several factors influence the slope stability of a dike, including (1) the geometry of the dike and subsurface materials; (2) the weight of soil materials overlying the failure surface; (3) the shear strength of soils along the failure surface; and (4) the hydrostatic pressure (groundwater levels) present within the dike and along the failure surface. The stability of a slope is expressed in terms of a factor of safety (FS), which is defined as the ratio of resisting forces to driving forces. At equilibrium, the FS is equal to 1.0 and the driving forces are balanced by the resisting forces. Failure occurs when the driving forces exceed the resisting forces, i.e., the FS is less than 1.0. An increase in the FS to greater than 1.0, whether by increasing the resisting forces or decreasing the driving forces, reflects a corresponding increase in the stability of the dike.

Seepage can also negatively impact the stability of the dike landside slopes. The seepage of water increases porewater pressures in and below the dike. The increase in pore pressure causes a decrease in effective stress which causes a decrease in shear strength of the soils. Because of this effect, our seepage analyses are coupled with our stability analyses. In a steady-state seepage analysis, the surface water elevation is considered to exist at a single stage indefinitely, and the model is run until the water pressures and flow rates have reached steady values. The use of steady-state conditions is consistent with the methodology included in EM 1110-1-1913 (USACE, 2000). In a transient seepage analysis, the flood is modeled as a stage hydrograph (surface water elevation versus time) and water pressures and flow rates vary for each time step.

7.3.2 Slope Stability and Seepage Criteria

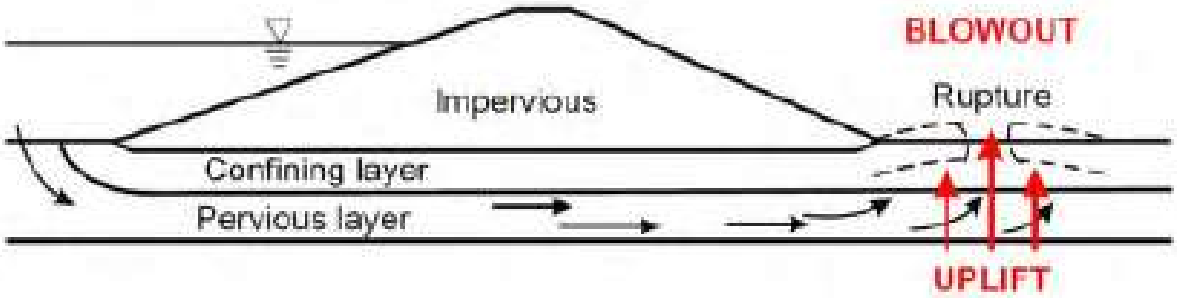
Stability and seepage analyses for the project were performed in general accordance with EM 1110-2-1913 (USACE, 2000) and EM 1110-2-1902 (USACE, 2003). EM 1110-2-1913 requires that slope stability be evaluated for three cases as follows:

- Case I – End of Construction: This case applies to both the water side and landside of the dike embankment. The USACE and Project minimum required FS for Case I is 1.3.
- Case II – Rapid Drawdown: This case applies to the water side of the dike embankment. The USACE and Project minimum required FS for Case II is 1.0.
- Case III – Steady Seepage: This case applies to the landside of the dike embankment. The USACE and Project minimum required FS for Case III is 1.4.

Seepage through and under a dike during a flood event can produce piping and uplift near the landside toe. Underseepage is of particular concern when there is a thin confining (low

permeability layer) overlying an aquifer (high permeability layer). In this case, the seepage pressures in the aquifer may exceed the overburden pressure of the confining layer, which could result in a rupture of the confining layer and possible internal erosion (Exhibit 7-5). As discussed in the U.S. Bureau of Reclamation Dam Safety Best Practices Chapter D-6 (USBR, 2019), underseepage can lead to a progressive failure that can occur in a single event or become progressively worse over the course of multiple events.

Exhibit 7-5: Underseepage Causing Uplift/Blowout (USBR, 2019)



The potential for underseepage related uplift/blowout is evaluated by comparing the calculated exit gradient to the allowable exit gradient of the surface soil near the dike toe (i.e., across the confining layer). The exit gradient is defined as the difference in total head between the bottom of the impervious (confining) soil layer and the ground surface divided by the thickness of the soil layer. The allowable exit gradient of a soil is directly proportional to its density. Therefore, the allowable gradient of the light-weight organic silt (OH) near the surface is relatively low (0.35) compared to typical non-organic soils (approximately 0.5). Allowable exit gradients were calculated using a FS of 1.6 (USACE, 2005a). Therefore, when the calculated exit gradient is equal to or less than the allowable exit gradient, the FS for underseepage is equal to or greater than 1.6.

Steady state in a tidal environment is conservative. If an analysis section did not meet design criteria for underseepage, based on the results of the steady-state seepage analysis, then a transient seepage/slope stability analysis was performed for that section.

7.4 Seepage and Slope Stability Models

SEEP/W and SLOPE/W (Bently Systems, 2025) were used to perform seepage and stability calculations, respectively. SEEP/W is a finite element analysis software program which we used to calculate porewater pressures and phreatic surfaces for tides and storm surges. SLOPE/W is a limit-equilibrium analysis software program. Failure surfaces were analyzed using the Morgenstern-Price method to calculate the FS. Failure surfaces less than 3-feet deep, i.e., shallow failures that are caused by sloughing and erosion and generally considered maintenance issues, were excluded from our analysis. The lowest FS from

thousands of potential failure surfaces as calculated by SLOPE/W is reported for each analysis.

Two model cross-sections were developed based on the assumed conditions and sensitivity analysis as described in Section 7.1. The stability and seepage models represent the dike post-construction. The end-of-construction case considers the overbuild amount and flatter post-construction slopes of 5H:1V. The long-term cases consider the long-term design dike crest elevation and the interior dike prism core (i.e., without the outer shell flatter slopes designed for planting) of 2H:1V. All models included a proposed basal geogrid (assumed Miragrid 5XT) at the bottom of the dike prism.

7.4.1 Soil Parameters for Seepage Analysis

Information from the available laboratory testing (i.e., soil gradation tests), hydraulic conductivities used in the hydrogeologic evaluation (Pacific Groundwater Group, 2019), typical values from literature, and our previous experience on similar projects was used to determine appropriate seepage parameters for the various soils. Soil parameters used in our seepage analyses are provided in Exhibit 7-6.

For materials expected to be below the phreatic surface, the Saturated Only material model was used. For dike materials the Saturated/Unsaturated material model was used. For Saturated/Unsaturated material models, the Volumetric Water Content and Hydraulic Conductivity Functions were developed using estimation methods included in the SEEP/W software package.

Exhibit 7-6: Soil Parameters for Seepage Analysis

Geologic Unit	Material	Kx (ft/s)	Ky/Kx
Fill	Dike Embankment Fill	1.0E-6	1
Fluvial and Estuarian Deposits	SP-SM	9.0E-4	0.1
	SM	4.0E-4	0.1
	MH	5.0E-6	1
Alluvial Fan Deposits	GM	4.0E-4	1
Marsh Deposits	OH	5.0E-6	1
Alsea Formation	Bedrock	1.0E-7	1

NOTES: Kx = Horizontal Hydraulic Conductivity, Ky/Kx = Anisotropy; ft/s = feet per second

7.4.2 Soil Parameters for Slope Stability Analysis

Soil parameters used in our stability analyses are provided in Exhibits 7-7 and 7-8. Information from the available subsurface explorations (i.e., SPT blow counts, pocket

penetrometer readings, and torvane shear data), laboratory testing (i.e., soil gradation tests and Atterberg Limits tests), typical values from literature, and our previous experience on similar projects was used to determine appropriate shear strength parameters for the various soils. Fully softened drained shear strengths were developed using the correlations by Stark & Idriss (2023) using liquid limit, clay-size fraction, and effective normal stress.

Exhibit 7-7: Soil Parameters for Slope Stability Analysis

Geologic Unit	Material	γ (pcf)	Peak Drained		Peak Undrained	
			ϕ' (degrees)	c' (psf)	ϕ (degrees)	c (psf)
Fill	Dike Embankment Fill	115	30	0	20	350
Fluvial and Estuarian Deposits	SP-SM	120	35	0	-	-
	SM	120	32	0	-	-
	MH	100	29	0	0	600
Alluvial Fan Deposits	GM	130	38	0	-	-
Marsh Deposits	OH	100	24	0	0	300
Alsea Formation	Bedrock	140	High Strength			

NOTES: γ = total unit weight, pcf = pounds per cubic foot, psf= pounds per square foot, ϕ' = effective friction angle, c' = effective cohesion, ϕ = total friction angle, c = total cohesion

Exhibit 7-8: Fully Softened Drained Shear Strengths for Marsh Deposits (OH)

Normal Stress (psf)	Shear Strength (psf)
251	126
1,044	438
2,089	800
8,354	2,671

NOTES: psf= pounds per square foot

Peak drained shear strengths were used for free draining soils, while peak drained, fully softened drained, and peak undrained strengths were used for low permeability soils.

For low permeability soils the shear strengths used for each analysis case are as follows:

- Case I – End of Construction: Peak undrained shear strengths
- Case II – Rapid Drawdown (Steady-State Analysis): Staged analysis using peak drained and peak undrained shear strengths
- Case III – Steady Seepage (Steady-State and Transient Analysis): Fully softened drained shear strengths (Marsh Deposits) and peak drained shear strength (Embankment Fill and Fluvial and Estuarian Deposits)

7.4.3 Groundwater and Floodwater Assumptions

7.4.3.1 Steady-State Seepage

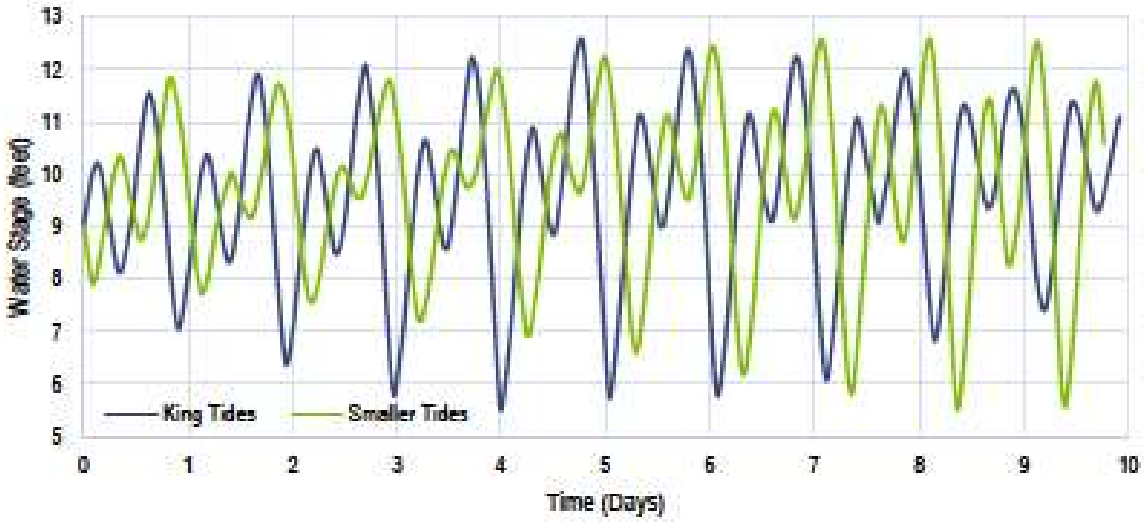
For the steady-state seepage analysis, ESA has chosen to use a design high water elevation of 12.6 feet, which is approximately equal to the king tide plus storm surge (ESA, 2026).

7.4.3.2 Transient Seepage

Initial porewater conditions for the transient seepage analysis were calculated from a steady-state parent analysis with the water stage at an average of the design high water elevation (king tide plus storm surge) and low tide elevation, equal to about El. 9 feet.

We obtained tidal data for the nearby station at Garibaldi, OR (Station 9437540) (NOAA, 2026) for two 10-day periods of interest: one representative of king tides (12/1/2025 – 12/10/2025), and another representative of smaller tides (8/1/2025 – 8/10/2025). We then scaled the tidal curves for each time period to have a maximum equal to the design high water elevation of 12.6 feet, and a minimum equal to the low tide elevation of 5.5 feet. The resulting stage hydrographs are shown in Exhibit 7-9.

Exhibit 7-9: Stage Hydrographs Considered for Transient Analysis

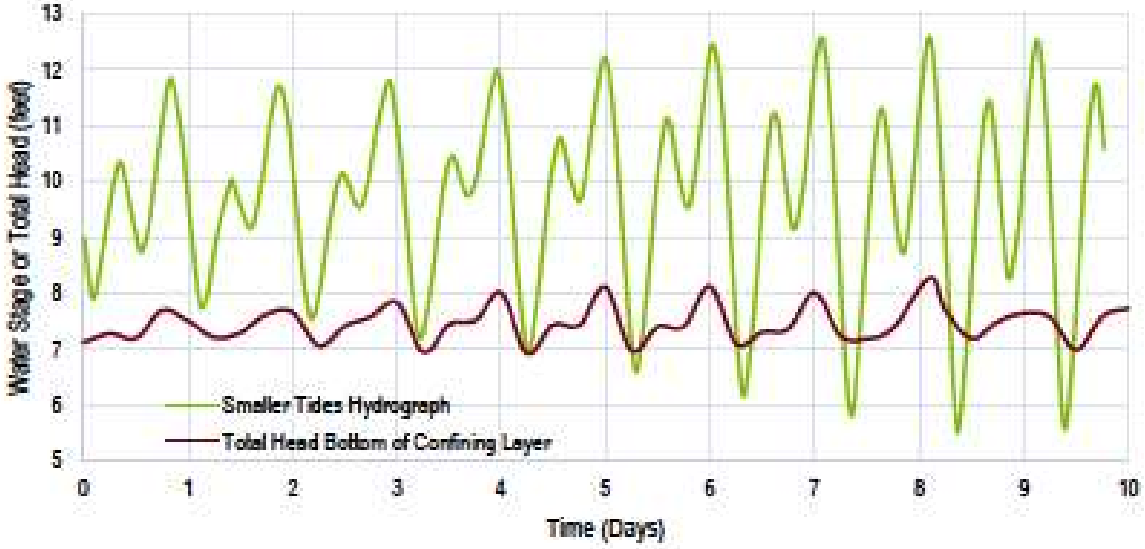


We compared the results of the transient analysis for the two stage hydrographs and found that the stage hydrograph for the smaller tides was more critical, so that hydrograph was used for further evaluations, as described in more detail below.

We also compared the total head at the bottom of the confining layer (used in the exit gradient calculation) versus the stage hydrograph and found that while the total head is fluctuating with the tide elevation, it is not generally increasing over time (Exhibit 7-10).

Therefore, the 10-day duration of the hydrograph used in the transient analysis appears sufficiently long for the purposes of evaluating underseepage.

Exhibit 7-10: Stage Hydrograph versus Total Head at Bottom of Confining Layer



7.4.3.3 End of Construction (Case I)

Groundwater levels for Case I (End of Construction) were held constant at the ground surface elevation.

7.4.3.4 Rapid Drawdown (Case II)

For Case II (Rapid Drawdown) using results of the steady-state seepage analysis, a two-stage analysis was performed: Stage 1) the water stage at the design high water elevation (El. 12.6 feet) with the phreatic surface within the dike determined from the steady-state seepage analysis, followed by; Stage 2) the water stage at the low tide elevation of 5.5 feet (ESA, 2026) with the phreatic surface within the dike held in place from Stage 1.

7.4.3.5 Steady Seepage (Case III)

For Case III (Steady Seepage), the pore water pressure distribution and phreatic surface calculated in the steady-state SEEP/W model were coupled to the stability model.

7.4.4 Seepage and Slope Stability Analyses Results

Exhibit 7-11 provides the calculated FS for seepage and slope stability, compared to the USACE and Project design criteria, for the assumed conditions and sensitivity analysis.

Exhibit 7-11: Seepage and Stability Analysis Results (Without Mitigation)

Analysis Case	Porewater Pressures	Slope	USACE and Project Design Criteria	Calculated FS
Assumed Conditions (Sand)				
End of Construction[1]	-	Waterside	1.3	2.9
	-	Landside		2.9
Rapid Drawdown[2]	-	Waterside	1.0	0.6
Steady Seepage[2]	Steady-State Seepage Analysis	Landside	1.4	1.2
Underseepage[3]	Steady-State Seepage Analysis	Landside	1.6	5.7
Sensitivity Analysis (Marsh Deposits)				
End of Construction[1]	-	Waterside	1.3	2.3
	-	Landside		2.3
Rapid Drawdown[2]	-	Waterside	1.0	0.6
Steady Seepage[2]	Steady-State Seepage Analysis	Landside	1.4	0.8
Underseepage	Steady-State Seepage Analysis	Landside	1.6	0.8
	Transient Seepage Analysis			0.9

NOTES:

1 End of Construction stability models include the planned 2-foot overbuild and 5H:1V side slopes.

2 Rapid Drawdown and Steady Seepage stability models have the long-term design dike crest elevation, and an internal 2H:1V dike prism core.

3 A transient seepage analysis was not performed if the calculated FS meets the USACE and Project design criteria from the steady-state seepage analysis.

As shown in Exhibit 7-11, the USACE and Project design criteria are not met for rapid drawdown or steady-seepage (both conditions), or underseepage (sensitivity analysis). We iteratively reviewed mitigation options and found that including four layers of Miragrid 5XT geogrid in the 2H:1V dike prism core, at 2.5-foot vertical spacing (El. 5 feet, 7.5 feet, 10 feet, and 12.5 feet) increases the FS to meet criteria for rapid drawdown and steady seepage conditions (Exhibit 7-12).

Exhibit 7-12: Seepage and Stability Analysis Results (With Mitigation)

Analysis Case	Porewater Pressures	Slope	USACE and Project Design Criteria	Calculated FS
Assumed Conditions (Sand)				
Rapid Drawdown[1]	-	Waterside	1.0	1.3
Steady Seepage[1]	Steady-State Seepage Analysis	Landside	1.4	3.1
Sensitivity Analysis (Marsh Deposits)				
Rapid Drawdown[1]	-	Waterside	1.0	1.2
Steady Seepage[1]	Steady-State Seepage Analysis	Landside	1.4	2.0

NOTES:

1 Rapid Drawdown and Steady Seepage stability models have the long-term design dike crest elevation, and internal 2H:1V dike prism core. The modeled mitigation included four layers of SXT geogrid at EL 5 feet, 7.5 feet, 10 feet, and 12.5 feet.

Due to the uncertainty of the subsurface conditions at the proposed dike location, we performed a series of transient seepage analyses to determine the minimum thickness of Marsh Deposits, if present to meet the USACE underseepage design criteria for varying ground surface elevations (Exhibit 7-13). If Marsh Deposits are found to be present during construction, and are thinner than the minimum thickness presented, then mitigations would be required (e.g., over-excavation or installing a trench drain or relief wells as discussed further below).

Exhibit 7-13: Transient Seepage Analysis Results, Sensitivity Analysis (Marsh Deposits), Minimum Confining Layer Thickness

Analysis Case	Ground Surface Elevation (feet)	Minimum Thickness of Confining Layer (Marsh Deposits) Without Mitigation[1]	USACE and Project Design Criteria	Calculated FS
Underseepage	5	11	1.6	1.6
	6	8		1.6
	7	6		1.6
	8	5		1.8
	9	3		1.6
	10	2		1.6

NOTES:

1 If Marsh deposits are found during construction and are thinner than the minimum thickness shown (after clearing and grubbing, as measured at the landward toe), then we would recommend installing a seepage mitigation design, as discussed in Section 7.4.5.

Figures of the seepage and stability model results are presented in Appendix D.

7.4.5 Mitigation Design Recommendations

Based on our experience with similar projects, mitigation design options to relieve underseepage pressures could include a cut-off wall, trench drain, passive relief wells, or a landside seepage berm. Advantages and disadvantages to each mitigation design option are presented in Exhibit 7-14, below.

Exhibit 7-14: Advantages and Disadvantages to Underseepage Mitigation Design Options

Mitigation Design Option	Advantages	Disadvantages
Sheet Pile or Bentonite Cut-Off Wall	▪ Would reduce additional water on the landside of the Dike (compared to relief wells and trench drain options)	▪ Would need to penetrate the entire pervious layer to be effective in reducing underseepage gradients (depth unknown, >70 feet below native ground surface) ▪ Difficulty sealing the joints in sheet pile cut-off walls for a waterproof barrier ▪ Additional explorations required to determine design depth ▪ Judged to be the highest cost mitigation design as well as the option with the most risk during installation ▪ Would require advanced lead-time and specialty contractor(s)/equipment
Trench Drain	▪ Relieves underseepage pressures, also resulting in higher factors of safety for landside stability ▪ Judged to be relatively low-cost with typical materials and construction methods ▪ Judged to be the most flexible mitigation option, and could be installed at specific reaches as necessary using equipment likely already on-site	▪ May require shoring and dewatering during construction, which increases risk and uncertainty of construction cost ▪ Would need to confirm filter design/compatibility for trench drain fill ▪ Trench drain fill meeting filter design would need to be purchased and imported to the site ▪ Would result in additional water on the landside of the Dike during flood events, which may be acceptable for this tidally influenced area

Mitigation Design Option	Advantages	Disadvantages
Passive Relief Wells	▪ Relieves underseepage pressures, also resulting in higher factors of safety for landside stability ▪ Would not require shoring or dewatering during construction ▪ Judged to be relatively low-cost considering short Dike setback length ▪ Could be installed at specific reaches as necessary	▪ Would need to perform filter design for the material that surrounds the well screen ▪ Filter compatible material surrounding well screen would need to be purchased and imported to the site ▪ Would require detailed well design (including calculations and seepage analysis) to confirm well size and spacing, with an additional boring at each proposed well location ▪ Would require advanced lead time, coordination of drillers and drill rig ▪ Would need to penetrate the entire pervious layer to be effective in relieving underseepage pressures (depth unknown, >70 feet below native ground surface) ▪ Requires more regular maintenance than a trench drain ▪ Will suffer loss in efficiency over time ▪ Would result in additional water on the landside of the Dike during flood events, which may be acceptable for this tidally influenced area
Landside Seepage Berm (i.e., raising the landward ground surface)	▪ Relieves underseepage pressures, also resulting in higher factors of safety for landside stability ▪ Uses common earthwork techniques for construction ▪ Minimal maintenance required over time ▪ Would not result in additional water on the landside of the Dike during flood events ▪ Judged to be relatively low-cost (ignoring potential costs due to negotiations with nearby property owners)	▪ Would need to perform filter design for the seepage berm fill material ▪ Would need to perform settlement evaluation of seepage berm to determine any overbuild required ▪ In our experience, seepage berms often need to be at least 300 feet long (distance from the dike toe to the seepage berm toe) to be effective in relieving underseepage pressures ▪ The amount of fill required for a seepage berm would likely be detrimental to the environmental and habitat benefits of this Project ▪ Judged to be the least flexible mitigation options, requiring significant imports of additional fill material and re-grading of the plans

Based on the advantages and disadvantages of each mitigation option, the construction schedule, and the uncertainties/lack of subsurface information, we believe that a trench drain is the most appropriate mitigation option to plan for this Project. Depending on the soils encountered in the explorations and during construction, the trench drain may be omitted. If a confining layer (i.e., Marsh Deposits) are encountered at the surface, with thicknesses less than shown in Exhibit 7-13, then a trench drain can be installed in that specific area.

7.5 Tide Gate Engineering Analysis and Design Recommendations

The proposed dike setback includes a 12' x 10' concrete cast-in-place box tide gate structure with concrete headwalls and MTR system, on a shallow-mat foundation of layered aggregate and geogrid. The following sections provide recommended bearing resistance, sliding resistance, lateral earth pressures, hydrostatic pressures, and uplift pressures for use in structural design.

7.5.1 Bearing and Sliding Resistance

The required FS for bearing capacity of normal structures (i.e., structures in which failure is not anticipated to result in loss of life), with ordinary site information, and unusual loading conditions is 2.0 (Table 7.1, USACE, 2022). While the current site information is limited, we expect that borings performed prior to construction will be used to confirm the bearing resistances provided herein. We calculated allowable bearing resistance for the proposed tide gate structure and associated walls for the Assumed Conditions (sand) and Sensitivity Analysis (Marsh Deposits). Conservatively, the bearing resistances provided are for native materials, not considering the proposed layers of geogrid and crushed surfacing base course (CSBC) below the foundations. The bearing resistance for the Sensitivity Analysis (Marsh Deposits) case considers undrained strengths. The recommended allowable bearing resistances are provided in Exhibit 7-15.

Exhibit 7-15: Recommended Allowable Bearing Resistance

Analysis Case	Allowable Bearing Resistance (psf)[1]
Assumed Conditions (Sand)	2,000
Sensitivity Analysis (Marsh Deposits)	900

NOTES: psf= pounds per square foot

1 Allowable bearing resistances provided include a FS of 2.0.

2 Allowable bearing resistances provided assume a footing width of 10 feet, and a footing aspect ratio (L/B) of 2.

We recommend that sliding for the tide gate structure and associated headwalls be evaluated in accordance with EM 1110-2-2502 Section 7.3 (USACE, 2022) and EM 1110-2-2100 (USACE, 2005b). The required FS for sliding of normal structures, with ordinary site information and unusual loading conditions, is 1.3 (Table 3-3, USACE, 2005b). While the current site information is limited, we expect that borings performed prior to construction will be used to confirm the sliding coefficients provided herein. We recommend using the sliding coefficients shown in Exhibit 7-16 for design of the tide gate and tide gate headwalls.

Exhibit 7-16: Recommended Ultimate Sliding Coefficients

Analysis Case	Sliding Coefficient[1]
Assumed Conditions (Sand)	0.5
Sensitivity Analysis (Marsh Deposits)	0.35

NOTES
1 We recommend using an FS of 1.3 (not included in the sliding coefficient above) for sliding calculations.

7.5.2 Lateral Earth Pressures, Hydrostatic Pressures, and Uplift Pressures

Lateral earth pressure behind the proposed tide gate walls and headwalls is a function of the properties of the retained material and the type of wall (yielding or non-yielding). Based on our understanding of the proposed structures, we have provided at-rest static earth pressures for the tide gate structure and at-rest and active earth pressures for the headwalls. The lateral earth pressures recommended for use in tide gate and headwall design are presented in Figures 3 and 4, respectively. The lateral earth pressures for the headwalls (Figure 4) include options for a 3H:1V and 5H:1V dike slope behind the wall. The lateral earth pressures for the tide gate structure (Figure 3) include the 1.5 feet of soil cover on top of the tide gate. If additional soil cover is planned, these lateral earth pressures should be revised accordingly. Passive earth pressures are not provided due to the lack of soil cover, possible scour, and mobility of riprap.

For hydrostatic pressures, the phreatic surface within the dike fill can be assumed to vary linearly from the design high water elevation on the water side to the ground surface elevation at the toe on the landward side.

If the base is entirely in compression, uplift pressures will vary linearly from the hydrostatic pressure on the water side of the tide gate to zero on the landside of the tide gate. If the base is not entirely in compression, the uplift pressure on the portion of the base not in contact with the foundation material should be a uniform value, equal to the hydrostatic pressure at the adjacent face, in accordance with EM 1110-2-2100 (USACE, 2005b). Recommended uplift pressures on the tide gate are shown in Figure 5.

8 GEOTECHNICAL CONSTRUCTION CONSIDERATIONS

8.1 General

These construction considerations are provided as a general guide and are for constructing the Project dike setback. We recommend that construction earthwork activities be

performed during periods of dry weather, between June and September, when soil moisture can generally be controlled by aeration.

8.2 Clearing, Grubbing, and Stripping

Areas that will receive fills should be stripped to a depth that is sufficient to remove existing surface vegetation and organic soil, trunk (or stem), stump, root ball, roots greater than ½-inch-diameter, and any other deleterious materials encountered. This includes old foundations, slabs, pavements, footings, base rock, and other placed stone fill. The resulting voids should be backfilled with dike embankment materials and compacted to a density equal to the surrounding soil. Prior to backfill, the excavated areas should be free of standing water. We anticipate that the required stripping depths will be up to 12 inches in heavily vegetated areas. Stripped organic soils will not be suitable for re-use as fill for the dike embankment, but may be re-used as topsoil for plantings. In accordance with EP 1110-2-18 (USACE, 2019), we recommend removing all woody vegetation within 15 feet of the toe of the proposed dike footprint.

8.3 Subgrade Preparation

After clearing, grubbing, and stripping, the subgrade of the new dike embankment areas should be rolled and compacted using a tamping foot roller to breakup potential seepage paths and surfaces created during the clearing, grubbing, and stripping process. After areas are stripped or excavated to design subgrade elevations, we recommend that the subgrade be evaluated by the Geotechnical Engineer of Record, or their representative, for the presence of potentially soft or unsuitable soils. Areas that pump, heave, or appear soft or muddy should be scarified, dried, and compacted, or else over-excavated to an engineer-approved subgrade and backfilled with dike embankment materials. We recommend limiting any over-excavation to a depth of 2 feet below the surface of the stripped subgrade.

8.4 Geogrid and Geotextile

To meet design criteria for slope stability, reduce differential settlement, and improve constructability, we recommend installing a Miragrid 5XT geogrid (or equivalent) on the prepared subgrade surface, or at the bottom of the over-excavation (if performed), as well as at Elevations 7.5 feet, 10 feet, and 12.5 feet in the dike prism (see Exhibit 8-1). At a minimum, the basal geogrid should cover the entire dike footprint and will vary in elevation depending on the existing ground surface elevation, amount of clearing/grubbing, and amount of over-excavation. The additional layers of geogrid can be limited to the 2H:1V dike prism (i.e., are not required within the planting berm).

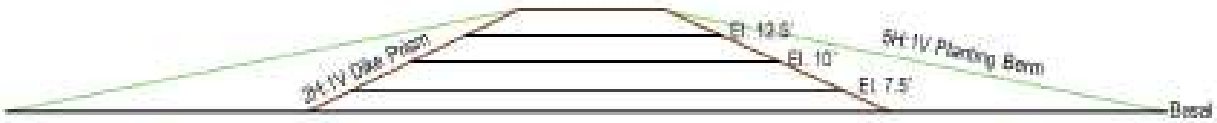


Exhibit 8-1: Recommended Geogrid Dike Cross-Section (not to scale)

In areas where the basal geogrid is within 6-inches (vertically) from the next horizontal geogrid, the horizontal geogrid can be terminated (see Exhibit 8-2).

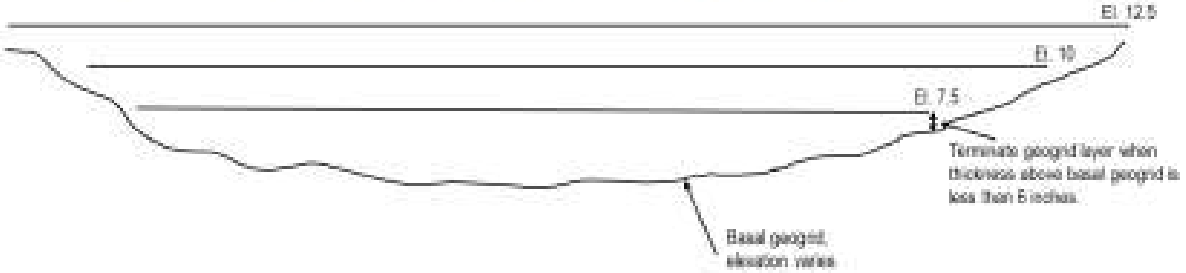


Exhibit 8-2: Recommended Geogrid Dike Profile (not to scale)

We recommend orienting geogrid soil reinforcements with the highest strength axis perpendicular to the dike alignment, and using a continuous length of geogrid perpendicular to the dike alignment. Adjacent rolls of geogrid (i.e., joints) should be overlapped by a minimum of 24 inches at the basal geogrid and 12 inches for the geogrid within the dike prism.

Embankment fill (cover) material should be placed on the geogrid such that a minimum compacted initial lift thickness of at least 6 inches is between the equipment tires or tracks and the geogrid at all times. Soil should not be dropped on the geogrid from a height greater than 3 feet. End-dumping the cover material directly on the geogrid should not be permitted. Turning of vehicles on the first lift above the geogrid should not be permitted.

We have reviewed the proposed tide gate structure foundation, including the layers of rock and geogrid proposed in the 60% plans. We recommend separating dissimilar (i.e., non-filter compatible) materials with nonwoven construction geotextile for soil stabilization, as shown in the 60% plans between the 1-1/4" crushed minus base course and the subgrade, and beneath the riprap. Adjacent sheets of construction geotextile for soil stabilization should be overlapped by a minimum of 2 feet when installed above water and a minimum of 3 feet when installed below water. As an alternative to overlapping panels, the adjacent geotextile panels could be sewn together with a minimum overlap of 6 inches above and below water. Additionally, we recommend separating the geotextile for separation and the bottom layer of geogrid with a minimum 3-inch-thick layer of 1-1/4-inch minus crushed base course. We recommend adding a minimum 9-inch-thick layer of 6"-0 stone embankment (ODOT OSSC 00330.16) between the geotextile and the riprap for a cushion

layer. Stone embankment and riprap should not be dropped from a height greater than 1 and 2 feet above the geotextile, respectively.

Geosynthetic (geogrid and geotextile) should be laid smooth without excessive wrinkles and should not be left exposed to sunlight during installation for a total of more than 14 calendar days.

8.5 Fill Placement on Slope

Fill placed on an existing slope that is steeper than 5H:1V should be keyed into the existing slope by benching the slope. Benches should be cut such that they are at least 12 inches high with a maximum height of 2 feet and of sufficient width to accommodate the compaction equipment. Each horizontal cut should begin at the intersection of the ground surface with the vertical side of the previous bench and should penetrate the existing slope at least 3 feet. The fill slope should be over-built by at least 2 feet and then cut back to grade to provide a compacted final slope.

8.6 Dike Embankment Materials

We recommend that dike embankment fill within the interior 2H:1V dike prism consist of imported material, either naturally occurring or processed, and free of debris, contaminants, invasive and noxious weeds, waste, frozen materials, wood, vegetation, and other deleterious matter. We recommend that the dike embankment fill meet the gradation requirements as shown in Exhibit 8-3, below, have a plasticity index (PI) of 4 or greater as determined by ASTM D4318, and an organic content less than or equal to 1% as determined by ASTM D2974.

Exhibit 8-3: Recommended Dike Embankment Fill Gradation, as determined by ASTM D6913

Sieve Size	Percent Passing, by Weight
2 inch	100
¾ inch	90 – 100
No. 4	80 – 100
No. 40	60 – 90
No. 200	50 – 80

For bidding purposes, we recommend that the Contractor assume all fill and backfill material will be imported. Other soils including Marsh Deposits and Fluvial and Estuarian Deposits, such as those from excavations on-site, may be acceptable in the 5H:1V planting berm, provided that they are filter compatible with the dike fill material and are free from organics. After laboratory testing data (soil gradation curves) are available from the proposed borrow source, Shannon & Wilson can provide a filter design band for the dike fill

material for comparison. We recommend that Shannon & Wilson be retained to perform laboratory testing and material review for all fill materials for compatibility. The excavated on-site soils will be wet of optimum. The excavated on-site soils may become unworkable because of excess moisture content. To reduce moisture content, discing and drying of the soils will likely be required.

8.7 Trench Drain Fill Materials

If a trench drain is installed at the landward toe of the dike setback as an underseepage mitigation measure, it will need to be backfilled with filter compatible materials for the surrounding subgrade soil. Filter compatible materials will help mitigate against the surrounding soil migrating into the trench drain and reducing its' functionality. A filter design band was developed for the Fluvial and Estuarian deposits based on applicable guidance in EM 1110-2-1913 (USACE, 2000) and FEMA (2011). The filter design is presented in Figure 6. ASTM C33 sand is a typical material that generally meets the filter design criteria. The portion of the filter material passing the No. 40 (0.425 mm) sieve must have a plasticity index (PI) of zero.

In order to relieve underseeapge pressures, the trench drain fill materials must be kept clean and free from topsoil. We recommend a 1-foot-thick Crushed Rock Trench Cap be placed on top of the trench drain, consisting of 3/8" to No. 8 gravel (ODOT OSSC, Section 02690). However, if topsoil and vegetation are desired on top of the trench drain, a nonwoven geotextile for separation would be required between the topsoil and trench drain fill materials, and a gravity-fed collector pipe would be required to allow for drainage. The trench drain width can be equal to one bucket width (typically 3 to 4 feet). We recommend full time geotechnical construction observation during trench drain construction to ensure that the trench drain fill completely penetrates the Marsh Deposits layer, with at least 1 foot overlap with the Fluvial and Estuarian Deposits below.

8.8 Fill Placement and Compaction

The dike embankment fill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 98 percent of the dry density as determined by the standard Proctor test, ASTM D698. We recommend that the moisture content of dike embankment fill be no more than 2 percent below or 2 percent above the optimum moisture content as determined by the Proctor test. At these moisture content ranges, it is generally easier to achieve the specified compaction.

If required, the trench drain backfill should be placed in loose lifts not exceeding 2 feet (below water) and 12-inches (above water). Each lift should be compacted with at least one continuous pass of an excavator plate compactor. The lift thicknesses, number of passes,

and size/type of plate compactor should be approved by the Geotechnical Engineer in the field. As shoring is removed, additional passes may be required to eliminate voids caused by shoring removal. The Crushed Rock Trench Cap should be placed in loose lifts not exceeding 12-inches and compacted with at least 2 passes of approved compaction equipment such as vibrating plate compactors or smooth drum rollers. The final surface of crushed rock shall be mounded over the trench at least 6 inches above grade to account for future settlement.

8.9 Temporary Cut Slopes

Temporary excavations should be constructed in accordance with Occupational Safety and Health Administration (OSHA) standards and applicable state and local regulations. The soils at the site classify as OSHA Type C, and excavations extending into these soils should be cut on a slope no steeper than 1.5H:1V. Flatter slopes may be required, and all operations should be performed under the supervision of qualified site personnel in accordance with OSHA regulations. Any excavation slope left exposed should be protected from erosion and saturation by rainfall and runoff. Adequately designed bracing may be used to support excavations. Dewatering of soil prior to or in conjunction with excavations may be required. Temporary excavations extending below approximately 15 feet may need additional analysis.

8.10 Wet Weather Earthwork and Erosion Control

Wet weather generally begins about mid-October and continues through about May, although rainy periods may occur at any time of year. Therefore, it would be advisable to schedule earthwork during the drier weather months of June through September. The soils expected at the dike setback location are mostly granular; however, if Marsh Deposits are present, they could contain significant amounts of silt and organics, making them moisture sensitive. Marsh Deposits would likely not be suitable for a working platform and may be unstable when wet and under equipment loads.

Performing earthwork during dry weather would reduce the problems and costs associated with rainwater, trafficability, and handling of wet soil. However, should wet weather/wet condition earthwork be unavoidable, we recommend the following:

- The ground surface in and surrounding the construction area should be sloped as much as possible to promote runoff of precipitation away from work areas to prevent ponding of water.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soil and placement and compaction of fill can be accomplished on the same

day. The size of construction equipment may have to be limited to prevent soil disturbance.

- In-place soil or fill that becomes wet and unstable and/or too wet to suitably compact should be removed and replaced.
- No soil should be left uncompacted and exposed to moisture.
- Excavation and placement of fill material should be observed on a full-time basis by a geotechnical engineer (or representative) experienced in earthwork to determine that all work is being accomplished in accordance with the Project specifications.
- Grading and earthwork should not be performed during periods of heavy, continuous rainfall.

The contract documents should include provisions for wet weather/wet condition earthwork. These provisions should include requirements for repairing damage to dike embankments from haul and construction equipment causing soil pumping and rutting.

The near-surface soils observed at the site are erosion susceptible. Erosion control for the site should include Best Management Practices incorporated into the Project Plans and may include the following recommendations:

- Limit exposed cut slopes.
- Route surface water through temporary drainage channels around and away from exposed slopes.
- Use silt fences, straw, and temporary sedimentation ponds to collect and hold eroded material at the site.
- Seed or plant vegetation on exposed areas where grading work is complete. If seeding/mulching is performed to control erosion, it should be in place early enough to allow significant vegetation growth before the start of the wet, winter season.
- Retain existing vegetation to the greatest possible extent.

8.11 Plantings

We understand that ESA prefers to include plantings, including forbs and shrubs, along the slopes of the dike setback. In our opinion, this is acceptable, so long as the root systems from plantings on the 5H:1V planting berm do not encroach on the interior 2H:1V dike prism. We recommend carefully reviewing the seed mixes and plantings proposed on the plans, and the locations of the plantings, to ensure that the dike prism remains free of roots. Further, we recommend that plantings within a minimum of 10 horizontal feet from the dike crest are limited to a mowable seed mix (preferably consisting of native grasses). This will be beneficial to maintain dike access for community use, allow for inspections and

maintenance, and provide long-term dike stability (by reducing the potential for piping, seepage, and structural damage due to root systems).

8.12 Temporary Construction Drainage

We believe temporary site and construction drainage can be reasonably well controlled by careful excavation practices. Typically, these include, but are not limited to, shallow upgrade perimeter ditches or low earthen berms and temporary sumps in excavations to collect seepage and prevent water from damaging exposed subgrade. Drains should be included at the bottom of all temporary slopes to collect surface water flow from the slope and prevent it from flowing onto exposed subgrade. All runoff water and groundwater encountered should be collected and drained outside the construction limits.

Groundwater should be expected to be encountered in the excavation for the dike setback. The design of the temporary dewatering systems is traditionally the responsibility of the Contractor. Any water collected during dewatering and any excavated soil should be treated and disposed of in a manner meeting local, state, and federal environmental regulations and requirements.

8.13 Pipe Penetrations

All new pipe penetrations should be designed in accordance with the requirements of Chapter 8, Section I, of EM 1110-2-1913 (USACE, 2000). This section specifies allowable materials, installation methods, and anti-seepage devices.

To meet the requirements of Section 8.5, the landside one-third of the pipe should be encased in drainage material. The gradation drainage material is determined on a case-by-case basis and should be filter compatible with the surrounding soil.

8.14 Construction Monitoring Requirements

Fill placement and checking of the exposed subgrade should be monitored by the Geotechnical Engineer of Record, or their representative, to verify that unsuitable materials are not present, that proper materials are used for fill, and that placement and compaction of materials has been accomplished. Before fill and backfill operations begin, representative samples of the proposed fill and backfill material should be tested for determination of laboratory compaction characteristics in accordance with ASTM D698, as recommended above. Laboratory gradation tests and Atterberg Limits tests should be accomplished in accordance with ASTM D6913 and ASTM D4318, respectively, to check material classification.

Compaction of subgrade surfaces, fill, and backfill should be checked with a sufficient number of density tests to ensure that adequate compaction is being achieved. Construction specifications should require at least one in-place density test of the compacted fill for every 2,000 square feet of fill placed in the dike prism and every 5,000 square feet of fill placed elsewhere. For backfill of utility trenches or around structures, construction specifications should require at least one in-place density test per lift for every 100 feet. At least one test should be completed per lift regardless of the size or location of the fill area. Where density tests failed minimum values, retesting should be performed after mitigation.

8.14.1 Settlement Monitoring Program

We recommend settlement monitoring plates be installed along the dike setback alignment, embedded in the subgrade and located within the dike crest footprint, at 200-foot spacing. Additionally, we recommend installing four settlement monitoring plates at the tide gate location, with two evenly spaced on each side of the proposed tide gate location. A typical detail of a settlement plate is shown on Figure 7. The riser pipes connecting to the settlement plate should be tied with visible warning flags or ribbons. We recommend that surveying of the settlement monitoring points be performed once a week during construction, once a month between the first and second seasons of construction, and once every three months post-construction for at least a year. Settlement monitoring data should be provided to Shannon & Wilson so that we can update our long-term settlement predictions, and confirm that preloading of the tide gate structure is complete.

8.15 Geotechnical Support During Construction

We recommend that Shannon & Wilson be retained to perform full-time construction observation of the dike setback including field density testing to check dike subgrade, thickness of Marsh Deposits if encountered, provide timely materials testing results during construction, and provide on-site consultation.

9 ADDITIONAL FIELD EXPLORATION AND EVALUATIONS PRIOR TO CONSTRUCTION

A geotechnical field exploration program for the dike setback will be performed prior to construction to confirm the assumed subsurface conditions along with the dike setback and our design. The proposed field exploration program for the dike setback consists of three (3) borings extending to a depth of 60 feet, as shown on Site and Exploration Plan, Figure 2. We recommend that we be on site during the drilling to log the boreholes, characterize the subgrade soil, and collect samples for laboratory testing. Having us on site will allow for

real-time interpretation if additional exploration or engineering analyses are required. Additional geotechnical services may include re-design and updating plans and specifications.

10 LIMITATIONS

This report was prepared by Shannon & Wilson for the Sitka Sedge Tidal Wetland Restoration (Dike Setback), for the exclusive use of ESA and Oregon Parks and Recreation Department. Agencies and contractors that reference this report should rely on factual data only, and not as a warranty of subsurface conditions, such as those interpreted from observations of the site or information we reviewed. The engineering analyses and recommendations were performed based on assumed subsurface conditions. This report will be updated after new field explorations for dike setback are completed. This report shall not be used for construction.

Interpretations, analyses, conclusions, and recommendations contained in this report are based on our observation of site conditions as they existed during our site visits and our interpretation of assumed subsurface conditions based on explorations we performed for the bridges and culverts; subsurface exploration logs prepared by others; and geologic and hydrogeologic data for the Project site. The professional opinions, recommendations, and conclusions contained in this report for the dike system are valid for a period not greater than 10 years from the date of this report. This time limitation is included in recognition that the conditions of dike systems can and do change with time as do the conditions that lead to water surface elevation determinations. If new information becomes available to Shannon & Wilson, the professional opinions, recommendations, and conclusions contained in this report may be modified by Shannon & Wilson.

Our analyses, and resulting conclusions and recommendations, rely on data provided by others, including, but not limited to, survey data, subsurface data, dike geometry information (plans and cross sections), and design flood water surface elevations. Shannon & Wilson makes no warranty, express or implied, as to the accuracy of the data relied on. Within the limitations of the scope, schedule, and budget of this project, the analyses, conclusion, and recommendations presented in this report were performed in accordance with generally accepted professional engineering principles and practices in use in the area of the Project at the time this report was prepared.

We assume that our interpretations of subsurface conditions are representative of subsurface conditions at the site. Due to the Project schedule, borings at the dike setback have not yet been performed. We recommend performing explorations at the dike setback

as soon as practical so that we can confirm the analyses and recommendations presented herein.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples, drilling test borings, pushing probes, or excavating test pits. If subsurface conditions different from our observations or interpretation are encountered or appear to be present, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

If there is a substantial lapse of time between the issuance of this report, or if conditions have changed due to natural causes or construction at or near the site, we recommend that site conditions and this report be reviewed to determine the applicability of the conclusions and recommendations.

The scope of our present work did not include environmental assessments or evaluations regarding the presence or absence of wetlands, or hazardous or toxic substances in the soil, surface water, groundwater, or air, on or below or around this site, or for the evaluation or disposal of contaminated soils or groundwater should any be encountered.

Shannon & Wilson has prepared and included "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our reports.

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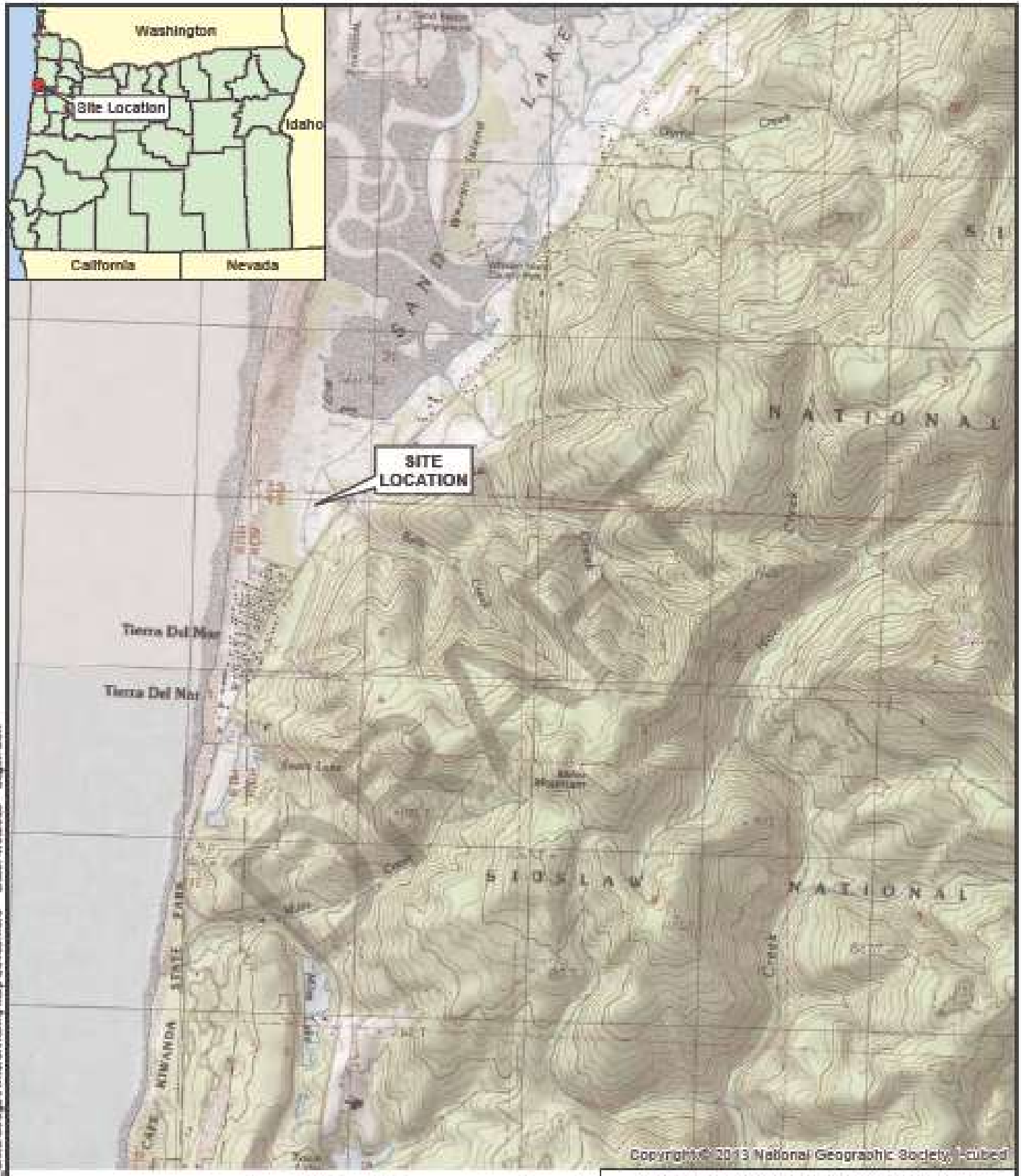
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0 0.25 0.5 1

Scale in Miles

N

Siitka Sedge Tidal Wetland Restoration
Dike Setback
Tillamook County, Oregon

VICINITY MAP

April 2026 112880

SHANNON & WILSON FIG. 1

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Filename: P:\2026\P009\112880\r\112880_Stiss Sedge\Arcmxd\Site Plan_2026.mxd Date: 4/13/2026 Login: ash



LEGEND

B-1 Designation and Approximate Location of Boring

B-8 Designation and Approximate Location of Proposed Boring to Depth of 80 feet

Proposed Setback Dike Centerline

NOTES

1. Aerial imagery obtained through Google Maps Satellite.
2. Existing contours derived from 2018 LiDAR data obtained through DOGAMI.
3. Proposed features from files provided by Environmental Science Associates on June 30, 2025.

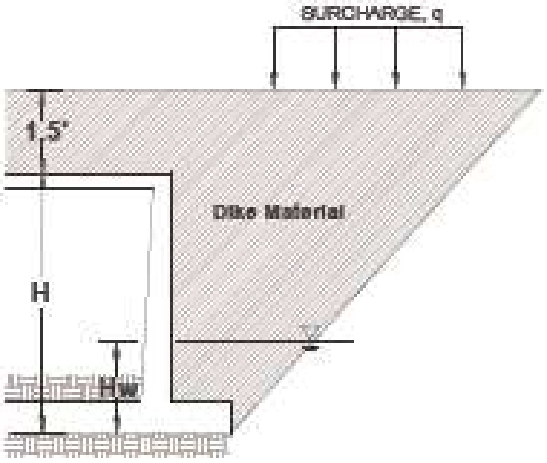
Stiss Sedge Tidal Wetland Restoration
Dike Setback
Tillamook County, Oregon

SITE AND EXPLORATION PLAN

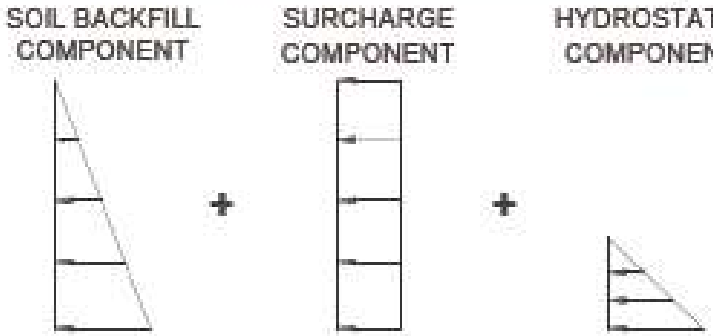
April 2026 112880

SHANNON & WILSON FIG. 2

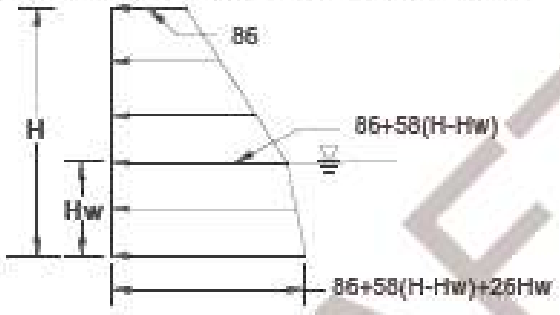
FIG. 2



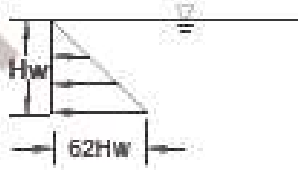
TOTAL LATERAL EQUIVALENT FLUID PRESSURES



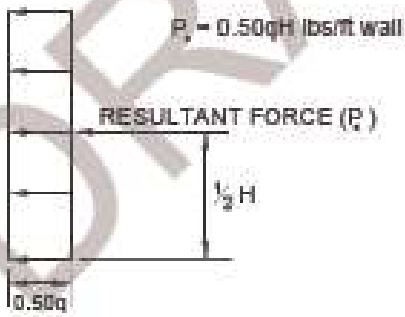
NON-YIELDING WALL SOIL COMPONENT



HYDROSTATIC COMPONENT



NON-YIELDING WALL SURCHARGE COMPONENT



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NOTES

1. Units are pounds per square foot (psf).
2. Backfill unit weight of 115 pcf.
3. Backfill friction angle is estimated as 30 deg.
4. Retained wall backfill is assumed to be dike material.
5. Wall design will be performed by others.

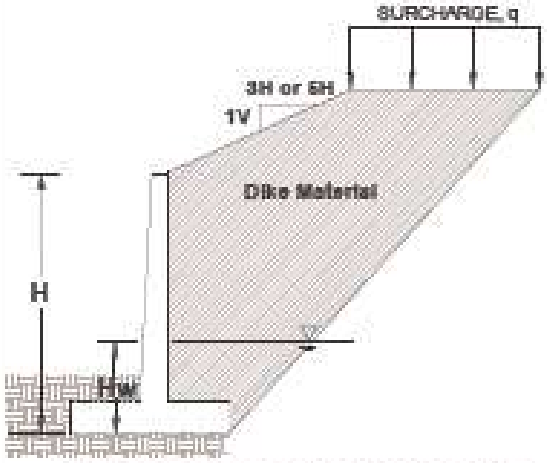
Sitka Sedge Tidal Wetland Restoration
Dike Setback
Tillamook County, Oregon

LATERAL EARTH PRESSURE DISTRIBUTION ON TIDE GATE

April 2026 112880

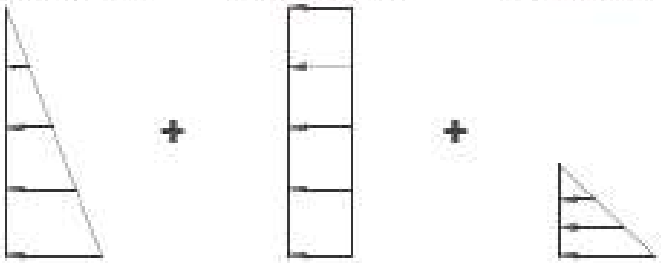
SHANNON & WILSON | FIG. 3

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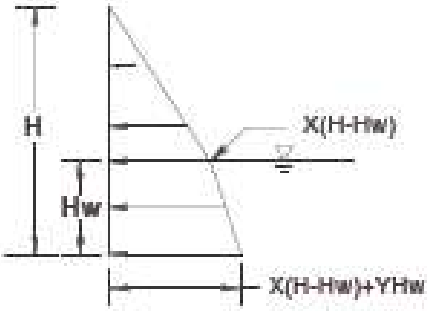


TOTAL LATERAL EQUIVALENT FLUID PRESSURES

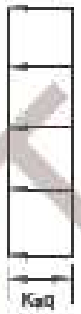
SOIL BACKFILL COMPONENT + SURCHARGE COMPONENT + HYDROSTATIC COMPONENT



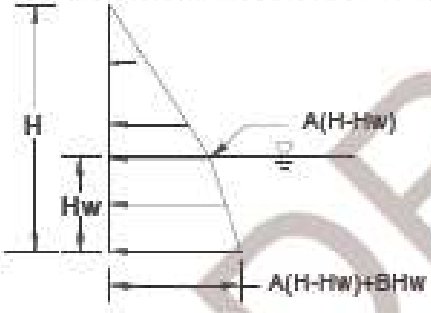
YIELDING WALL SOIL COMPONENT



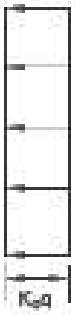
YIELDING WALL SURCHARGE COMPONENT



NON-YIELDING WALL SOIL COMPONENT



NON-YIELDING WALL SURCHARGE COMPONENT



HYDROSTATIC COMPONENT

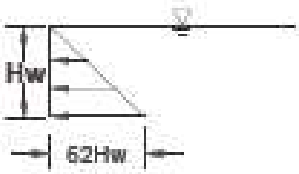


TABLE 1		
BACKSLOPE:	5H:1V	3H:1V
X	44	49
Y	20	22
A	69	75
B	32	34
K_a	0.38	0.42
K_o	0.60	0.65

NOTES

1. Units are pounds per square foot (psf).
2. Backfill unit weight of 115 pcf.
3. Backfill friction angle is estimated as 30 deg.
4. Retained wall backfill is assumed to be dike material.
5. Wall design will be performed by others.

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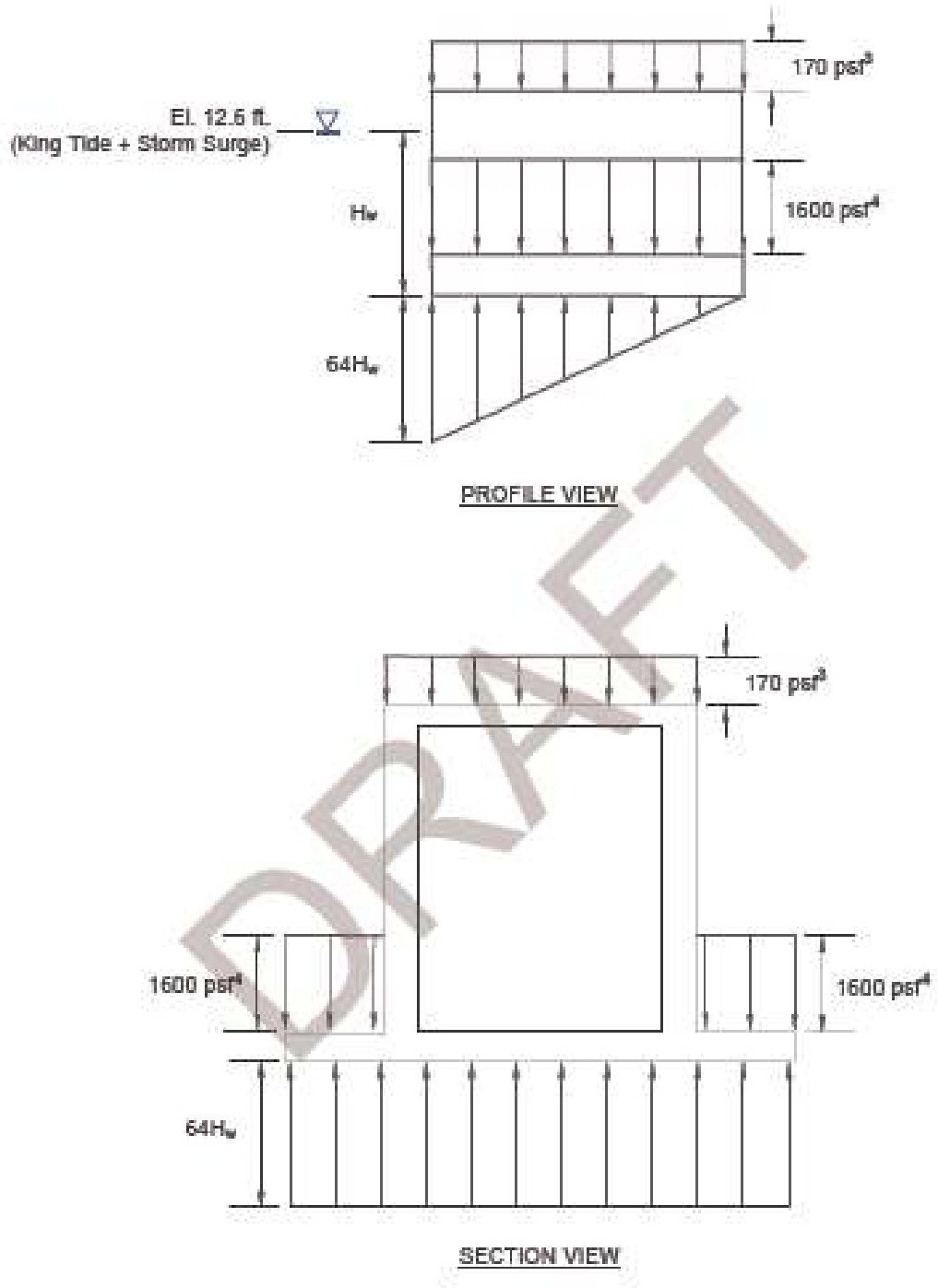
Sitka Sedge Tidal Wetland Restoration
Dike Setback
Tillamook County, Oregon

LATERAL EARTH PRESSURE DISTRIBUTION ON TIDE GATE WING WALLS

April 2026 112880

SHANNON & WILSON | FIG. 4

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NOTES

1. Units are pounds per square foot (psf).
2. Uplift pressures shown assume the foundation is entirely in compression.
3. Assumes 1.5 feet of overburden soil.
4. Assumes 14 feet of overburden soil.

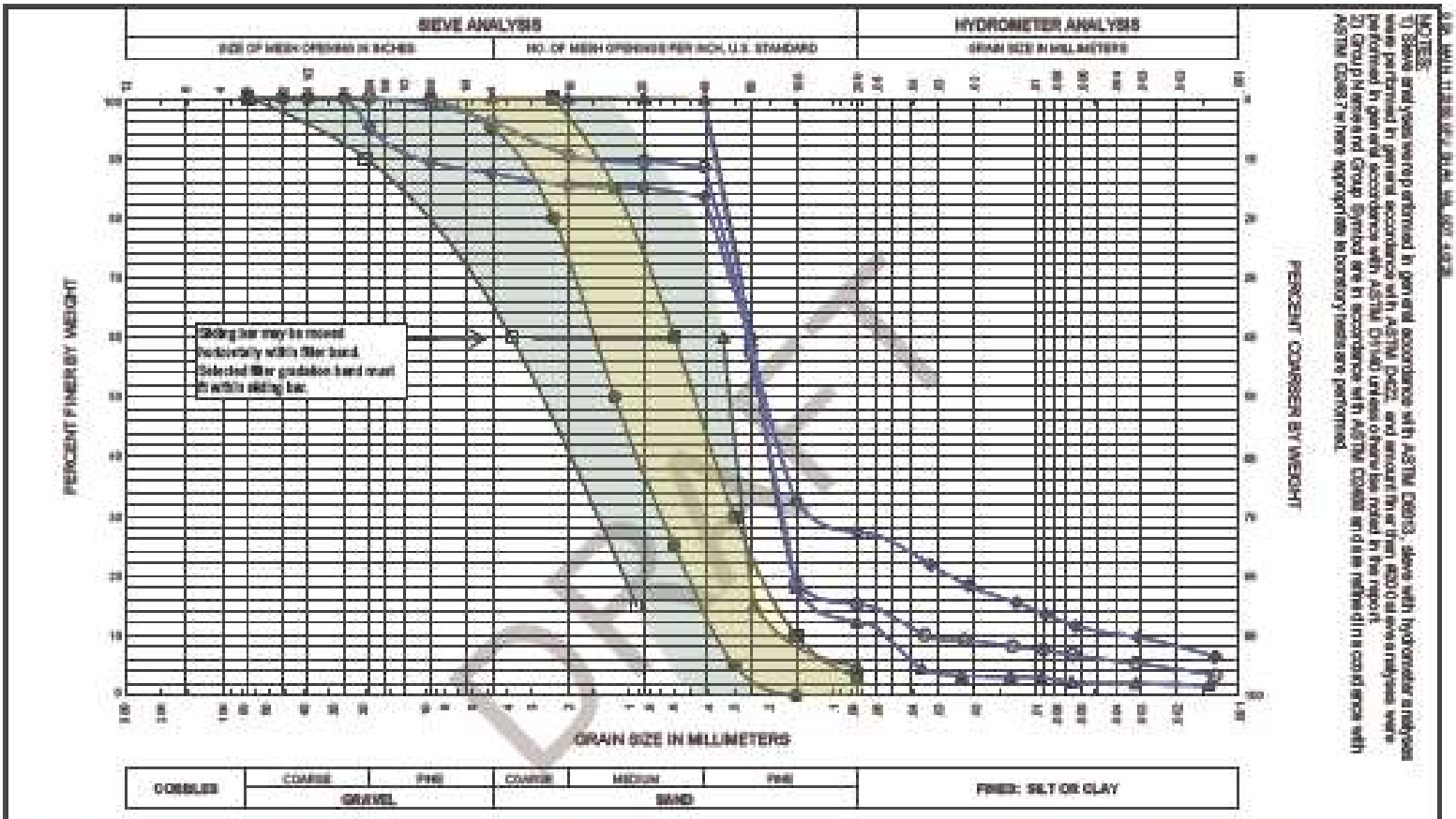
Sitka Sedge Tidal Wetland Restoration
Dike Setback
Tillamook County, Oregon

UPLIFT PRESSURES ON TIDE GATE

April 2026 112880

SHANNON & WILSON FIG. 5

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NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913, sieve with hydrometer analyses were performed in general accordance with ASTM D422, and percent fines (#200 sieve) analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Names and Group Symbol are in accordance with ASTM D2487 and are determined using laboratory tests as performed.

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
ASTM C33, UB	0.0		ASTM C33 Sand (Possible Filter Material)	-				
ASTM C33, LB	1.0			-	-	3		
B-2, S-4	10.0	SM	Silty Sand	-	88	12	21	
B-2, S-6	20.0	SM	Silty Sand	13	60	27	25	
B-7, S-6	20.0	SM	Silty Sand	4	81	15	20	
Filter Design, UB	0.0			-	-			
Filter Design, LB	1.0		Filter Design Band	-	-	5		

Fluvial & Estuarian Deposits (Base Soil)

Silka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

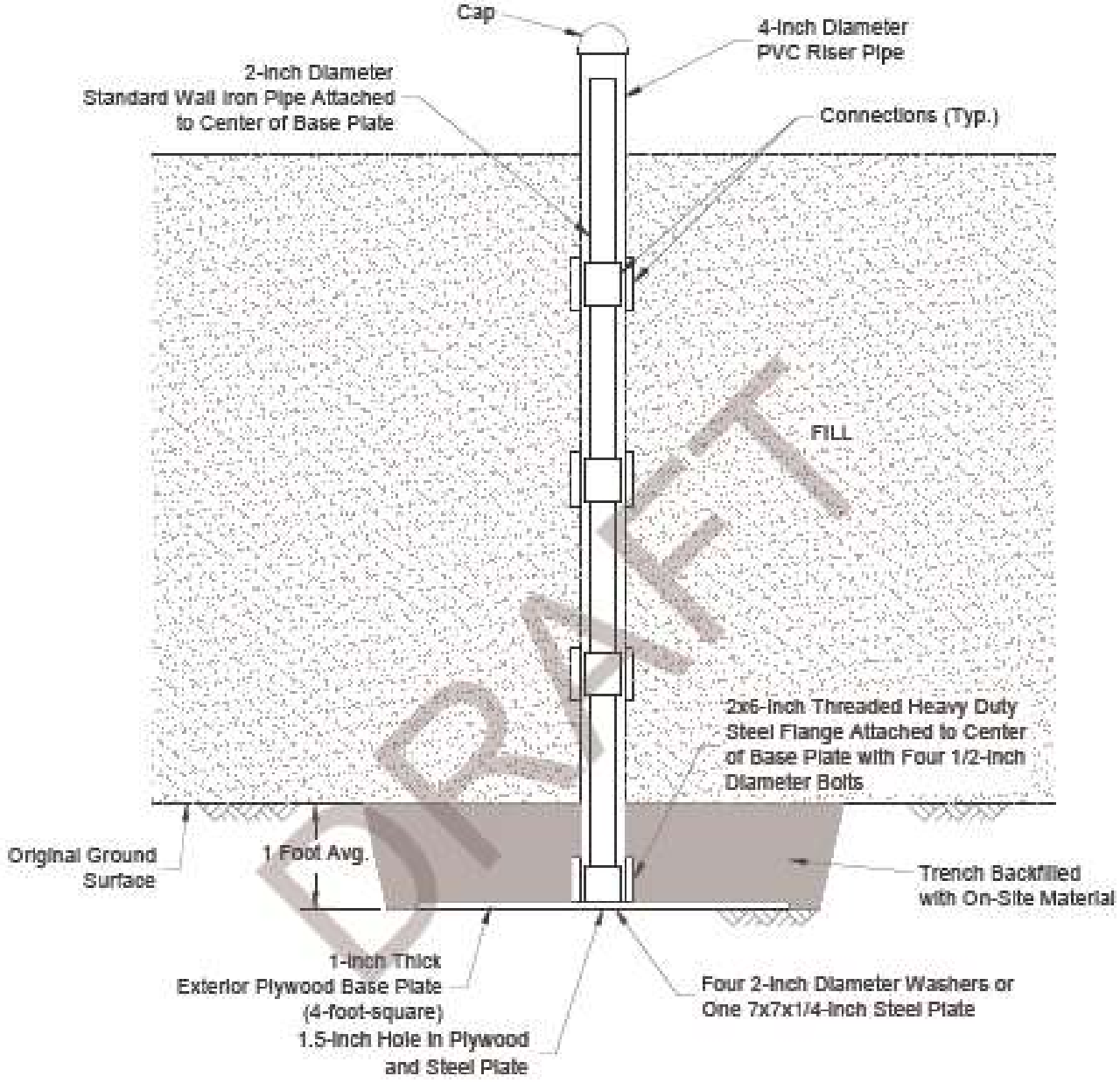
GRAIN SIZE DISTRIBUTION
Trench Drain Filter Design

April 2026 112880

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FIG. 6

FIG. 6



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Sitka Sedge Tidal Wetland Restoration
Dike Setback
Tillamook County, Oregon

TYPICAL SETTLEMENT PLATE DETAIL

April 2026 112880

SHANNON & WILSON FIG. 7

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APPENDIX A: PRELIMINARY DESIGN DRAWINGS

Appendix A

Preliminary Design Drawings

CONTENTS

- **Sitka Sedge Tidal Wetland Restoration, 60% Design - Preliminary (ESA, 2025)**

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TILLAMOOK ESTUARIES PARTNERSHIP

SITKA SEDGE TIDAL WETLAND RESTORATION

60% DESIGN - PRELIMINARY - NOT FOR CONSTRUCTION

PROJECT LOCATION: TIERRA DEL MAR, OREGON

SHEET LIST

Sheet Number	Sheet Title
G1.0	COVER
G1.1	GENERAL NOTES, ABBREVIATIONS, AND LEGEND
G1.2	EXISTING CONDITIONS AND SURVEY CONTROL
G1.3	SITE PLAN AND OVERVIEW
G1.4	ACCESS, STAGING, PERMIT, AND PROTECTION
G1.5	BRIDGE CREEK EAST NO. 1 CONSTRUCTION & SURVEY CONTROL
G1.6	CONSTRUCTION LOCATION - PEDESTRIAN BRIDGE
G1.7	CONSTRUCTION LOCATION - WETLAND UNIT
C1.1	SITE GRADING PLAN AND NOTES
C1.2	SITE GRADING CHANGE PROFILE
C1.3	CHANNEL ADJUST NO. 1 GRADING PLAN AND PROFILE - 1
C1.4	CHANNEL ADJUST NO. 1 GRADING PLAN AND PROFILE - 2
C2.1	CHANNEL GRADING AND TYPICAL SECTIONS
C2.2	BRIDGE CREEK GRADING PLAN
C2.3	BRIDGE CREEK CHANNEL PROFILES
C2.4	BRIDGE CREEK GRADING SECTIONS
C2.5	CHART OF CUT AND FILL

BRIDGE CREEK BRIDGE SHEETS

S0.1	BRIDGE BRIDGE PLAN AND ELEVATION
S0.2	BRIDGE BRIDGE GENERAL NOTES
S0.3	BRIDGE BRIDGE FOUNDATION PLAN
S0.4	BRIDGE BRIDGE DECK DETAILS
S0.5	BRIDGE BRIDGE RAILING DETAILS
S0.6	BRIDGE BRIDGE ABUTMENT DETAILS

PEDESTRIAN BRIDGE SHEETS

S0.0	PEDESTRIAN BRIDGE PLAN AND ELEVATION
S0.1	PEDESTRIAN BRIDGE GENERAL NOTES
S0.2	PEDESTRIAN BRIDGE FOUNDATION PLAN
S0.3	PEDESTRIAN BRIDGE DECK DETAILS
S0.4	PEDESTRIAN BRIDGE RAILING DETAILS

C1.1	SETBACK AND PLAN APPROVE
C1.2	SETBACK AND PLAN ENLARGEMENT B
C1.3	SETBACK AND SECTIONS
C1.4	TIDE GATE STRUCTURE PLAN AND PROFILE
C1.5	TIDEGATE DETAIL
C1.6	SMALL BRIDGE CREEK
C1.7	BRIDGE CREEK EAST NO. 1 CONTROL PLAN OVERVIEW
C1.8	BRIDGE CREEK EAST NO. 1 CUT AND FILL 1
C1.9	BRIDGE CREEK EAST NO. 1 CUT AND FILL 2
C1.10	STAKE PLAN NOTES
C1.11	STAKE PLAN SURVEY OVERVIEW
C2.1	STAKE PLAN SHEET 1
C2.2	STAKE PLAN SHEET 2
C2.3	STAKE PLAN SHEET 3
C2.4	PLANT MIX SCHEDULE
C2.5	PLANT MIX SCHEDULE
C2.6	PLANT MIX SCHEDULE
C2.7	PLANT MIX SCHEDULE
C2.8	TYPICAL PLANTING PROFILES
C2.9	PLANT MIX DETAILS
C2.10	PLANT MIX DETAILS



VICINITY MAP

PROJECT TEAM

DRAFT

Dig|Safely.

ESSA

SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

COVER

G1.0

GENERAL CONSTRUCTION

1. ALL WORK SHALL CONFORM TO THE LATEST VERSION OF TILLAMOOK COUNTY CONSTRUCTION STANDARDS AND SPECIFICATIONS, AND ALL APPLICABLE LOCAL AND STATE CODES AND REGULATIONS. WHERE CONFLICTS EXIST BETWEEN THE REQUIREMENTS OF THESE PLANS AND THE REQUIREMENTS OF THE APPLICABLE CODES, STANDARDS OR REGULATIONS, THE MORE STRINGENT REQUIREMENT SHALL PREVAIL.
2. A COPY OF THE APPROVED PLANS SHALL BE KEPT ON SITE DURING CONSTRUCTION AT ALL TIMES.
3. THE CONTRACTOR SHALL NOT CHANGE, REVISE, OR VARY THE APPROVED PLANS WITHOUT THE EXPRESS WRITTEN APPROVAL OF THE ENGINEER AND ANY AFFECTED AGENCY.
4. THE CONTRACTOR SHALL OBTAIN AND PAY ALL REQUIRED CONSTRUCTION PERMITS AND APPROVALS PRIOR TO STARTING CONSTRUCTION.
5. LOCATIONS OF KNOWN AND/OR SUSPECTED UNDERGROUND FACILITIES AS SHOWN ON THE PLANS ARE APPROXIMATE ONLY. THE CONTRACTOR SHALL DETERMINE THE EXACT LOCATION OF ALL EXISTING UTILITIES BEFORE BEGINNING ANY WORK. THE CONTRACTOR SHALL BE RESPONSIBLE FOR ANY DAMAGE TO EXISTING UTILITIES, STRUCTURES, IMPROVEMENTS, AND PROPERTY RESULTING FROM THE CONTRACTOR'S OPERATIONS.
6. CONSTRUCTION NOISE SHALL BE LIMITED IN ACCORDANCE WITH LOCAL CODE.
7. ARCHEOLOGICAL RESOURCES: IF ANY ARCHEOLOGICAL RESOURCES OR HUMAN REMAINS ARE DISCOVERED DURING CONSTRUCTION, ALL WORK IN THE IMMEDIATE VICINITY SHALL STOP IMMEDIATELY AND THE CONTRACTOR SHALL NOTIFY THE ENGINEER, THE SHPO, AND THE APPROPRIATE TRIBAL GOVERNMENT REPRESENTATIVE. WORK SHALL NOT RESUME UNTIL AUTHORIZED BY THE ENGINEER AND ALL REQUIRED CONSULTATIONS ARE COMPLETE. IF HUMAN REMAINS ARE DISCOVERED, THE CONTRACTOR SHALL ALSO NOTIFY THE COUNTY MEDICAL EXAMINER AND LAW ENFORCEMENT IMMEDIATELY.
8. CONTACT THE OREGON UNDERGROUND UTILITY NOTIFICATION CENTER (811) AT LEAST TWO BUSINESS DAYS BEFORE BEGINNING EXCAVATION. THE CONTRACTOR SHALL ENSURE THAT ALL UTILITIES WITHIN THE CONSTRUCTION AREA ARE LOCATED AND MARKED BEFORE ANY EXCAVATION BEGINS. THE CONTRACTOR SHALL NOTIFY ALL UTILITY OWNERS WITHIN THE CONSTRUCTION AREA AT LEAST 48 HOURS PRIOR TO THE START OF CONSTRUCTION. THE CONTRACTOR SHALL MAINTAIN ACCURATE RECORDS OF THE LOCATIONS OF ALL UTILITIES ENCOUNTERED DURING CONSTRUCTION AND SHALL SUBMIT THESE RECORDS TO THE ENGINEER UPON COMPLETION OF THE WORK.
9. ALL UTILITIES WITHIN OR ADJACENT TO THE CONSTRUCTION AREA SHALL BE PROTECTED IN PLACE UNLESS OTHERWISE NOTED ON THE PLANS. THE CONTRACTOR SHALL NOTIFY THE ENGINEER IMMEDIATELY IF ANY UTILITY CONFLICTS WITH THE PROPOSED WORK OR IF ANY UTILITY APPEARS TO BE DAMAGED DURING CONSTRUCTION. THE CONTRACTOR SHALL NOT PROCEED WITH ANY WORK AFFECTING A UTILITY UNTIL DIRECTED BY THE ENGINEER.
10. REMOVE ALL DEBRIS AND EXCESS MATERIALS FROM THE SITE AND DISPOSE OF THEM IN A LEGAL MANNER AT AN APPROVED DISPOSAL FACILITY.

SITE WORK

1. THE CONTRACTOR SHALL BE RESPONSIBLE FOR ENSURING ADEQUATE DRAINAGE AND EROSION CONTROL DURING CONSTRUCTION. THE CONTRACTOR SHALL NOT ALTER DRAINAGE PATTERNS IN A MANNER THAT CAUSES WATER TO POND ON ADJACENT PROPERTIES OR TO FLOW ONTO PUBLIC ROADS IN QUANTITIES EXCEEDING THE CAPACITY OF THE EXISTING DRAINAGE SYSTEM.
2. THE CONTRACTOR SHALL BE RESPONSIBLE FOR SITE SECURITY, INCLUDING PROTECTION OF WORK AND SITE AGAINST VANDALISM, THEFT, AND UNAUTHORIZED ACCESS.
3. THE CONTRACTOR SHALL BE RESPONSIBLE FOR MAINTAINING THE INTEGRITY OF ADJACENT UTILITIES DURING ALL WORK, INCLUDING, BUT NOT LIMITED TO, WATER, POWER, COMMUNICATIONS.
4. THE CONTRACTOR SHALL NOTIFY PROPERTY OWNERS IN ADVANCE OF ANY WORK AFFECTING ACCESS OR SERVICES, AND SHALL MINIMIZE DISRUPTION TO DRIVEWAYS FOR RESIDENTS AND BUSINESSES ADJACENT TO THE PROJECT.
5. ALL SIGNS AND POSTED AREA DEFINED WILL BE RESTORED TO ORIGINAL CONDITION.
6. DISTURBANCE OR DAMAGE TO OTHER PROPERTY OR ADJACENT PARCELS OUTSIDE THE PUBLIC RIGHT OF WAY SHALL ALSO BE REPAIRED OR RESTORED BY THE CONTRACTOR.
7. THE CONTRACTOR SHALL REMOVE, ADJUST, AND REPLACE AT NECESSARY ALL EXISTING SIGNS AS DIRECTED.
8. TRENCH EXCAVATIONS SHALL NOT BE LEFT OPEN OVERNIGHT.
9. CONNECT NEW DOWNSPOUTS TO EXISTING AS SHOWN ON THE PLANS OR AS DIRECTED BY THE ENGINEER.
10. REPAIR ALL OTHER DAMAGE TO PARK AREA THAT ARE NOT SHOWN AS REMOVED OR REPLACED.
11. PROVIDE AN ADEQUATE LOCATION AS NEEDED FOR ALL STAGING AND STORAGE AREAS, AS DETERMINED BY KATE & HOT SPOT METHOD.

TIMING FOR IN-WATER WORK

1. THE CONTRACTOR SHALL ADHERE COMPLIANCE WITH PROJECT PERMITS AND OBTAIN THE APPROPRIATE IN-WATER WORK WINDOW FOR BANDLAKE / TIDE GATES, AND OTHER AREAS:

   BANDLAKE EXCAVATION: DECEMBER 1ST - FEBRUARY 15TH

   BANDLAKE PLANTINGS: JULY 1ST - SEPTEMBER 30TH

   ADDITIONAL WORK CLOSURES TO OTHER PERMITS SHALL BE AT DISCRETION OF THE ENGINEER. WORK WINDOW CHANGES REQUIRED BY USACE AND OTHER AGENCIES MAY REQUIRE REVISIONS.

ABBREVIATIONS

ATTN'D	ATTENDANTS
AVG	AVERAGE/AVERAGING/AVGS
BM	BENCHMARK
BMP	BEST MANAGEMENT PRACTICE
CONC	CONCRETE
CONST	CONSTRUCTION
CY	CUBIC YARD
DBH	DIAMETER AT BREAST HEIGHT
DEQ	DEPARTMENT OF ENVIRONMENTAL QUALITY
DW	DOWNSPOUT
EA	EACH
EC	EROSION & SEDIMENT CONTROL
ESC	ENHANCED ESTUARINE CONNECTION
EW	EXISTING WATER
EX	EXISTING
FT	FEET
NAVD88	NORTH AMERICAN VERTICAL DATUM OF 1988
GY	GYPSUM
HT	HEIGHT
HWM	HIGH WATER MARK
INV	INVERT
LWM	LOW WATER MARK
MAX	MAXIMUM
MIN	MINIMUM
NOAA	NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
NRCS	NATURAL RESOURCES CONSERVATION SERVICE
OHW	ORDINARY HIGH WATER
PC	POINT OF CURVATURE
PCS	PRECAST CONCRETE SYS
PI	POINT OF INTERSECTION
PVC	POLYVINYL CHLORIDE
RCP	REINFORCED CONCRETE PIPE
RD	ROAD
SF	SQUARE FEET
SHT	SHEET
SPEC	SPECIFICATION
SQ	SQUARE
STA	STATION
TC	TOP OF CURB
TYP	TYPICAL
UNO	UNLESS NOTED OTHERWISE
W	WIDTH

DRAFT

ESSA

SITKA SEDGE TIDAL WETLAND RESTORATION

ISSUED	
#	DATE

DESIGNED: XXX
DRAWN: XXX
CHECKED: XXX
REVIEWED: XXX
APPROVED: XXX

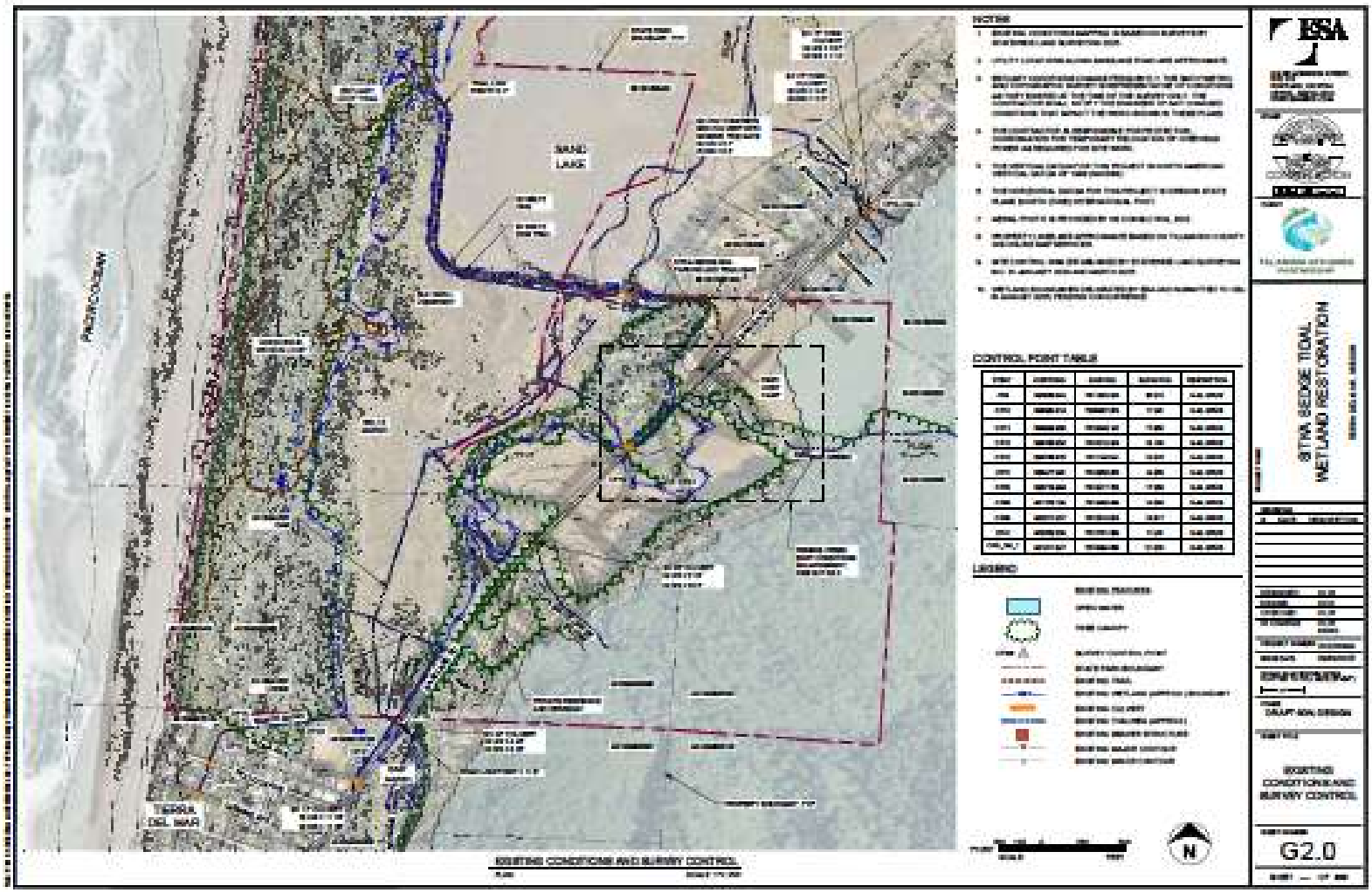
PROJECT NO. 0000.00
ISSUE DATE: MM/DD/YY

DRAFT 60% DESIGN

GENERAL NOTES, GROUNDS, AND ABBREVIATIONS

G1.1

SHEET ## OF ##



NOTES

CONTROL POINT TABLE

POINT	NORTHING	EASTING	ELEVATION	DESCRIPTION

LEGEND

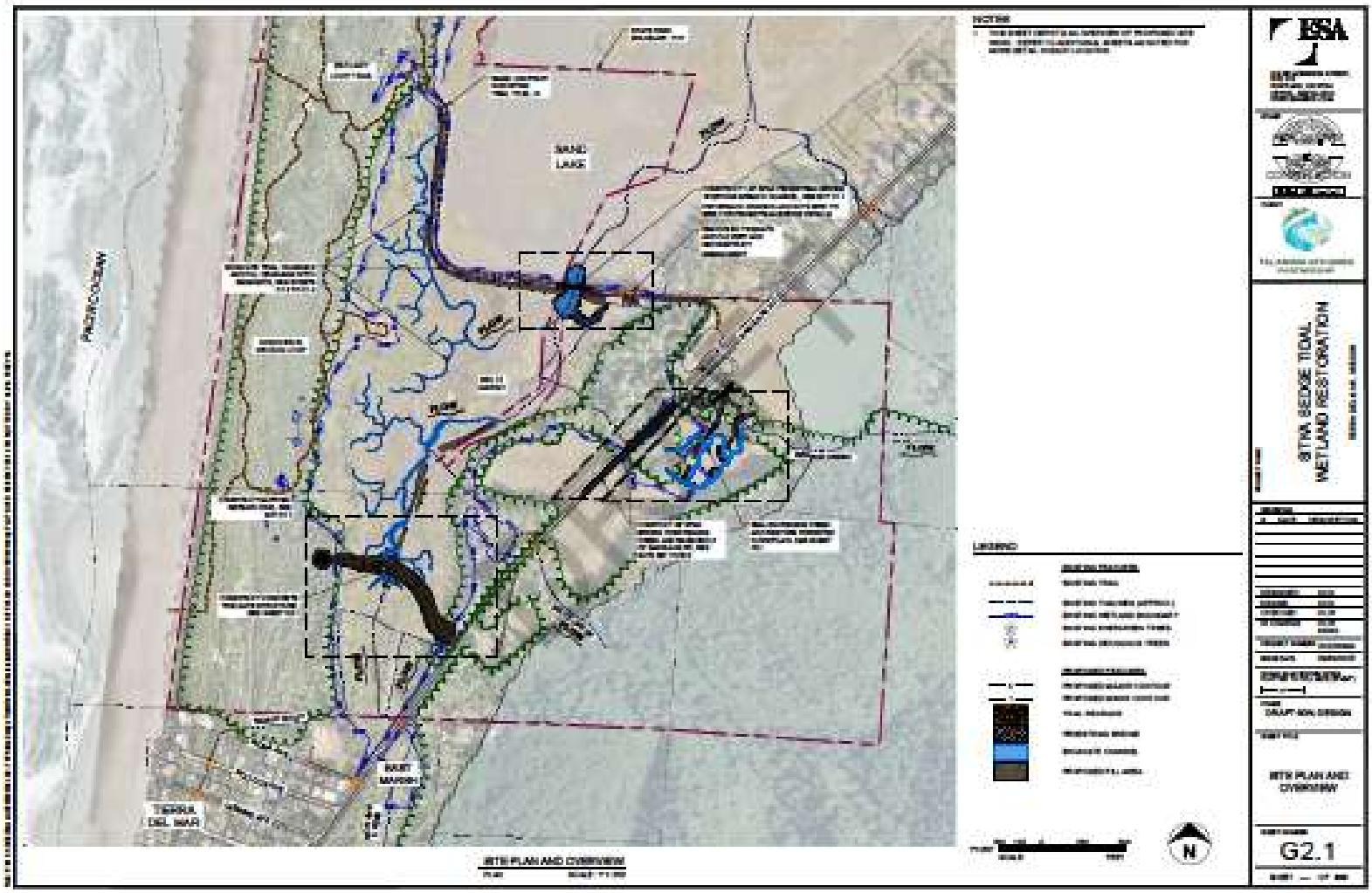
ESSA

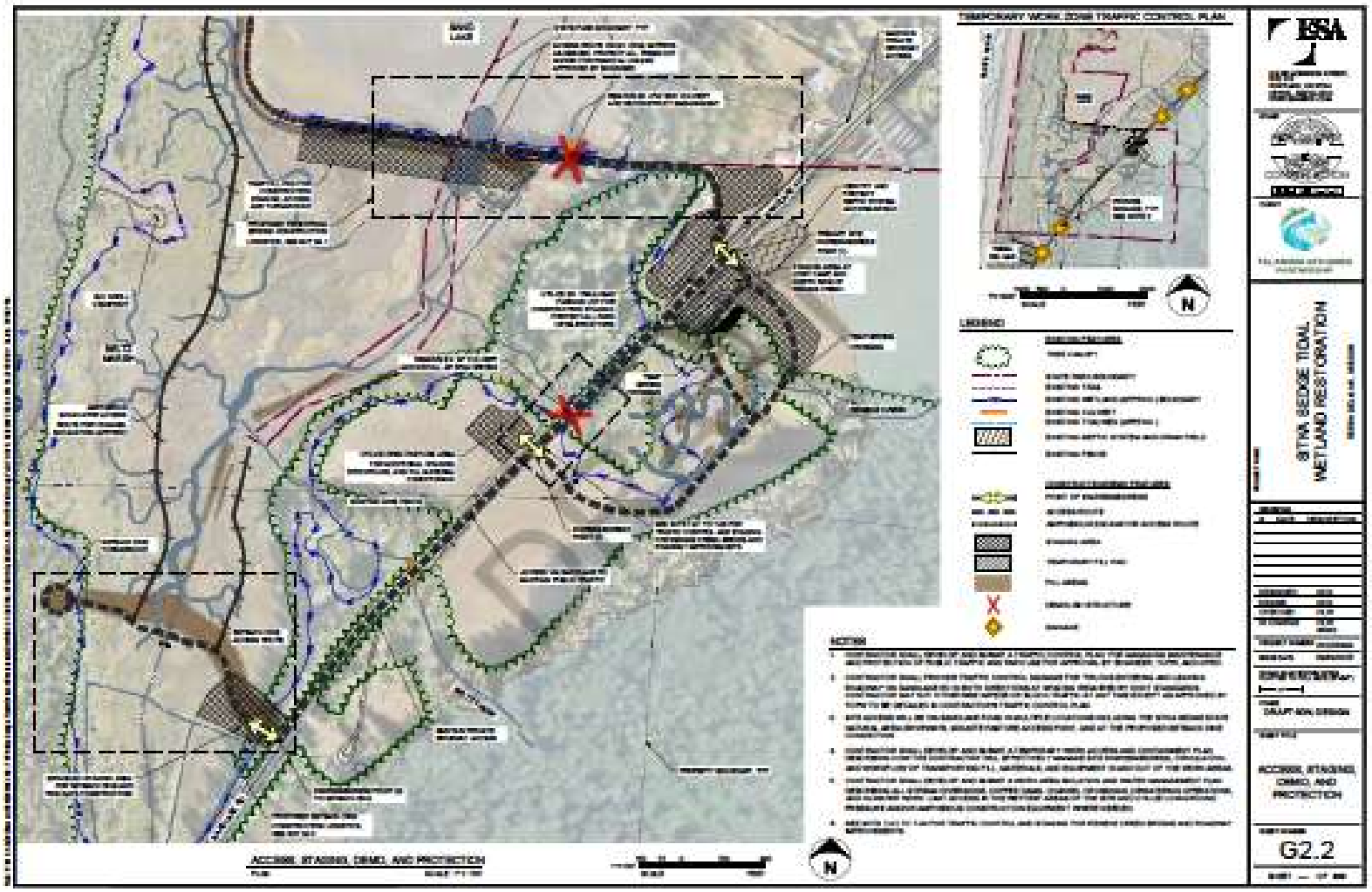
STRA SEDGE TIDAL WETLAND RESTORATION

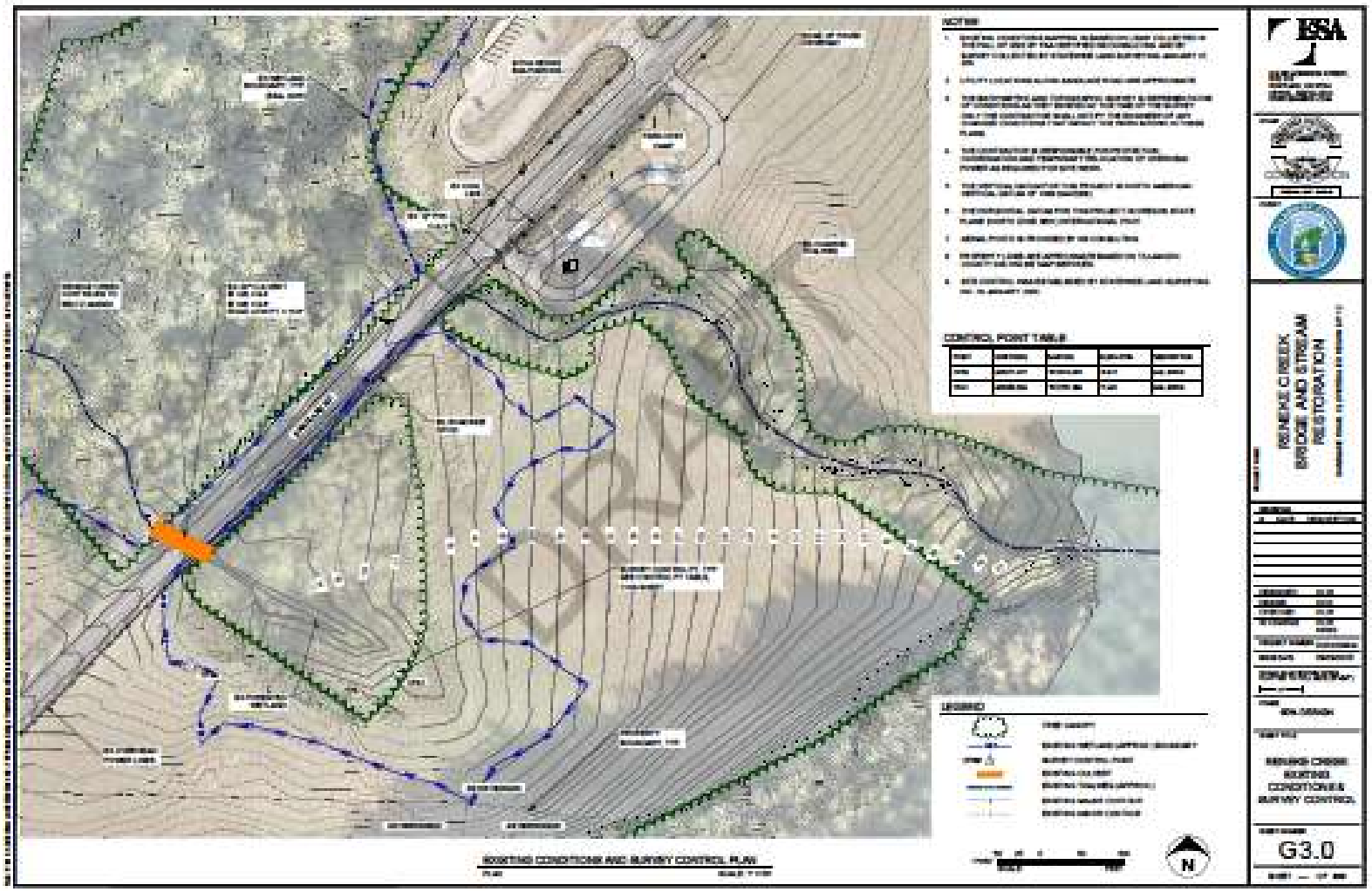
DRAFT-90% DESIGN

EXISTING CONDITIONS AND SURVEY CONTROL

G2.0


NOTES
LEGEND
SAND LAKE
PACIFIC OCEAN
TIERRA DEL MAR
MUST MARSH
BTS PLAN AND OVERVIEW
ESSA
BTRA SEDGE TIDAL WETLAND RESTORATION
DRAFT 90% DESIGN
BTS PLAN AND OVERVIEW
G2.1


TEMPORARY WORK ZONE TRAFFIC CONTROL PLAN
LEGEND
NOTES
ACCESS, STAGING, DEMO, AND PROTECTION
PLAN
ESSA
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT-90% DESIGN
ACCESS, STAGING, DEMO, AND PROTECTION
G2.2


NOTES
CONTROL POINT TABLE
LEGEND
EXISTING CONDITIONS AND SURVEY CONTROL PLAN
ESSA
REMEKE CREEK BRIDGE AND STREAM RESTORATION
EXISTING CONDITIONS & SURVEY CONTROL
G3.0



CONSTRUCTION ENGINEERING AND LOGISTICS - PEDESTRIAN BRIDGE

PLAN

SCALE: 1"=30'

NOTES

1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]
5. [illegible]
6. [illegible]
7. [illegible]
8. [illegible]

LEGEND

[illegible]

ESSA

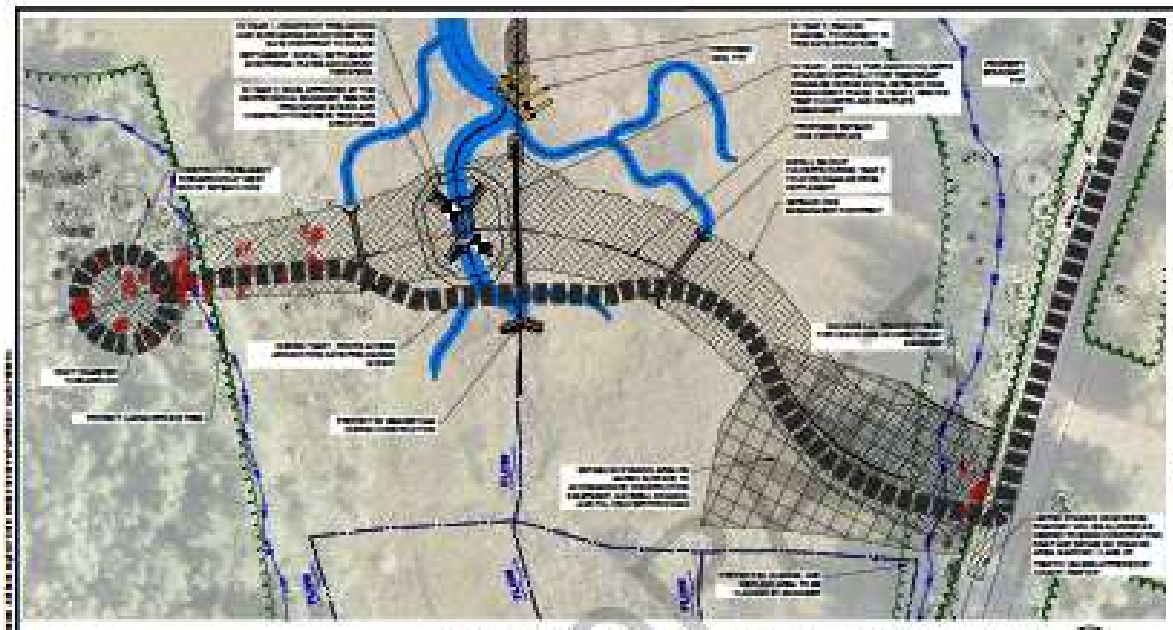
STRA SEDGE TIDAL WETLAND RESTORATION

DRAFT 90% DESIGN

CONSTRUCTION LOGISTICS - PEDESTRIAN BRIDGE

G4.1

DRAFT



CONSTRUCTION SEQUENCING AND LOGISTICS - SETBACK DIKE

NOTES

LEGEND

SETTLEMENT MONITORING PLATE

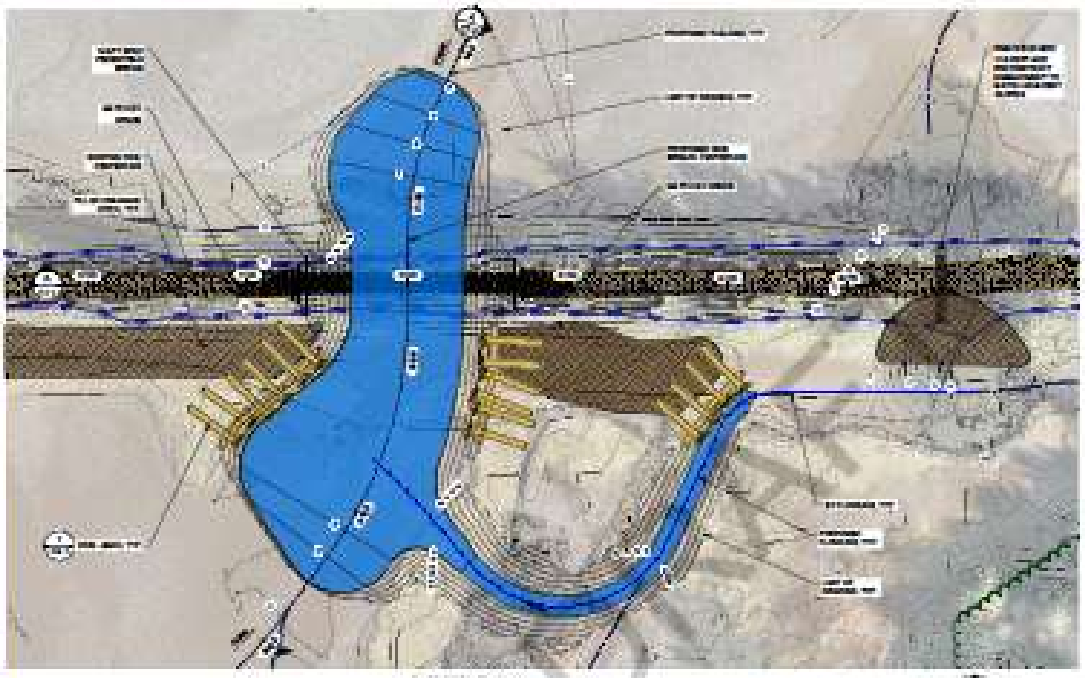
SETTLEMENT MONITORING LEGEND

ESSA

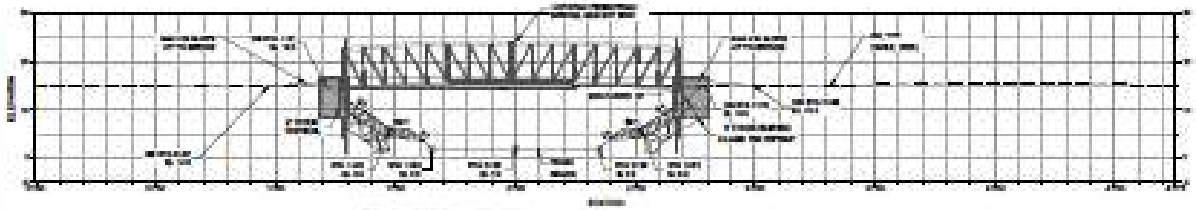
STRA SEDGE TIDAL WETLAND RESTORATION

CONSTRUCTION LOGISTICS - SETBACK DIKE

G4.2



DIKE BREACH PLAN



DIKE BREACH SECTION

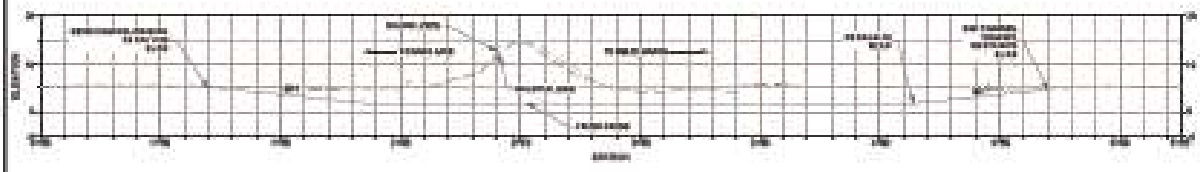
ESSA

STRA SEDGE TIDAL WETLAND RESTORATION

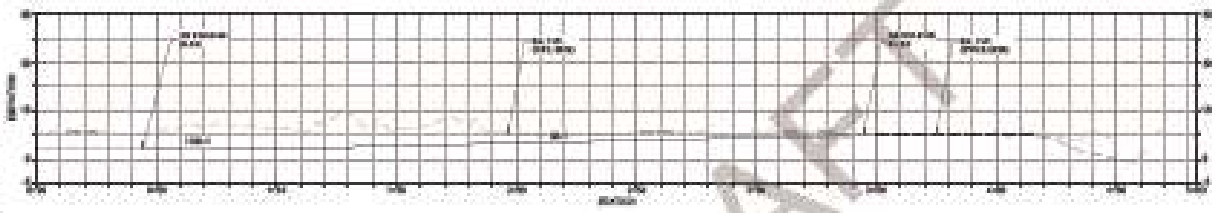
DRAFT 60% DESIGN

DIKE BREACH PLAN AND SECTION

C1.1



A DIKE BREACH CHANNEL PROFILE



B SIDE CHANNEL PROFILE

NOTES

DRAFT

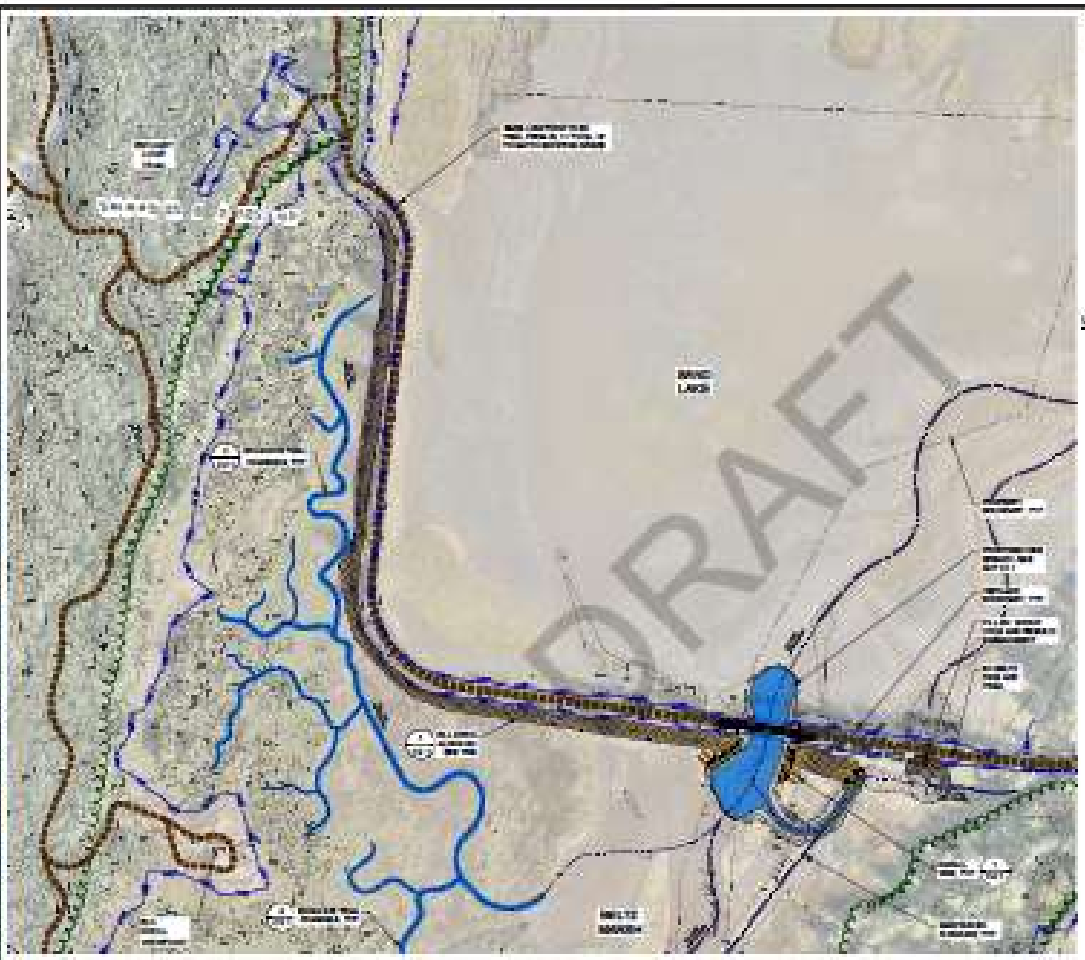
ESSA

SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

DIKE BREACH CHANNEL PROFILE

C1.2



CHANNEL AND DITCH GRADING ENLARGEMENT - 1

PLAN

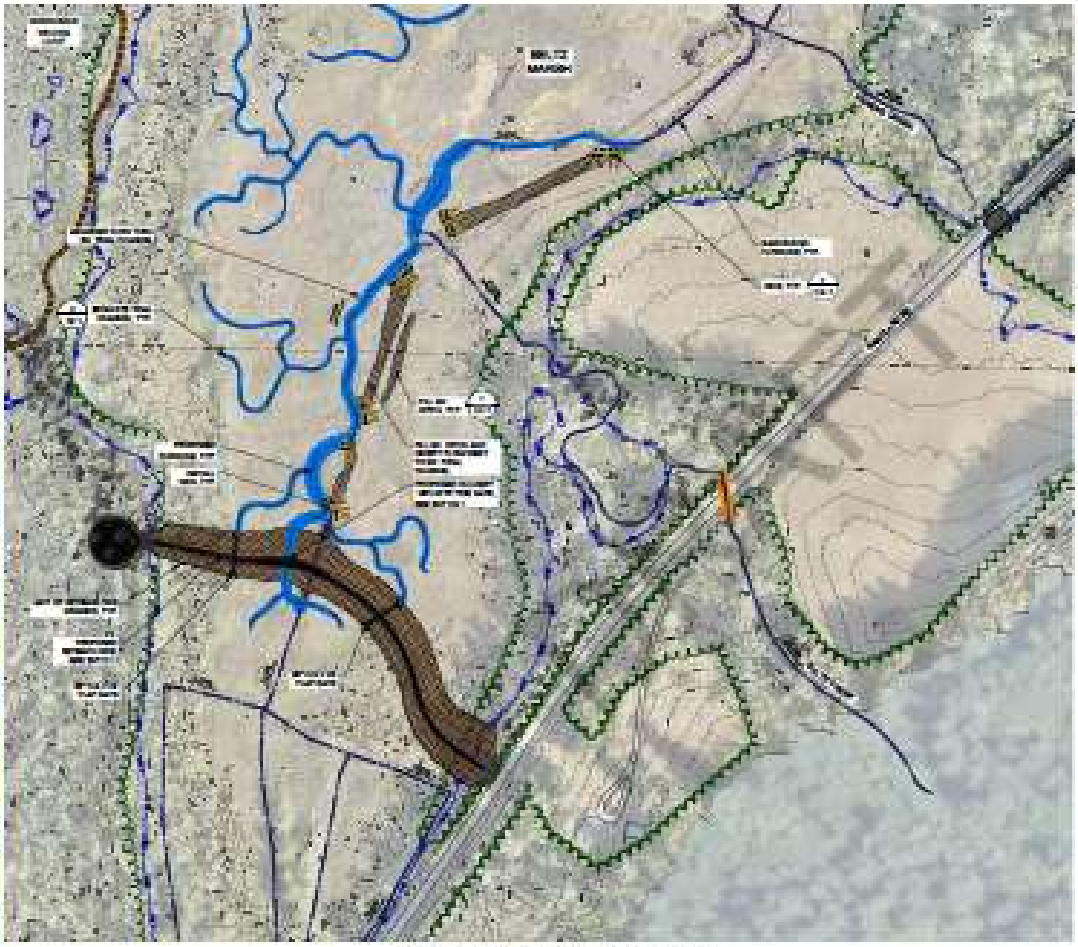
ESSA

STRA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

CHANNEL AND DITCH GRADING ENLARGEMENT - 1

C1.3



CHANNEL AND DITCH GRADING INLARGEMENT - 2

PLAN

SCALE 1" = 50'

NOTES

1. [illegible]

LEGEND

KEY MAP

SCALE

FEET

N

ESSA

STRA SEDGE TIDAL WETLAND RESTORATION

DRAFT 90% DESIGN

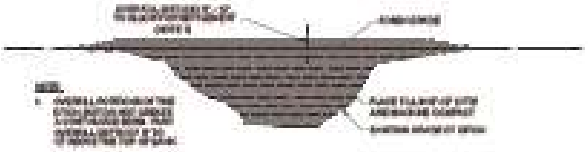
CHANNEL AND DITCH GRADING INLARGEMENT - 2

C1.4

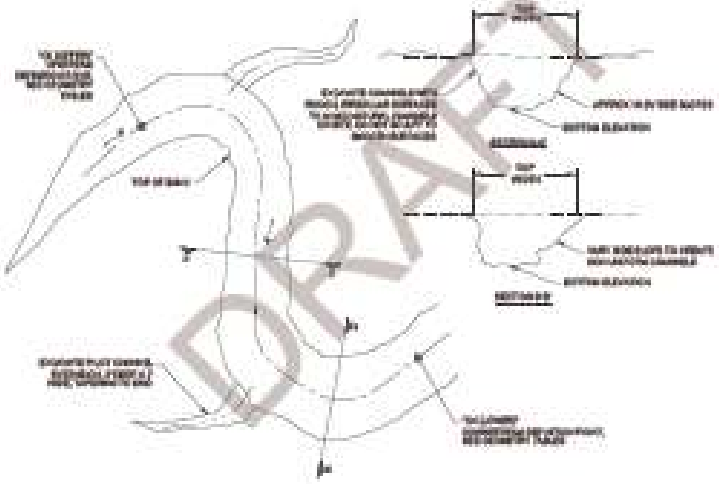
NOTES

1.

2.



TYPICAL DITCHFILL SECTION



TYPICAL NEW CHANNEL PLAN AND CROSS SECTION

DRAFT

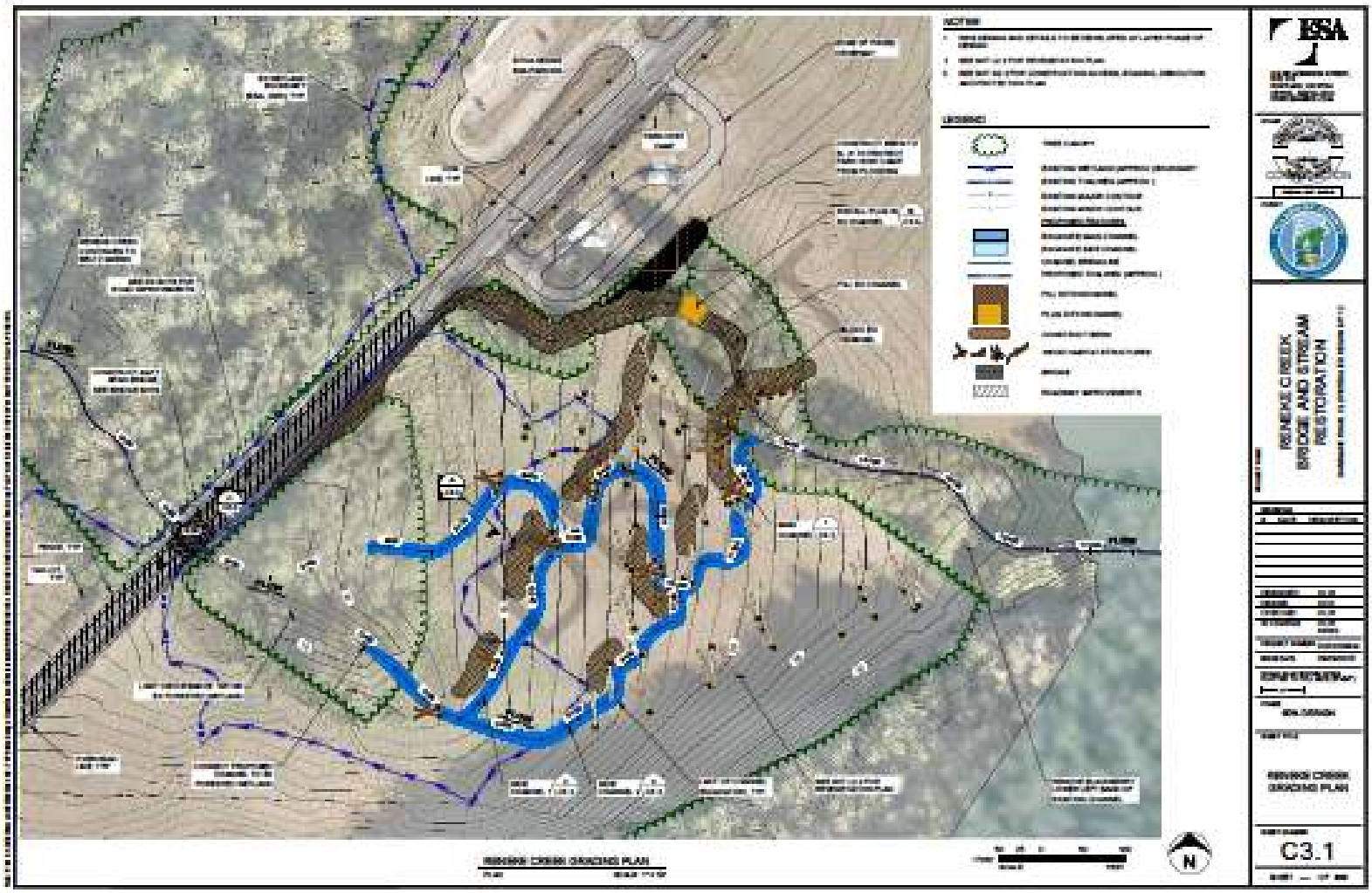
ESSA

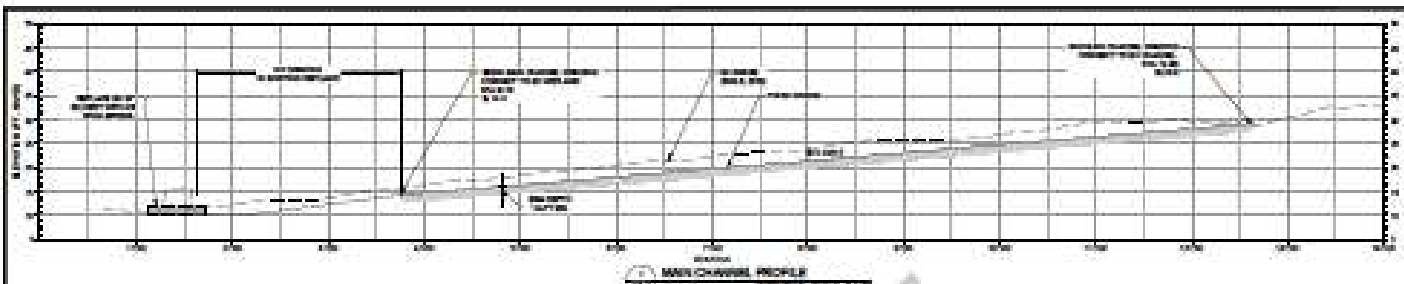
SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

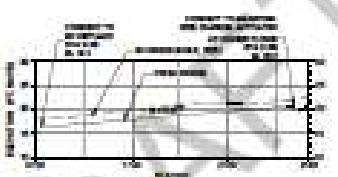
CHANNEL DRAINING SECTIONS AND DETAILS

C2.1

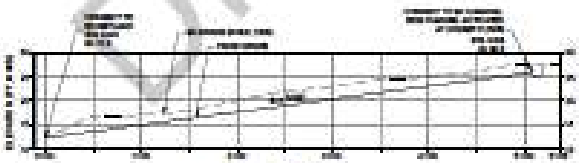

NOTES
LEGEND
REMEMBER CREEK
BRIDGE AND STREAM
RESTORATION
REMEMBER CREEK GRADING PLAN
C3.1
N



MAIN CHANNEL PROFILE



SIDE CHANNEL 1 PROFILE



SIDE CHANNEL 2 PROFILE

DRAFT

ESSA

PENNEYS CREEK
BRIDGE AND STREAM
RESTORATION

REVISIONS

DESIGNED
DRAWN
CHECKED
AS CONSTRUCTED

PROJECT NUMBER

ISSUED FOR/REVISION

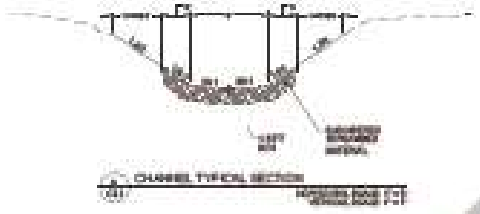
SCALE

BRIDGE CROSS-CHANNEL PROFILES

DRAWING NUMBER

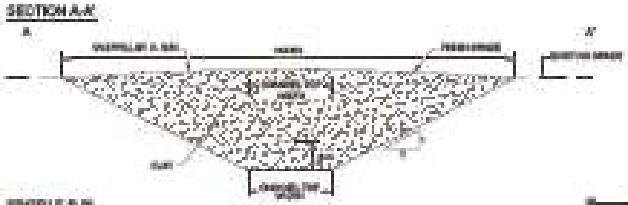
C3.2

SHEET ___ OF ___

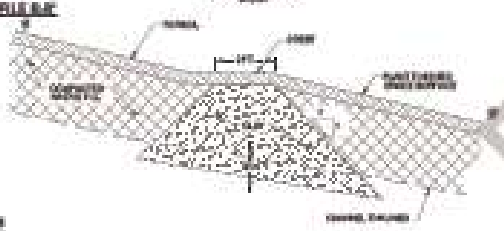


CHANNEL TYPICAL SECTION

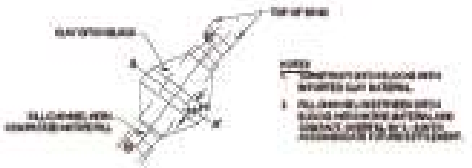
SECTION A-A'



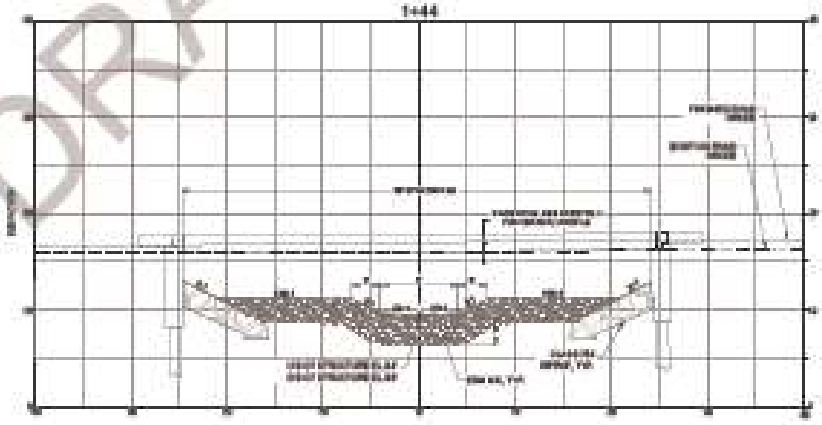
SECTION B-B'



PLAN



DITCH BLOCK



PROPOSED 40-FT SPAN BRIDGE

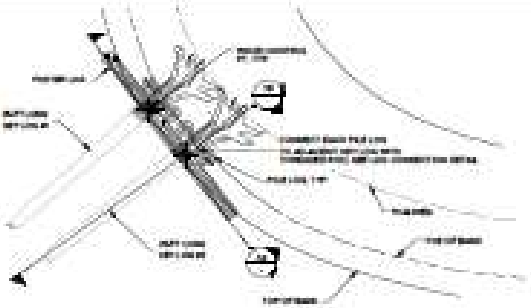
DRAFT

ESSA

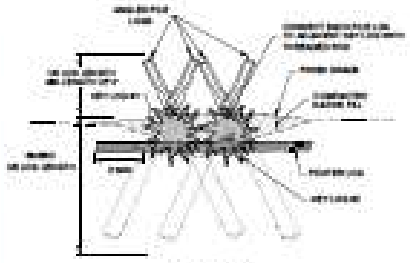
RENDIC CREEK BRIDGE AND STREAM RESTORATION

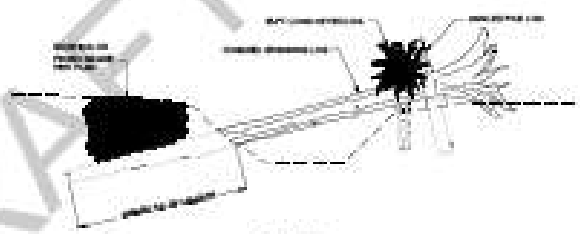
RENDIC CREEK GRADING SECTIONS

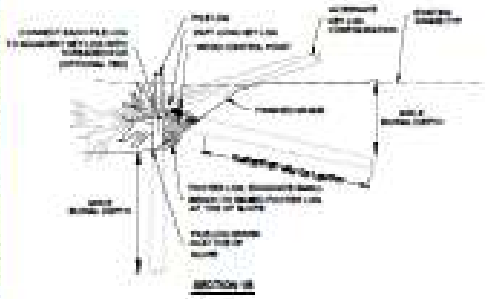
C3.3

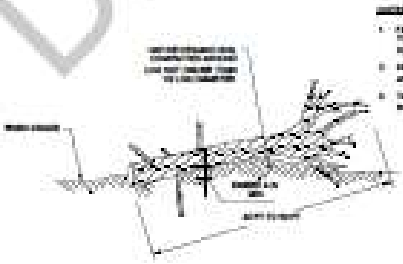














DRAFT

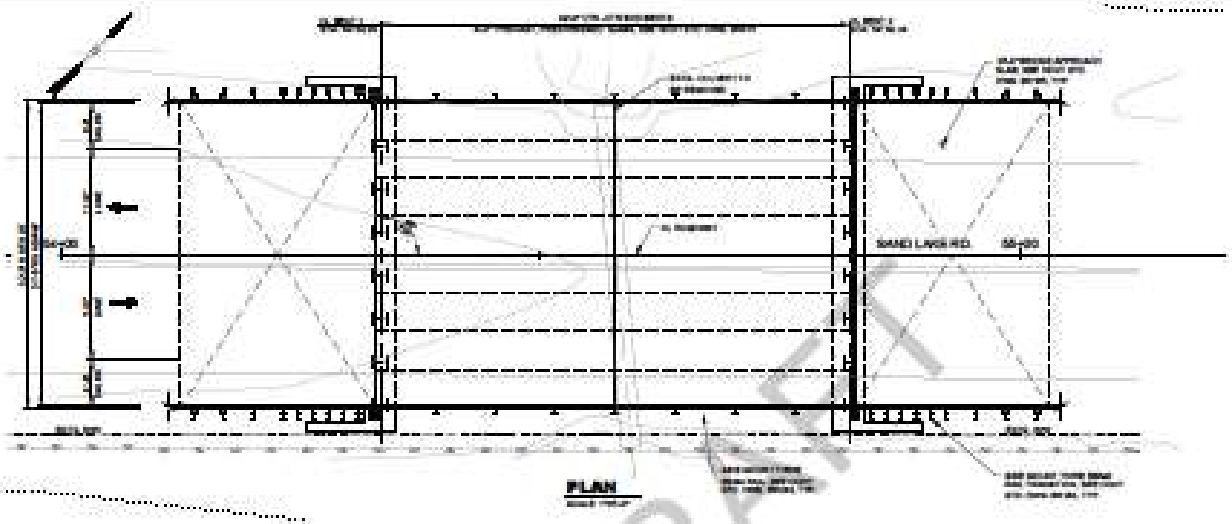
ESSA

STRA SEDGE TIDAL WETLAND RESTORATION

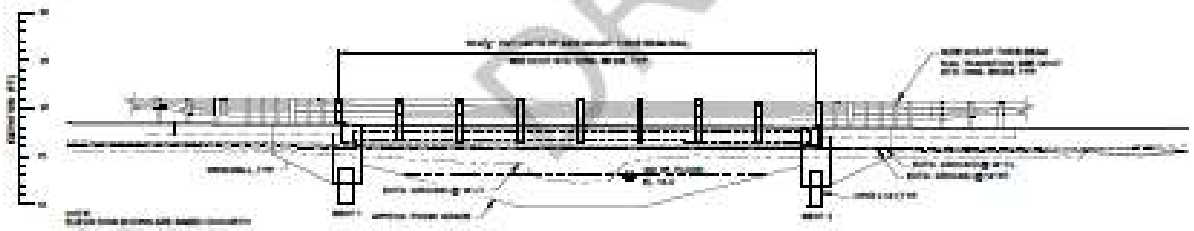
DRAFT 90% DESIGN

HABITAT DETAILS

C4.1



PLAN



ELEVATION

DRAFT

PRELIMINARY
NOT FOR CONSTRUCTION

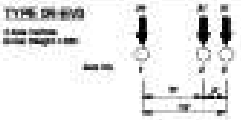
RENEVE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

ISSUED FOR CONSTRUCTION DOCUMENTATION

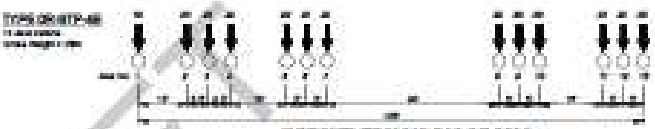
PLAN AND ELEVATION

B01

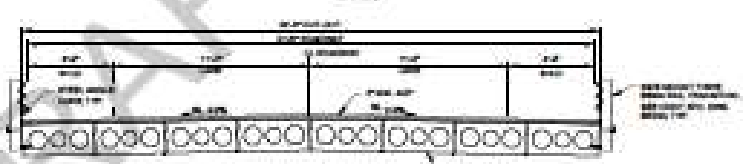
GENERAL NOTES



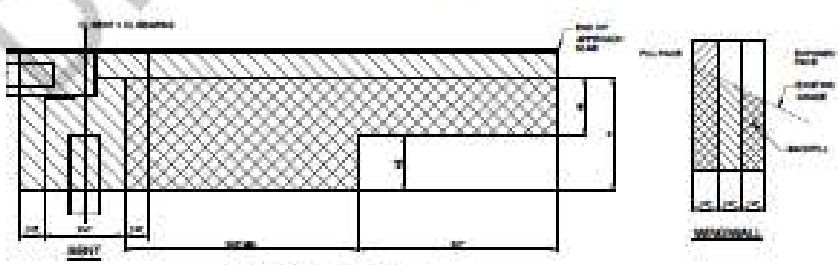




PERMIT TRUCK DIAGRAMS



TYPICAL SECTION



EXCAVATION/BACKFILL DIAGRAM

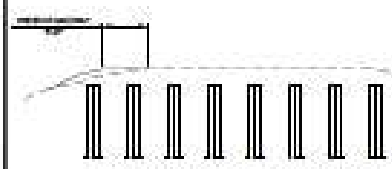
PRELIMINARY
NOT FOR CONSTRUCTION

RENEKE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

AS CONSTRUCTED DOCUMENTATION

GENERAL NOTES AND TYPICAL SECTION

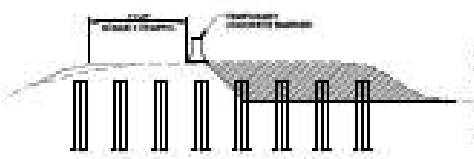
B02



SINGLE LANE CLOSURE UNDER FLAGGING

STAGE I

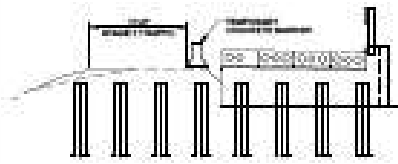
STAGE I NOTES



WEST BOUND LANE CLOSURE

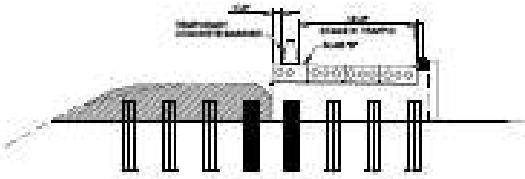
STAGE II

STAGE II NOTES



STAGE III

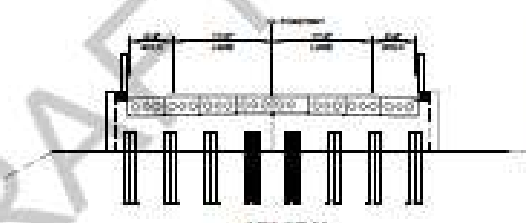
STAGE III NOTES



SOUTH BOND LANE CLOSURE

STAGE IV

STAGE IV NOTES



STAGE V

STAGE V NOTES

DRAFT

PRELIMINARY

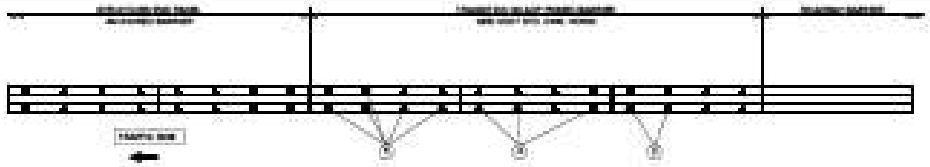
NOT FOR CONSTRUCTION

RENEKE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

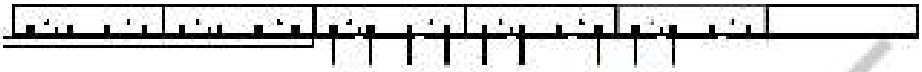
FOR CONSTRUCTION DOCUMENTATION

CONSTRUCTION STAGING

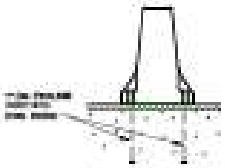
B03



RAIL PLAN



RAIL ELEVATION



PINNED BARRIER SECTION

GENERAL NOTES

DRAFT

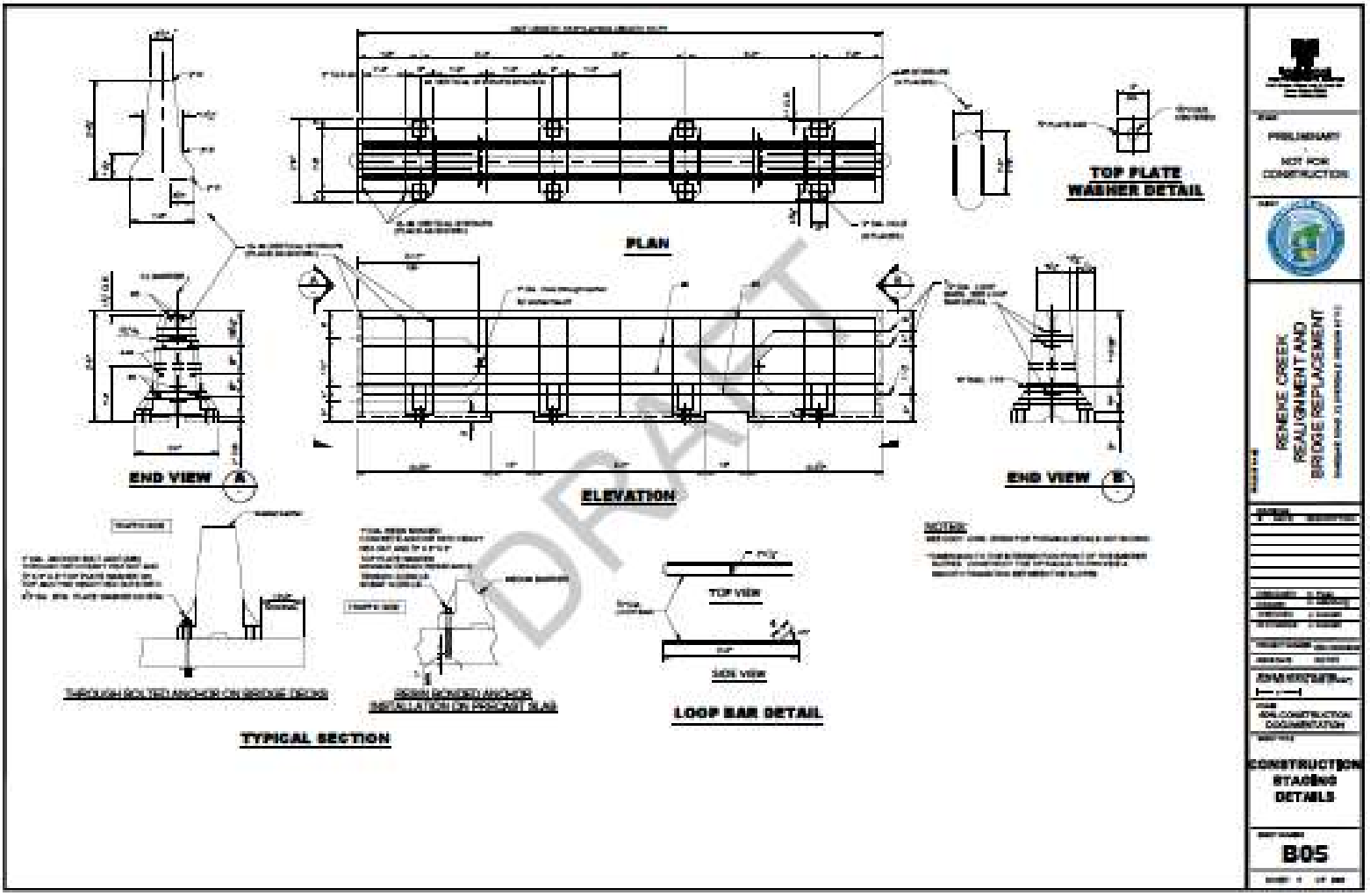
PRELIMINARY
NOT FOR CONSTRUCTION

RENEVE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

FOR CONSTRUCTION DOCUMENTATION

CONSTRUCTION STAGING DETAILS

B04



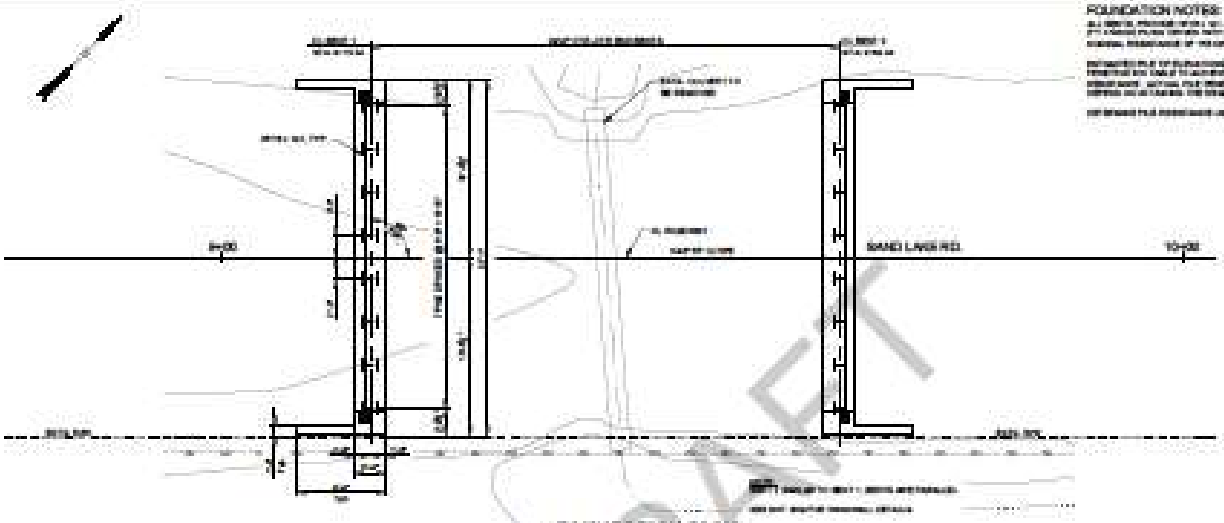
PRELIMINARY
NOT FOR CONSTRUCTION

RENEVE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

FOR CONSTRUCTION DOCUMENTATION

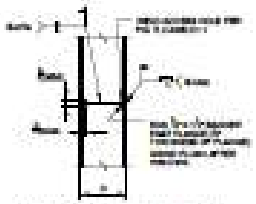
CONSTRUCTION STAGING DETAILS

B05



FOUNDATION PLAN

FOUNDATION NOTES:



H-PILE SPLICE DETAIL

PILE PENETRATION TABLE

DRAFT

PRELIMINARY
NOT FOR CONSTRUCTION

RENEAU CREEK REALIGNMENT AND BRIDGE REPLACEMENT

AS-CONSTRUCTION DOCUMENTATION

FOUNDATION PLAN

B06

21" STANDARD PRECAST PRESTRESSED SLABS

A	1	1	44-0½	11	5	30	3	3.01	4.38	9.5	4.1	19.4	¾"	1½"	-	9"	9"	¾"
B	1	1	44-0½	11	5	30	3	3.01	4.38	9.5	4.1	19.4	¾"	1½"	-	5"	5"	¾"

ACP BUILD-UP DETAIL

SLAB SHEAR KEY DETAIL

NOTES:

1.
2.
3.
4.

TYPICAL 21" SLAB "A" SECTION

TYPICAL 21" SLAB "B" SECTION

CONCRETE PAD DETAIL

BEARING DETAIL

PLAN

DRAFT

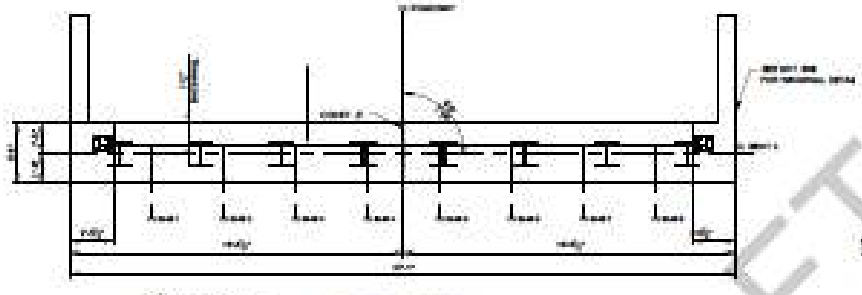
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NOT FOR CONSTRUCTION

RENEHE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

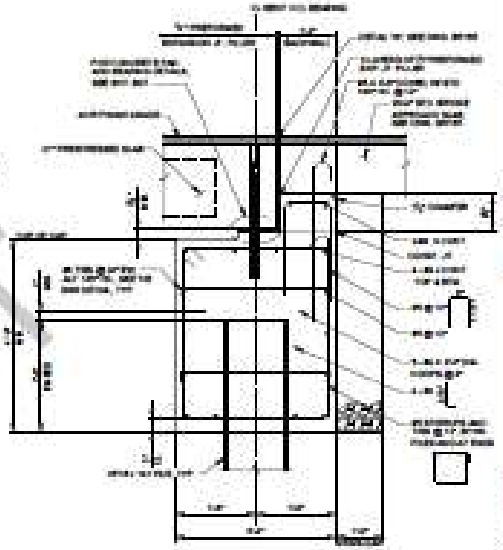
AS-CONSTRUCTED DOCUMENTATION

PRECAST SLAB DETAILS

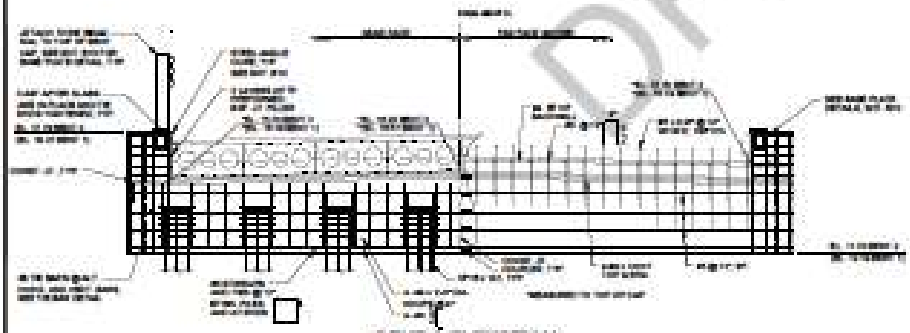
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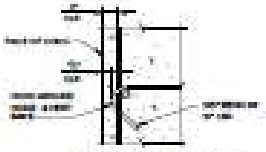
BENT 2 PLAN



TYPICAL BENT SECTION



BENT 2 ELEVATION



TIE BAR DETAIL

DRAFT

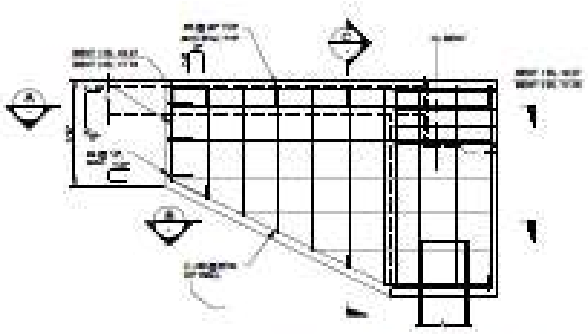
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NOT FOR CONSTRUCTION

RENEWE CREEK
REALIGNMENT AND
BRIDGE REPLACEMENT

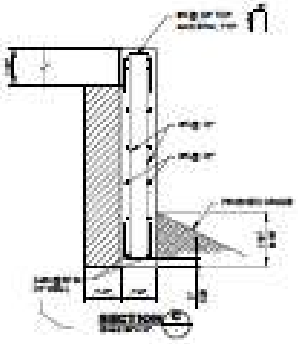
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DOCUMENTATION

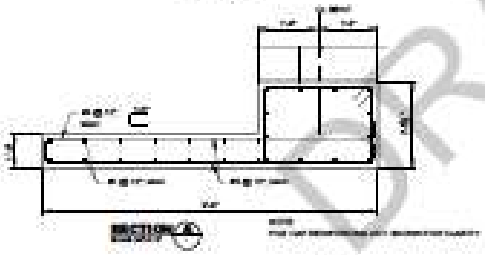
BENT
DETAILS

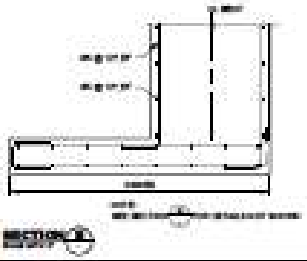
B08



TYPICAL WINGWALL ELEVATION







DRAFT

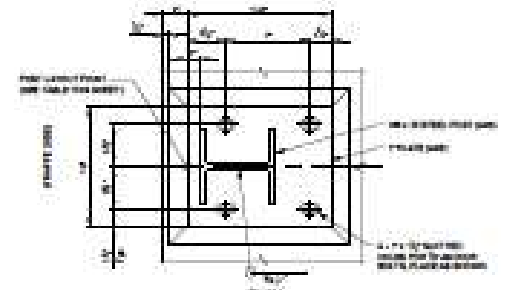
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NOT FOR CONSTRUCTION

RENEKE CREEK
REALIGNMENT AND
BRIDGE REPLACEMENT

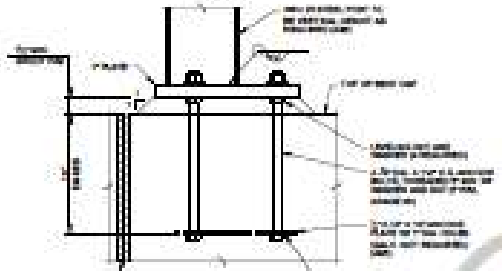
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WINGWALL DETAILS

B09

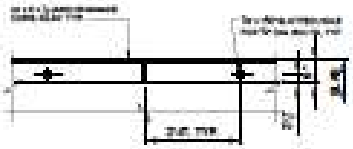


PLAN

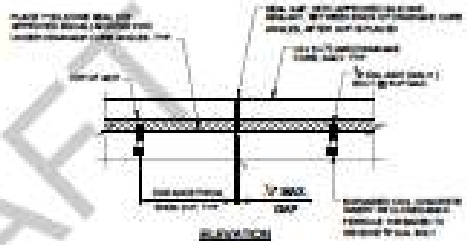


ELEVATION

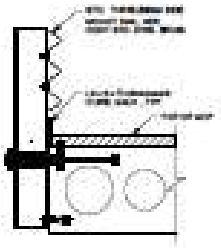
RAIL POST BASE PLATE DETAILS



PLAN



ELEVATION



SECTION

DRAINAGE CURB DETAIL

DRAFT

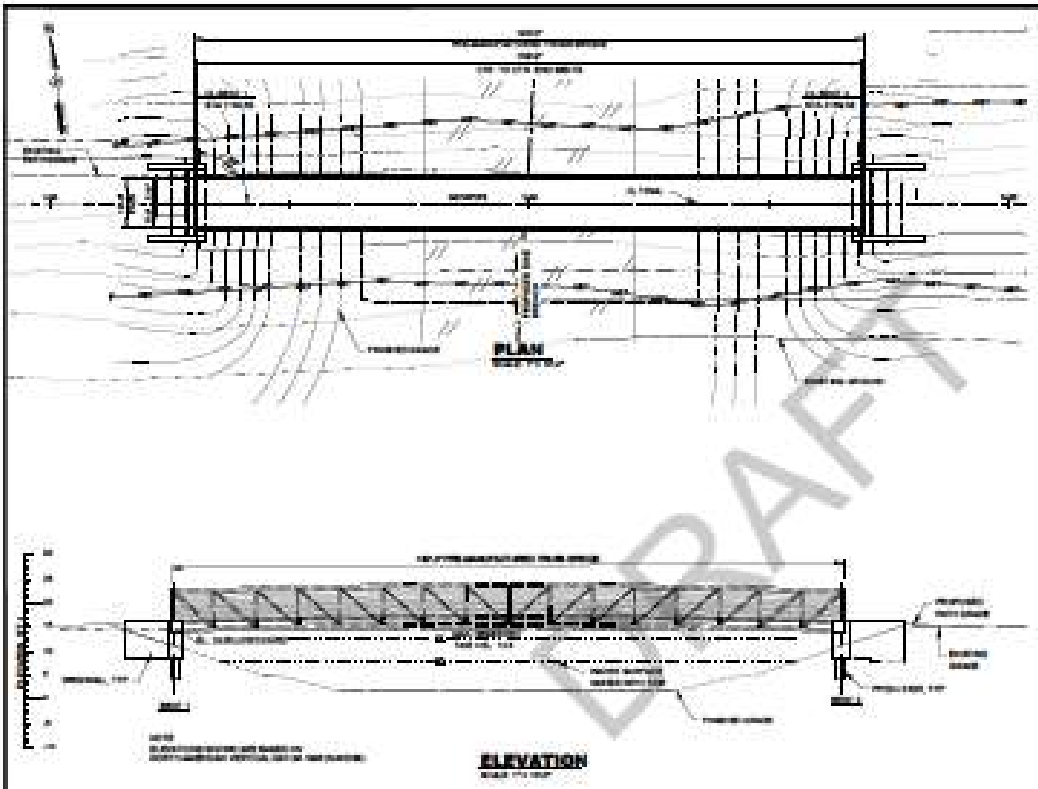
PRELIMINARY
NOT FOR CONSTRUCTION

RENEVE CREEK REALIGNMENT AND BRIDGE REPLACEMENT

FOR CONSTRUCTION DOCUMENTATION

MISCELLANEOUS DETAILS

B10



DRAFT

PRELIMINARY
NOT FOR CONSTRUCTION

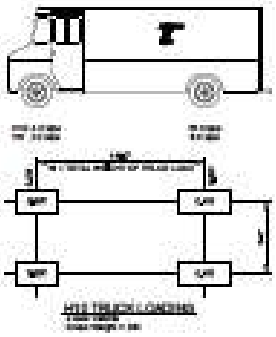
SITKA SEDGE
PEDESTRIAN BRIDGE

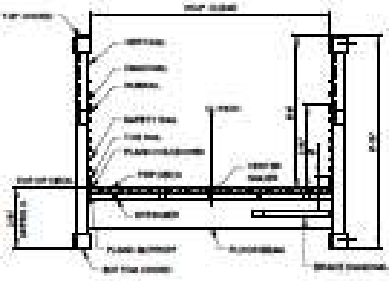
FOR CONSTRUCTION DOCUMENTATION

PLAN AND ELEVATION

BR01

GENERAL NOTES





TYPICAL SECTION

DRAFT

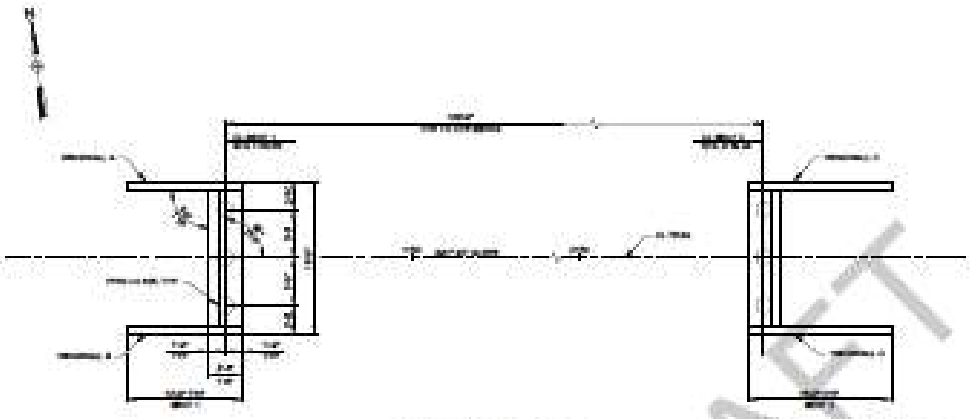
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NOT FOR CONSTRUCTION

9TH AVENUE
PEDESTRIAN BRIDGE

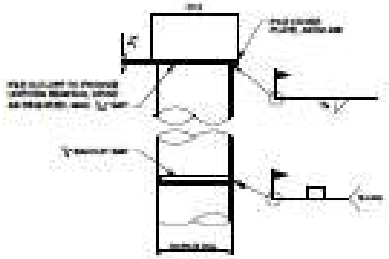
FOR CONSTRUCTION DOCUMENTATION

GENERAL NOTES

BR02



FOUNDATION PLAN



PIPE PILE DETAIL

FOUNDATION NOTES

PILE INSTALLATION TABLE

DRAFT

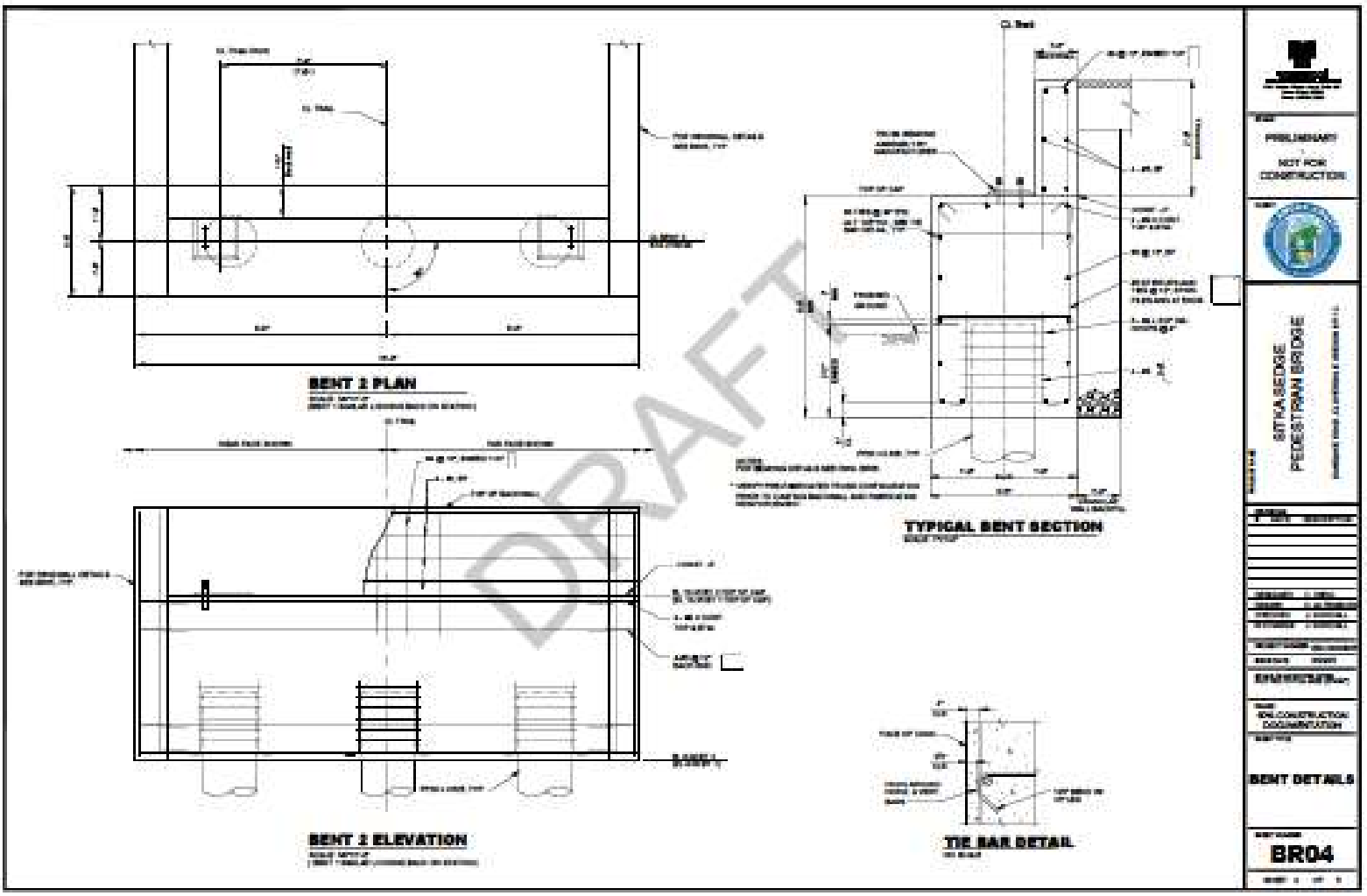
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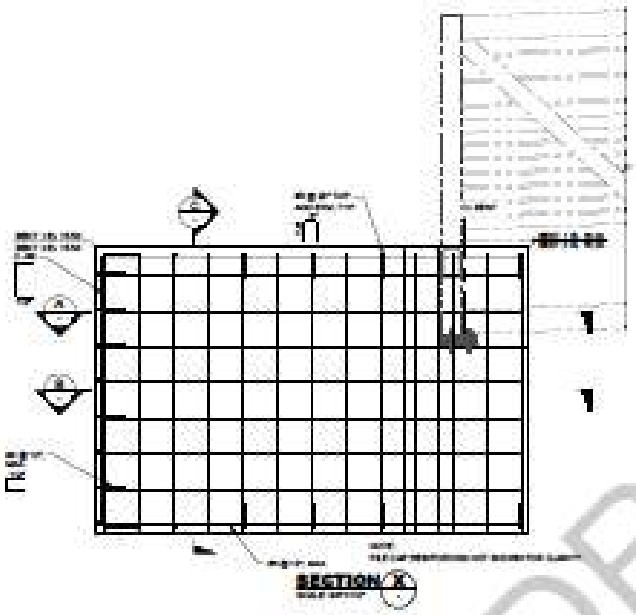
SITKA SEDGE
PEDESTRIAN BRIDGE

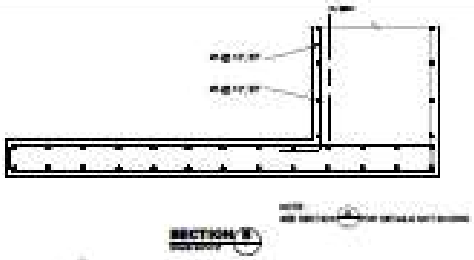
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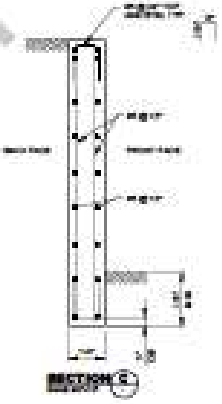
FOUNDATION PLAN

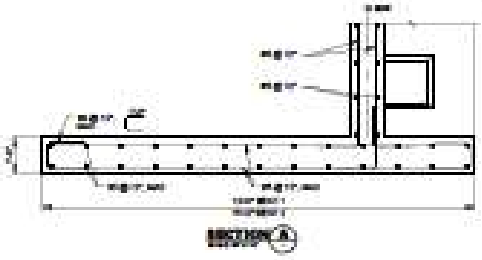
BR03


BENT 2 PLAN
TYPICAL BENT SECTION
BENT 2 ELEVATION
TIE BAR DETAIL
DRAFT
PRELIMINARY
NOT FOR CONSTRUCTION
SITKA SEDGE
PEDESTRIAN BRIDGE
FOR CONSTRUCTION DOCUMENTATION
BENT DETAILS
BR04









PRELIMINARY

NOT FOR CONSTRUCTION

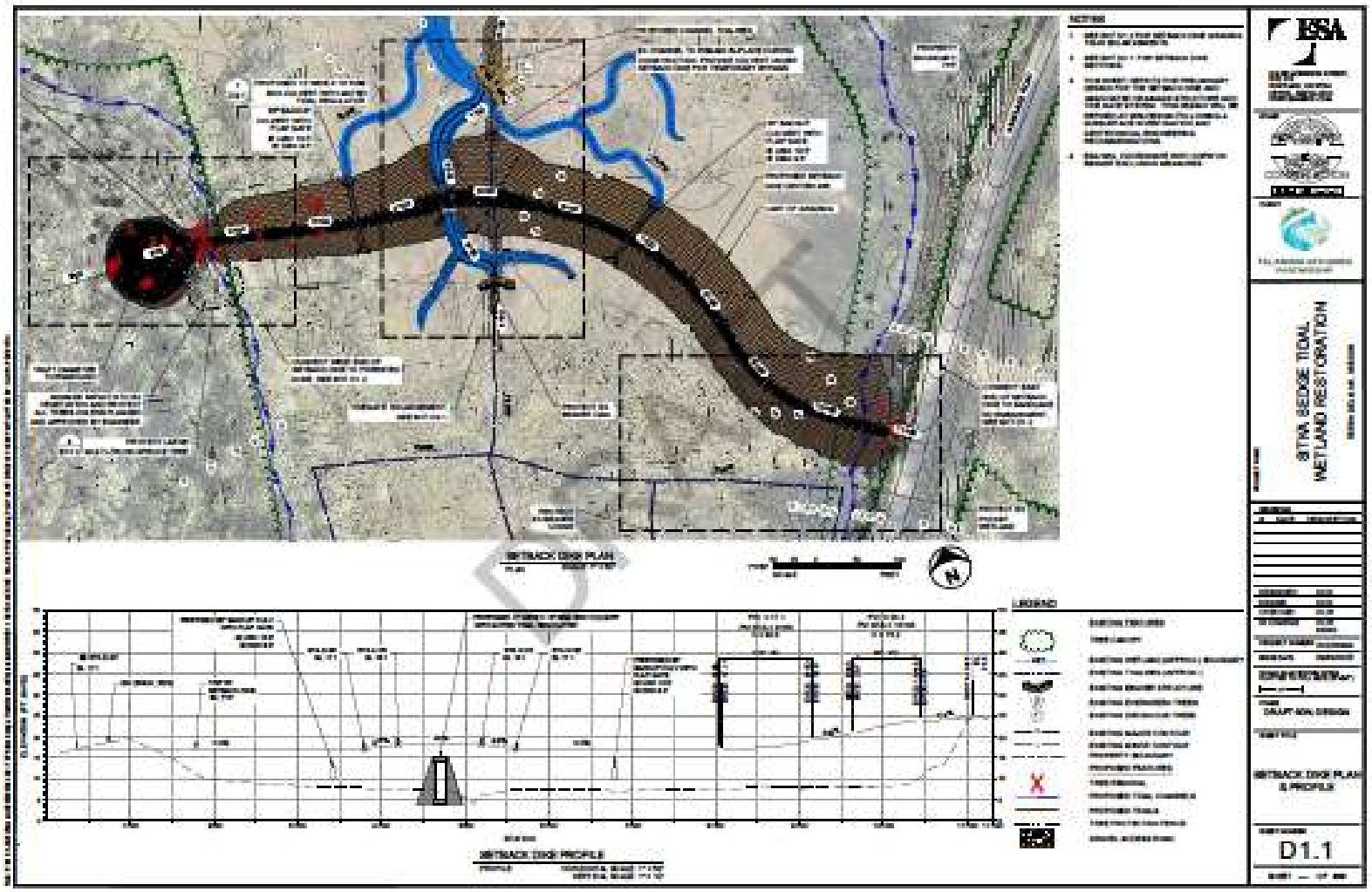
STKASEDGE
PEDESTRIAN BRIDGE

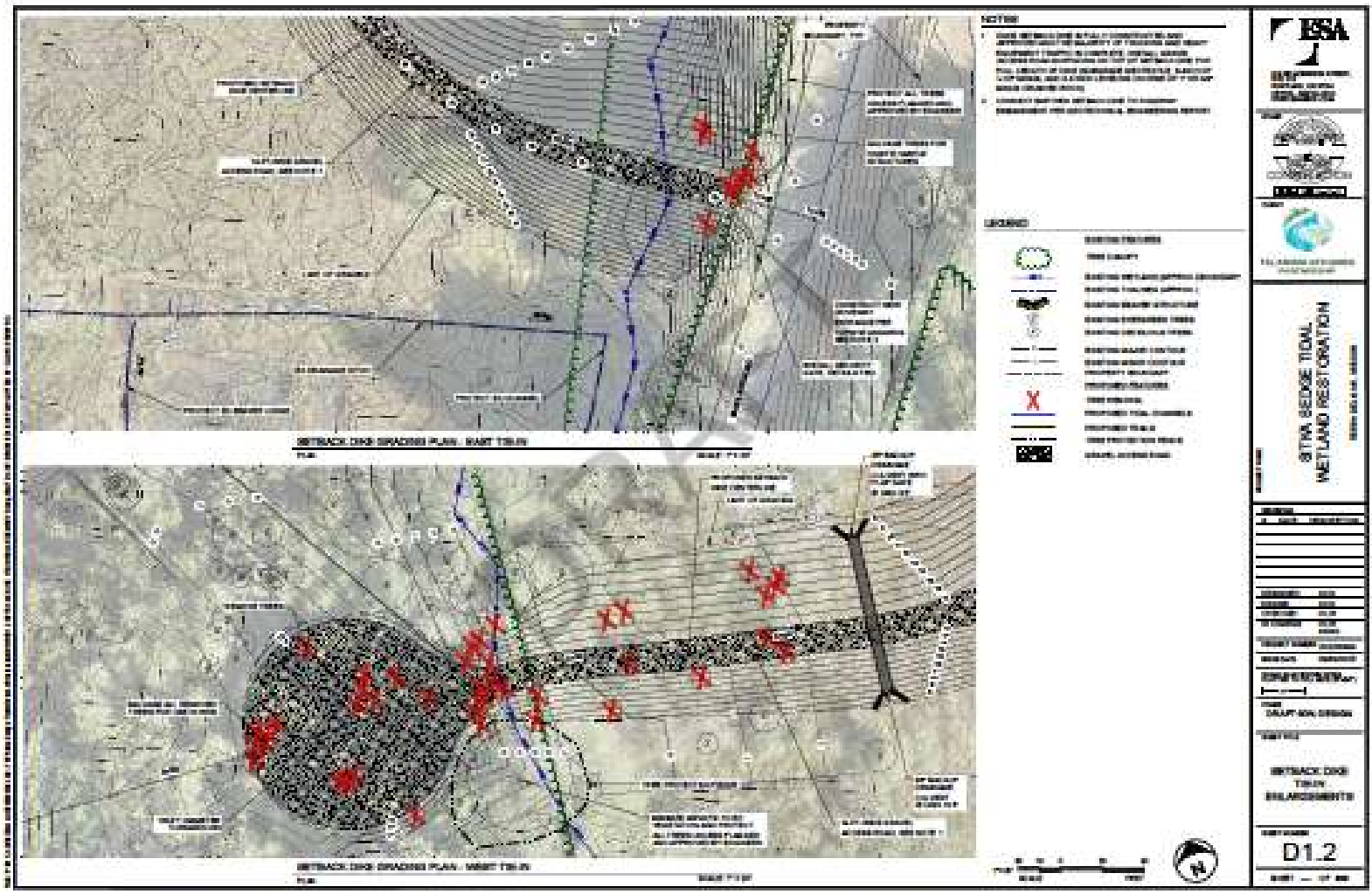
FOR CONSTRUCTION DOCUMENTATION

WINGWALL DETAILS

BR05

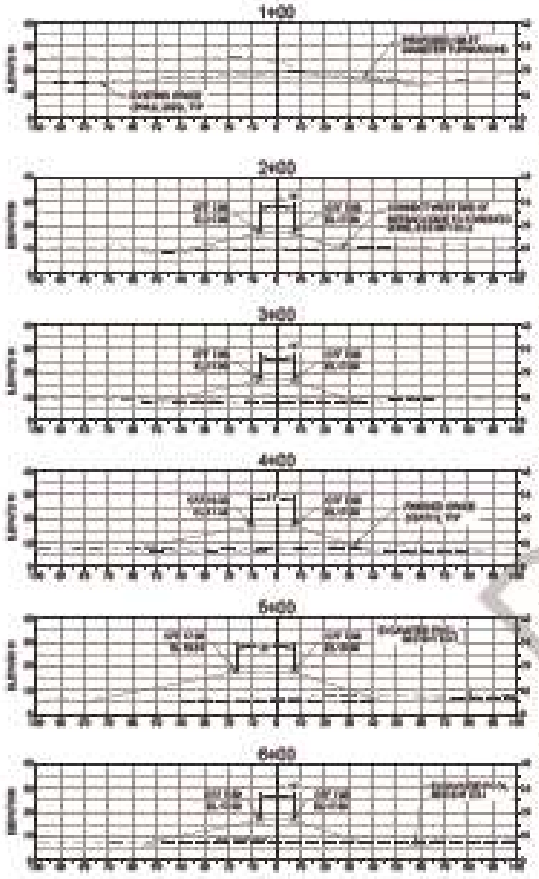
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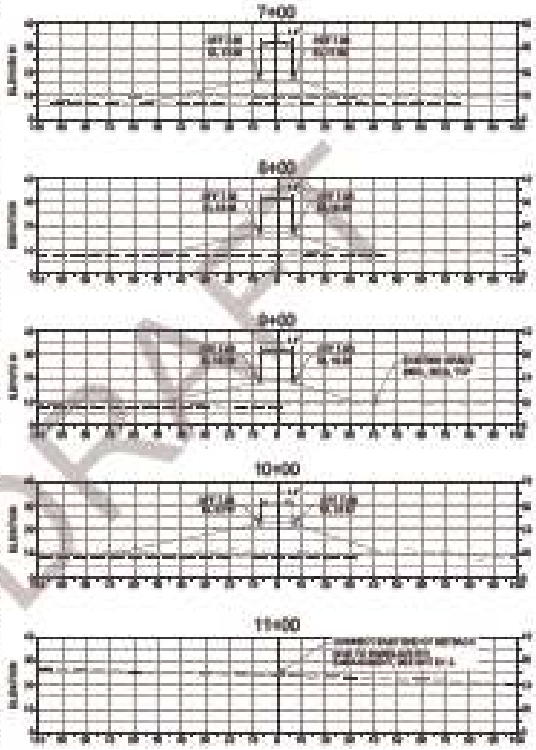

ESSA
STRA SEDGE TIDAL WETLAND RESTORATION
SETBACK DIKE PLAN
SETBACK DIKE PROFILE
LEGEND
DRAFT 90% DESIGN
SETBACK DIKE PLAN & PROFILE
D1.1


SETBACK DIKE GRADING PLAN - EAST TIBURON
SETBACK DIKE GRADING PLAN - WEST TIBURON
NOTES
LEGEND
ESSA
STRAWBERRY POINT
TIBURON
MARIN COUNTY
CALIFORNIA
STRA SEDGE TIDAL
WETLAND RESTORATION
DRAFT 90% DESIGN
SETBACK DIKE
TIBURON
ENLARGEMENTS
D1.2

NOTES

1. SEE NOTES 1 FOR SETBACK DIKE PLAN & PROFILE





PROPOSED SETBACK DIKE
SECTIONS

SCALE 1" = 10'

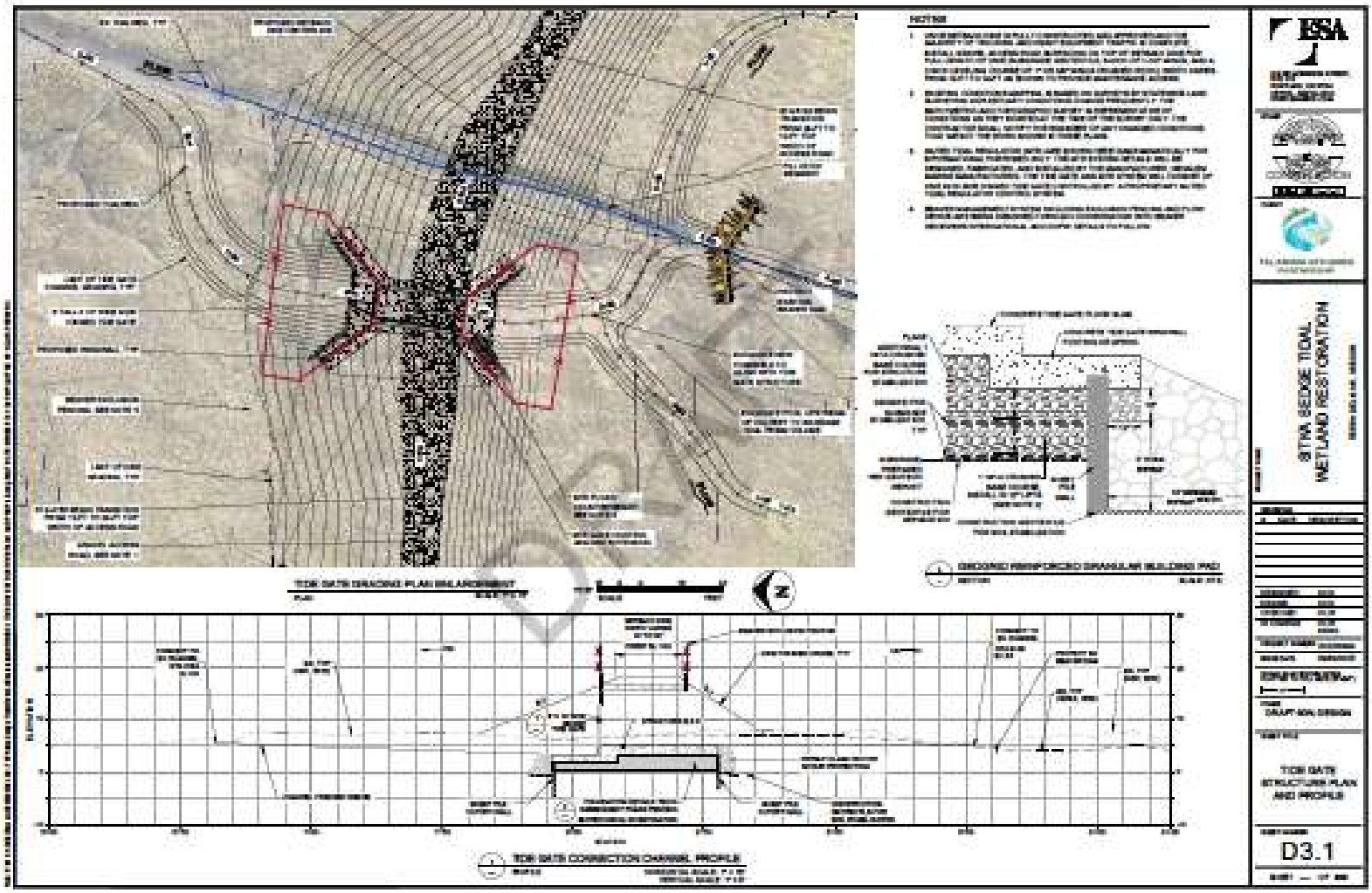
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SITIA SEDGE TIDAL WETLAND RESTORATION

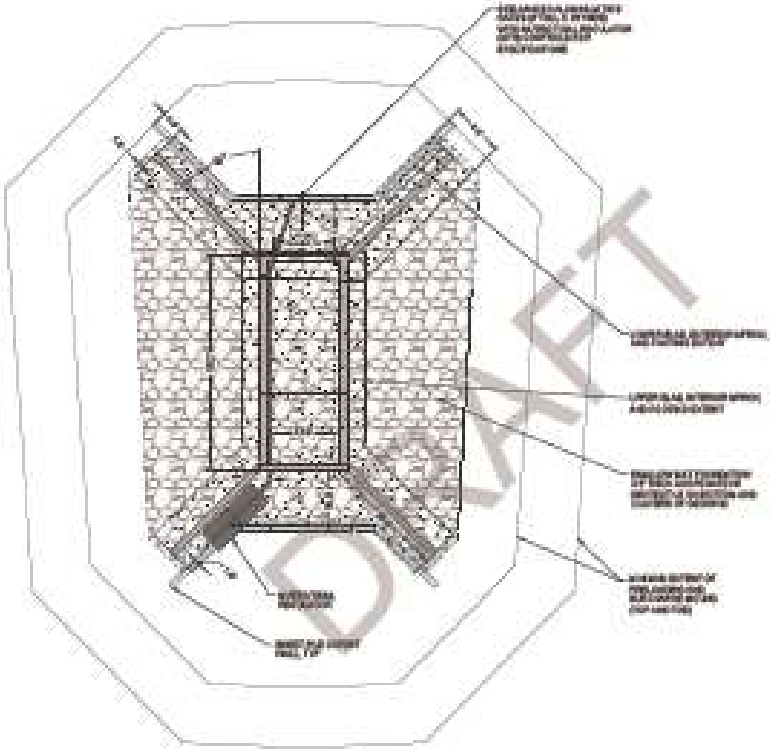
DRAFT 60% DESIGN

SETBACK DIKE SECTIONS

D2.1


ESSA
STRA SEDGE TIDAL WETLAND RESTORATION
TIDE GATE STRUCTURE PLAN AND PROFILE
D3.1
TIDE GATE GRADING PLAN
TIDE GATE CONNECTION CHANNEL PROFILE
GEOGRID REINFORCED GRANULAR BUILDING PAD

NOTES



DRAFT

ESSA

STRA SEDGE TIDAL WETLAND RESTORATION

DRAFT EIA DESIGN

TIDEGATE DETAIL

D3.2

SITKA SEDGE
TIDAL WETLAND RESTORATION
1200-C PERMIT

SHEET LIST

PROJECT TEAM

VICINITY MAP



DRAFT

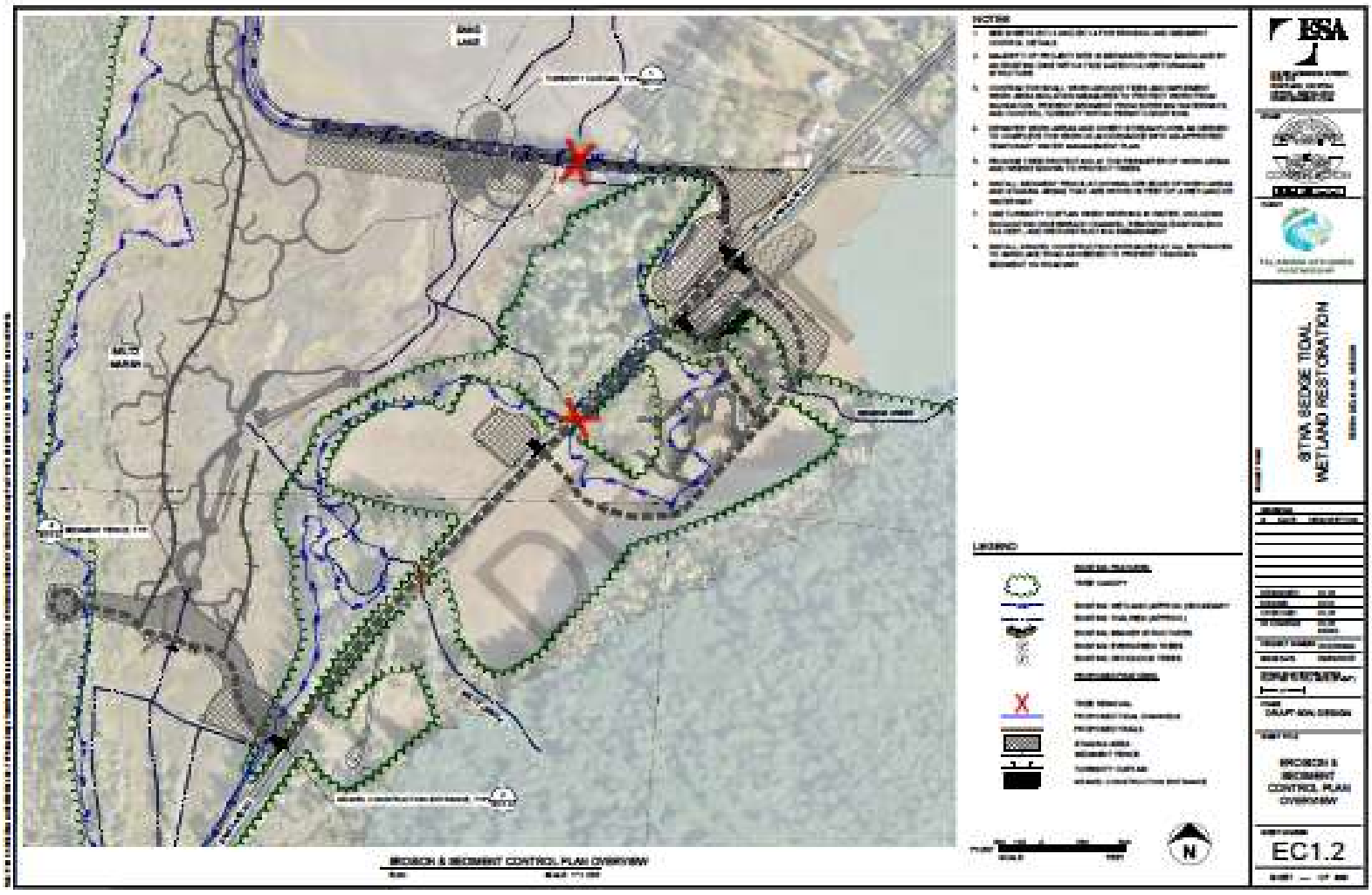
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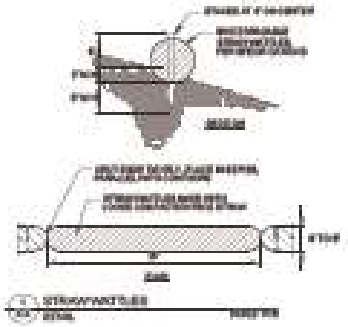
SITKA SEDGE TIDAL WETLAND RESTORATION

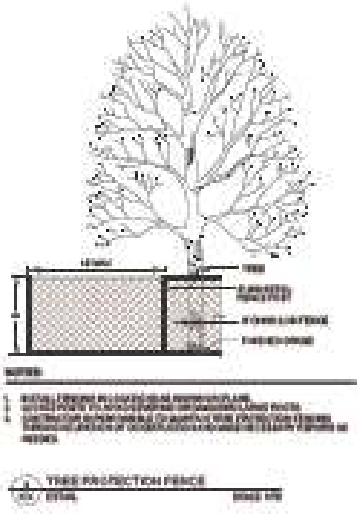
DRAFT 60% DESIGN

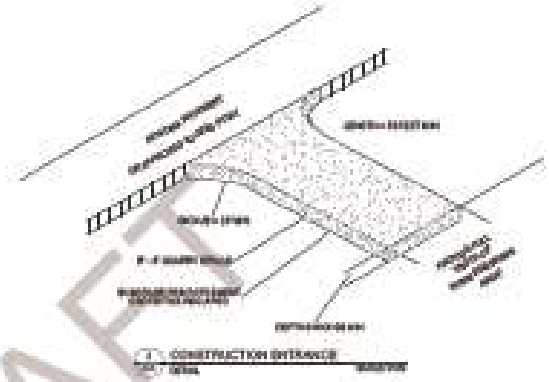
1200-C ESC COVER

EC1.1

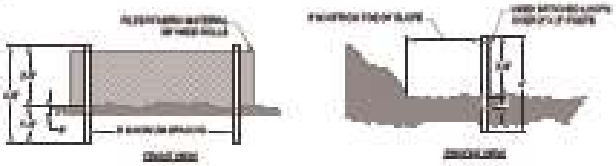

NOTES
LEGEND
ESSA
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT-90% DESIGN
BROOCH & SEDIMENT CONTROL PLAN OVERVIEW
EC1.2
BROOCH & SEDIMENT CONTROL PLAN OVERVIEW
N











DRAFT

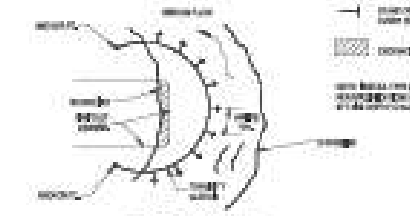
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SITKA SEDGE TIDAL WETLAND RESTORATION

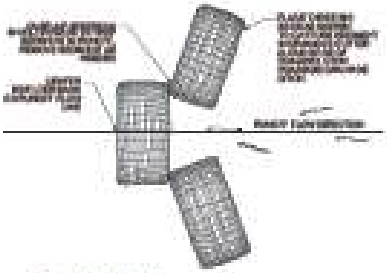
DRAFT EDA DESIGN

EROSION CONTROL DETAILS - 1

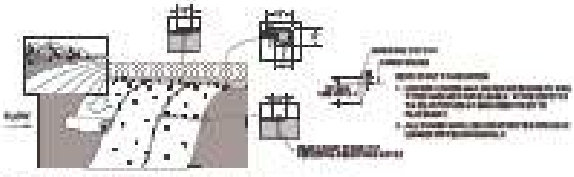
EC1.3



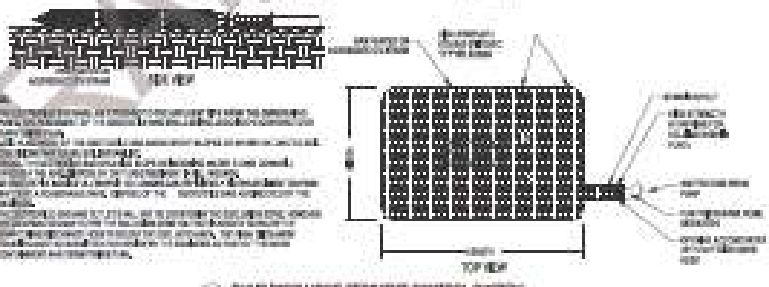
TURBIDITY CURTAIN



BIO FILTER BAG



COIR FABRIC



PUMP DISCHARGE SEDIMENT CONTROL SYSTEM



DEWATERING SYSTEM

ESSA

SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT-EIA DESIGN

EROSION CONTROL DETAILS - 2

EC1.4

NOTES

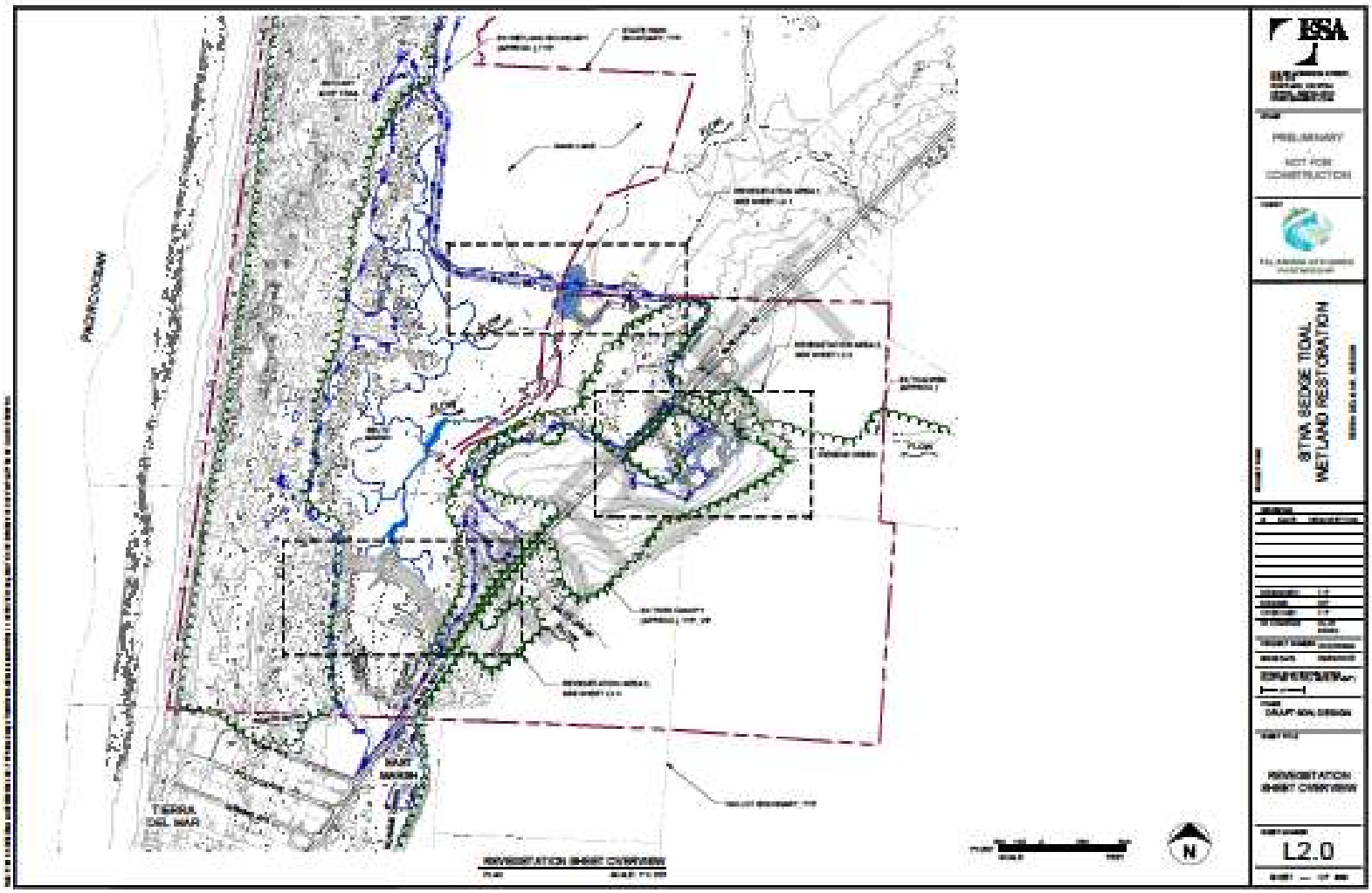
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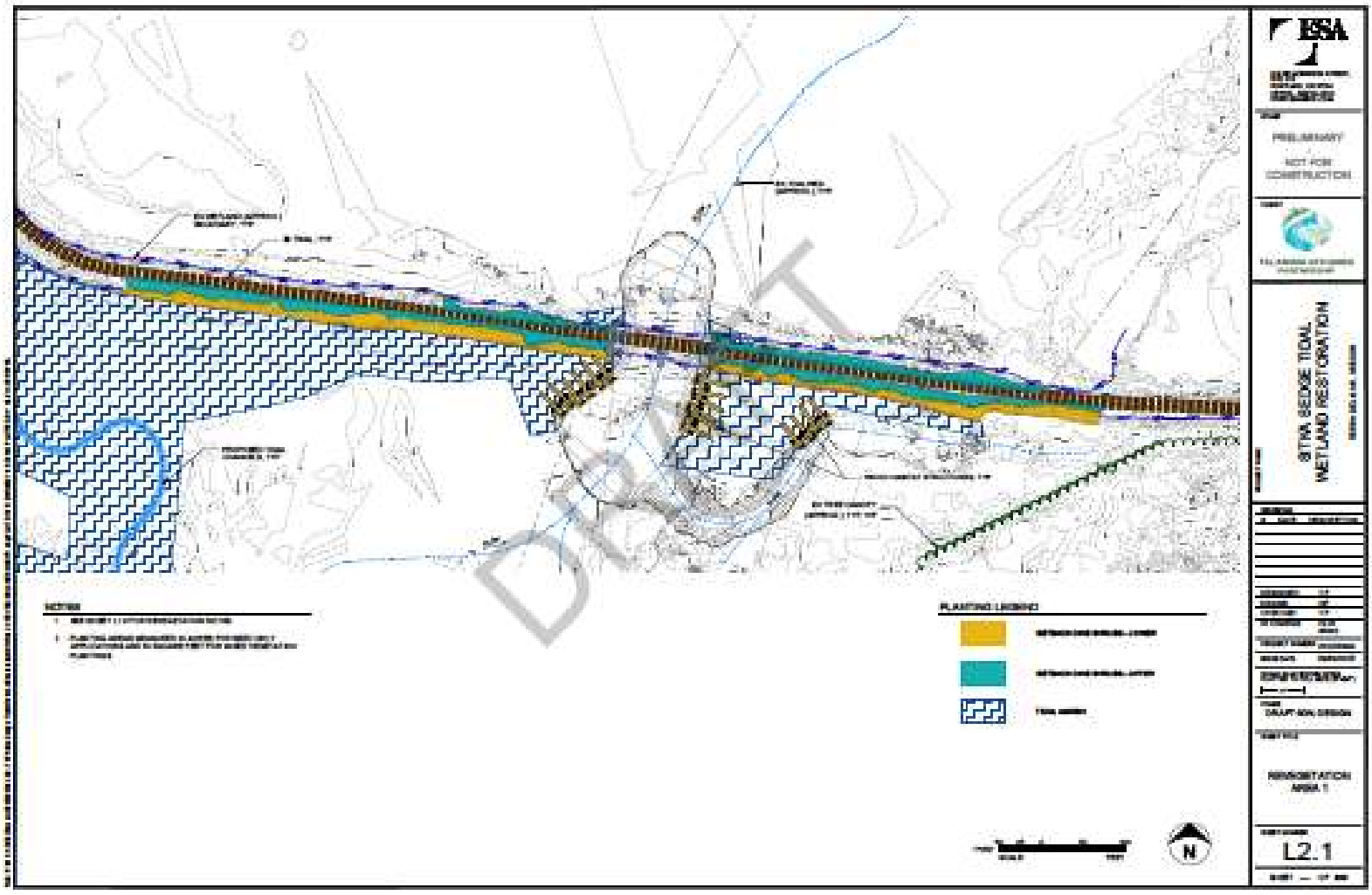
PRELIMINARY
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NOT FOR CONSTRUCTION

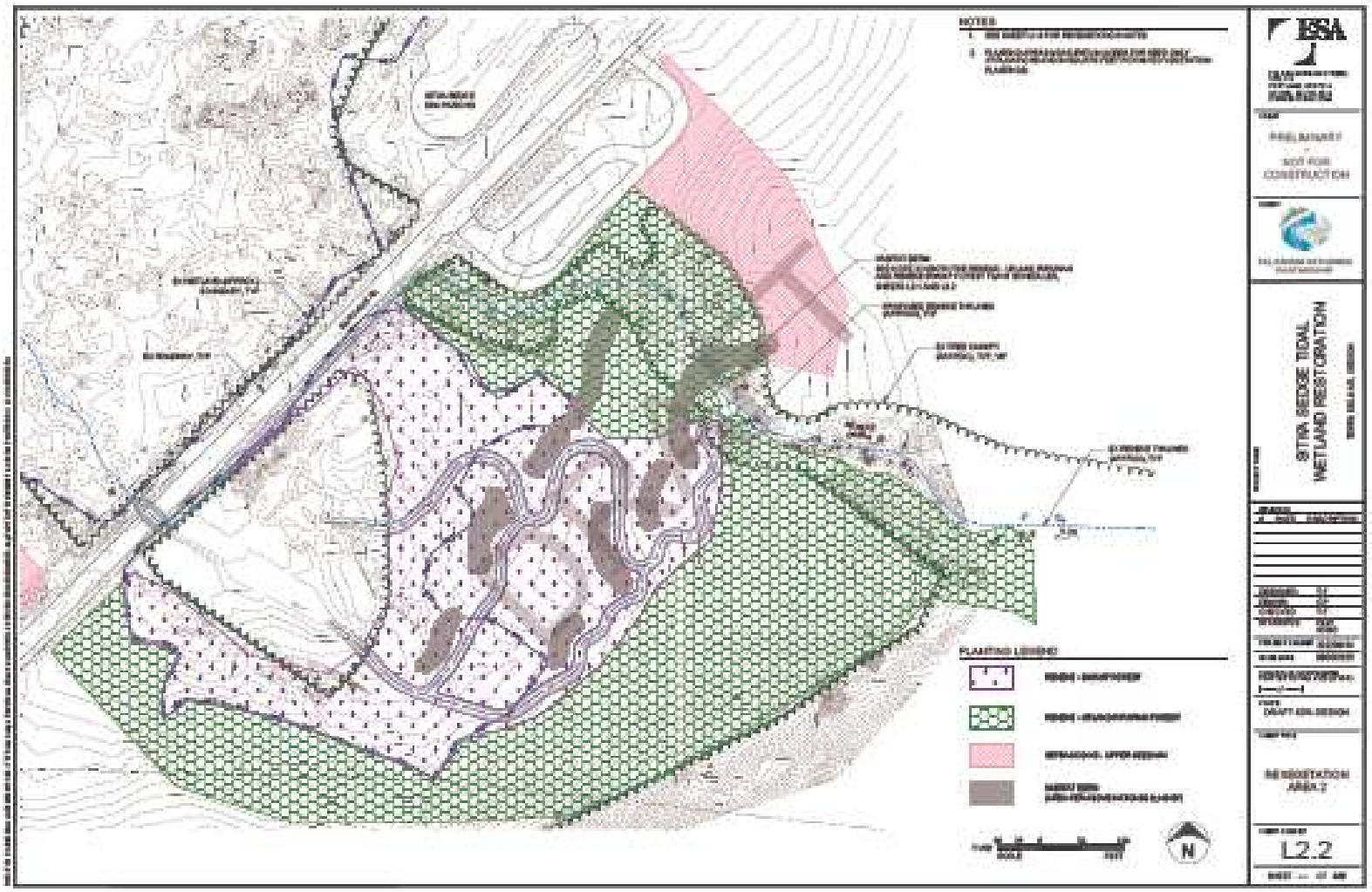
SITKA SEDGE TIDAL WETLAND RESTORATION

REVEGETATION NOTES

L1.0


PACIFIC OCEAN
TERRA DEL MAR
REVEGETATION SHEET OVERVIEW
ESSA
PRELIMINARY
NOT FOR CONSTRUCTION
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT 60% DESIGN
REVEGETATION SHEET OVERVIEW
L2.0


ESSA
PRELIMINARY
NOT FOR CONSTRUCTION
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT
NOTES
PLANTING LEGEND
DRAFT FOR DESIGN
REVEGETATION AREA 1
L2.1
N



ESSA

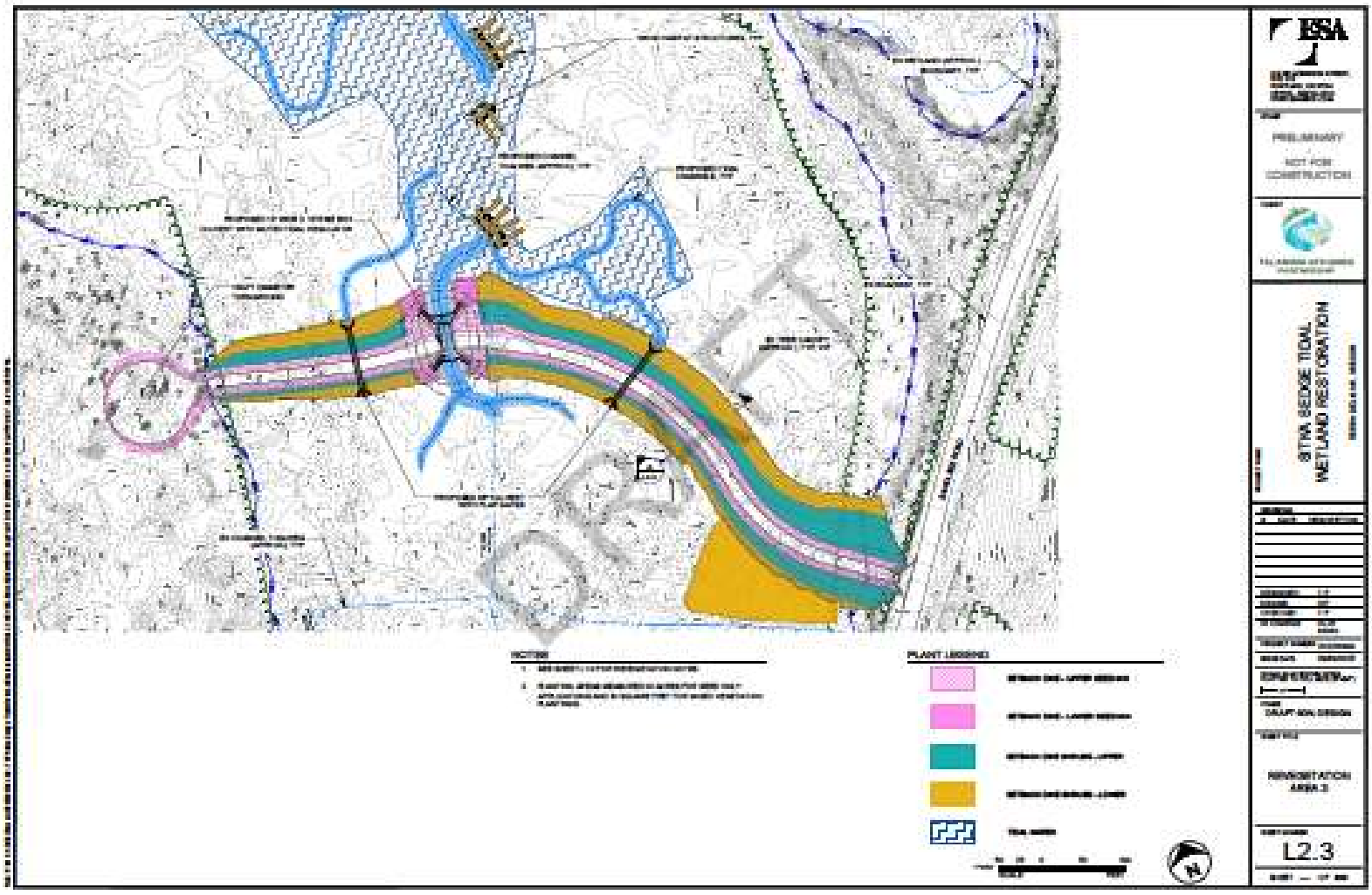
PRELIMINARY
-
NOT FOR CONSTRUCTION

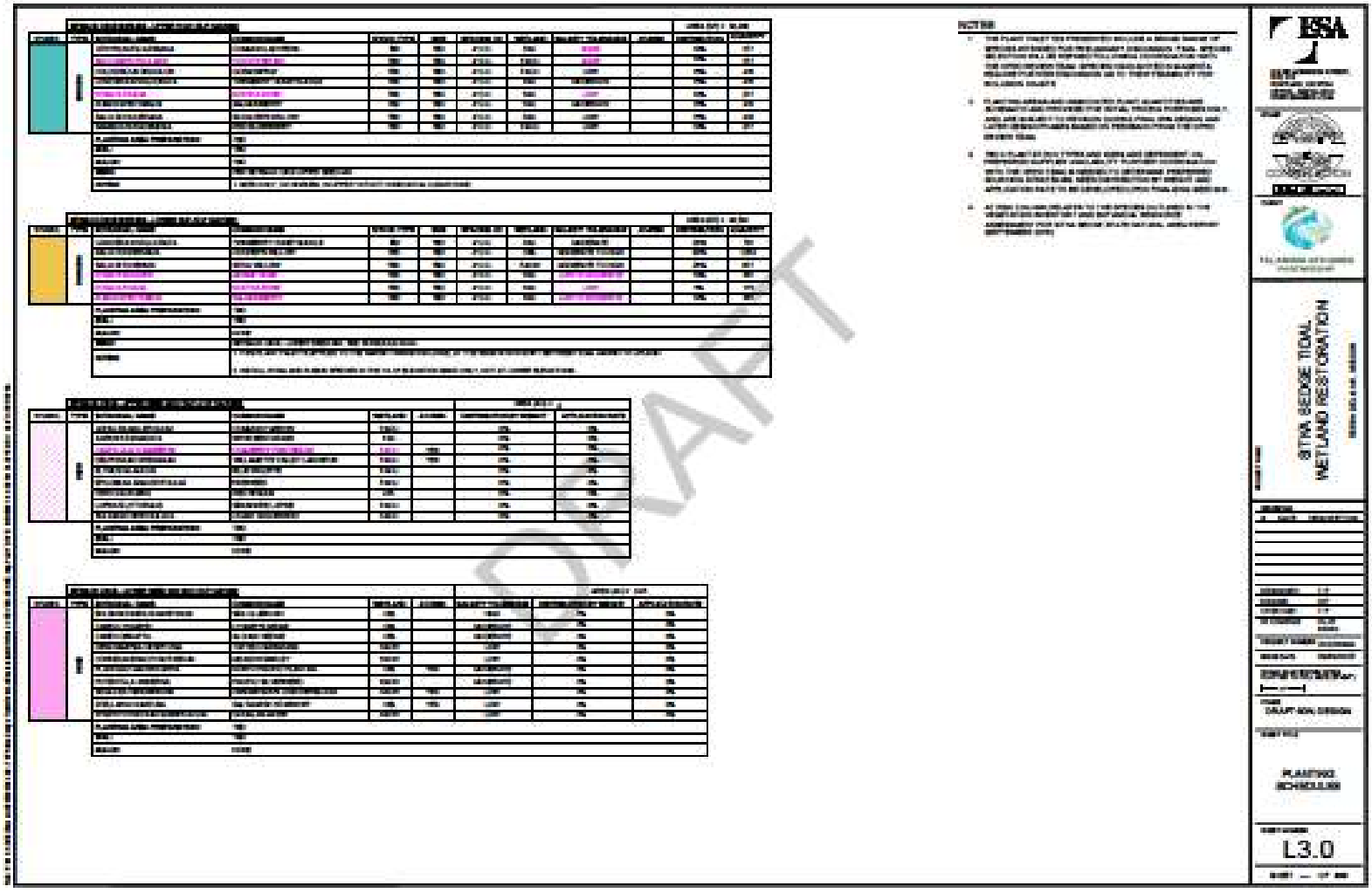
SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT EIA DESIGN

REVEGETATION AREA 2

L2.2


ESSA
PRELIMINARY
NOT FOR CONSTRUCTION
STRA SEDGE TIDAL WETLAND RESTORATION
DRAFT 60% DESIGN
REVEGETATION AREA 3
L2.3



DRAFT

NOTES

1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]

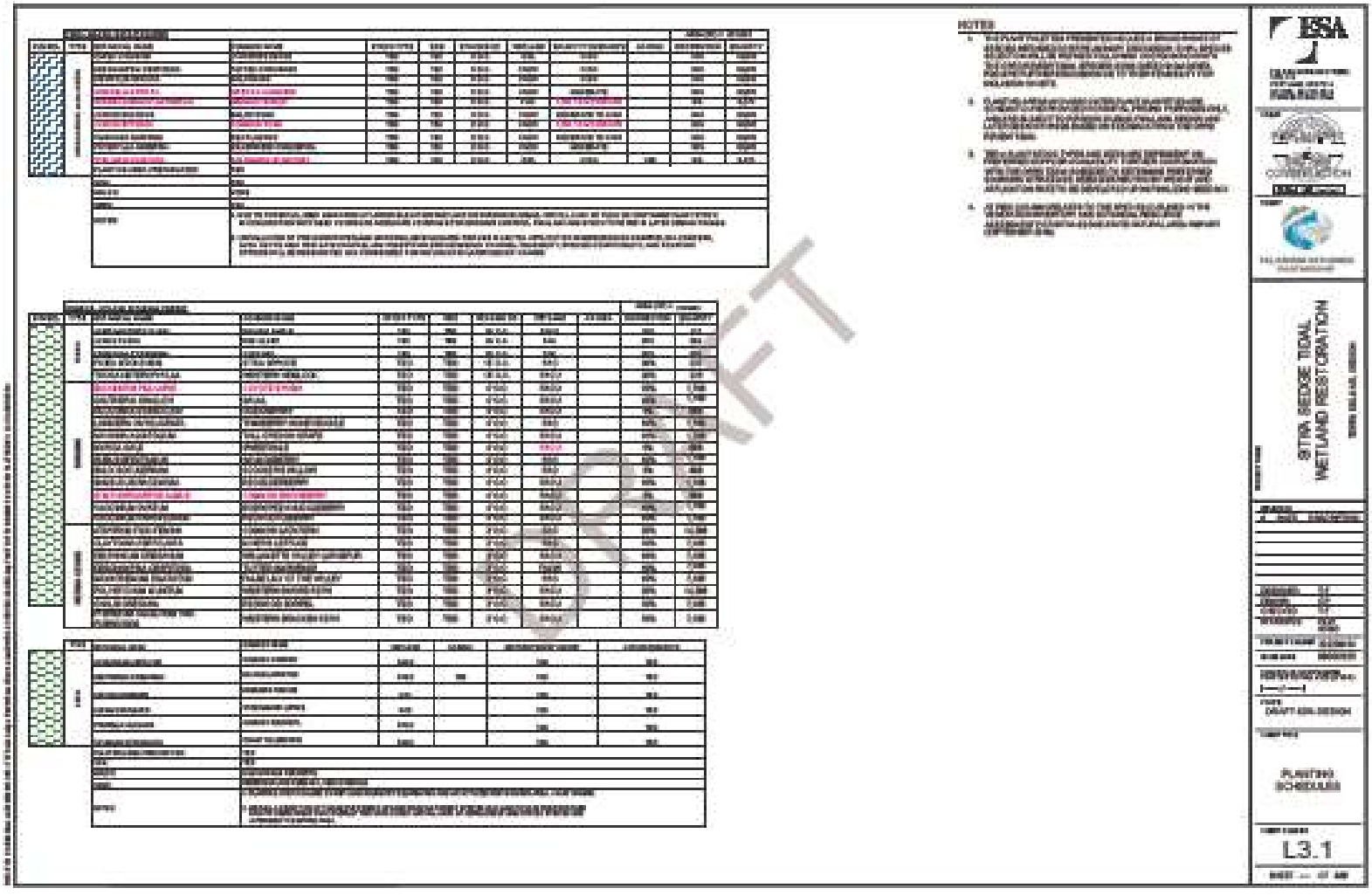
ESSA

STRA SEDGE TIDAL WETLAND RESTORATION

DRAFT-90% DESIGN

PLANTING SCHEDULES

L3.0



NOTES

1.

2.

3.

4.

ESSA

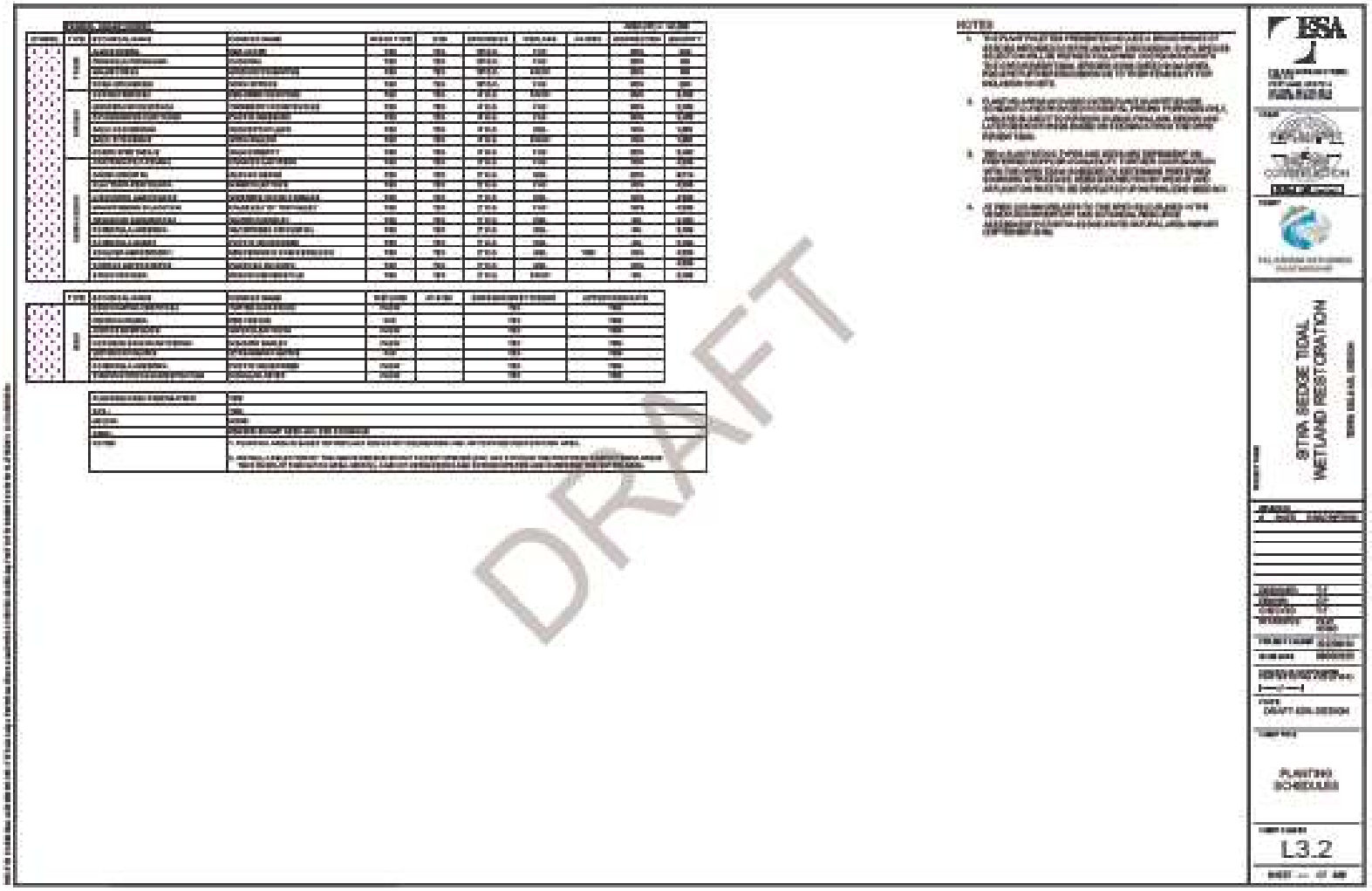
SITE: SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

PLANTING SCHEDULES

L3.1

DRAFT



NOTES

1.

2.

3.

4.

DRAFT

ESSA

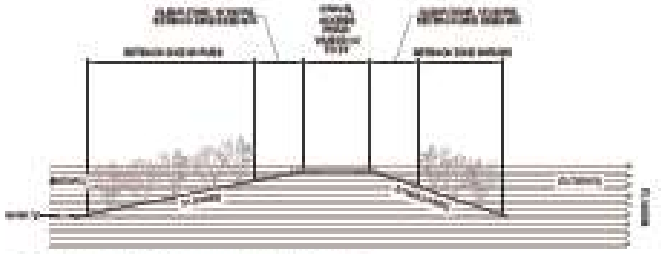
SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

PLANTING SCHEDULES

L3.2

NOTES



SETBACK DIKE TYPICAL CROSS SECTION - PLANTING

DRAFT

ESSA

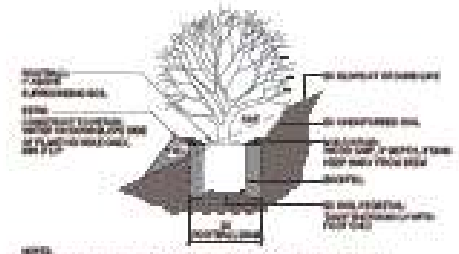
PRELIMINARY
-
NOT FOR CONSTRUCTION

SITKA SEDGE TIDAL WETLAND RESTORATION

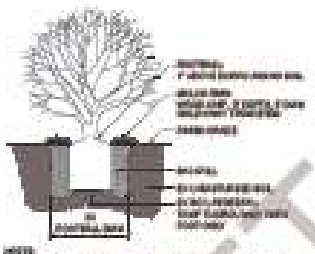
DRAFT ESA DESIGN

TYPICAL PLANTING PROFILES

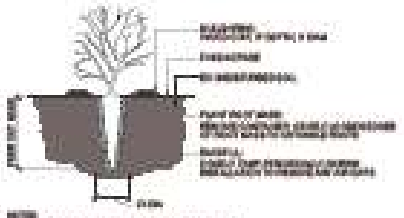
L4.0



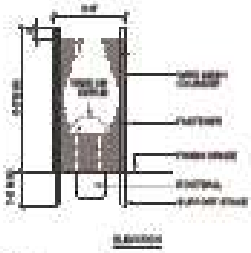
1 CONTAINER PLANTING – ON SLOPE



2 CONTAINER PLANTING – STANDARD



3 CHAPARRAL PLANTING



ELEVATION



4 PROTECTION CAGE

DRAFT

ESSA

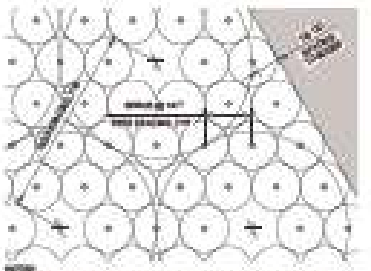
PRELIMINARY
NOT FOR CONSTRUCTION

SITKA SEDGE TIDAL WETLAND RESTORATION

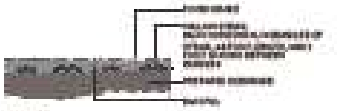
DRAFT 60% DESIGN

PLANTING DETAILS

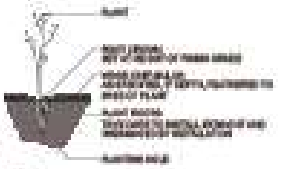
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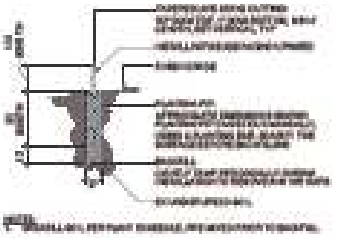
1 TRIANGULAR PLANT SPACING



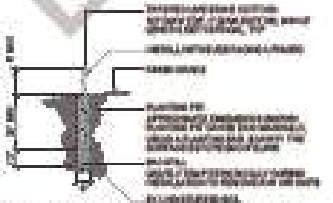
2 HORIZONTAL PLANTING



3 PLUG OR BARE ROOT PLANTING



4 LIVE STAKE PLANTING - STANDARD



5 LIVE STAKE PLANTING - SHORT

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ESSA

PRELIMINARY
-
NOT FOR CONSTRUCTION

SITKA SEDGE TIDAL WETLAND RESTORATION

DRAFT 60% DESIGN

PLANTING DETAILS

L5.1

APPENDIX B: NEARBY FIELD EXPLORATIONS

Appendix B

Nearby Field Explorations

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B.1 GENERAL

Shannon & Wilson explored subsurface conditions at the project site with seven geotechnical borings designated B-1 through B-7. Borings B-1 through B-7 were drilled for the design of the bridges and culvert project features. The locations of the explorations were determined in the field using a GPS, and points were taken to reference the locations. The approximate exploration locations are shown on the Site and Exploration Plan, Figure 2. Additional borings are planned to confirm subsurface information assumed at the dike setback location, with proposed locations shown on Figure 2. This appendix describes the techniques used to perform the explorations for the bridges and culvert (B-1 through B-7) and presents logs of the materials encountered.

B.2 GEOTECHNICAL DRILLING

Borings B-3 through B-7 were drilled within the roadway of Sandlake Road between March 24, 2025, through March 28, 2025. Borings B-1 and B-2 were drilled along the trail on the embankment between March 31, 2025, through April 1, 2025. The borings were drilled using a CME-55LC track rig provided and operated by Western States Soil Conservation Inc. (Western States) of Hubbard, Oregon. A Shannon & Wilson geologist was present during the explorations to locate the borings, observe the drilling, collect soil samples, and log the materials encountered. Below are descriptions of the drilling method used during the drilling.

B.2.1 Open Hole Mud Rotary Drilling

Mud-rotary borings are typically advanced using a tri-cone bit and a string of hollow drill rods (narrower than the bit) through which bentonite drilling mud is pumped. The mud is mixed on-site using water and powdered bentonite. The drilling mud serves to cool the bit, keep the hole open, and flush the cuttings to the surface, and the borehole is considered "open hole" since the borehole is not cased. Returning drill mud is typically passed through a screen and into a tub over the hole. The screen collects the cuttings, and the tub collects the mud for recirculation back into the hole. If fine-grained, cohesive soils are encountered, other styles of drill bits may also be used with the mud-rotary method, such as scraper or drag bits. The common size mud rotary drill bit is 4 7/8-inch, which produces an approximate 5-inch-diameter hole.

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B.3 SOIL SAMPLING

B.3.1 Disturbed Sampling

Disturbed samples were collected in the borings, typically at 2.5- to 5-foot depth intervals, using a standard 2-inch-outside-diameter (O.D.) split spoon sampler in conjunction with Standard Penetration Testing. In a Standard Penetration Test (SPT), ASTM D1586, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance, or N-value. The SPT N-value provides a measure of in situ relative density of cohesionless soils (non-plastic silt, sand, and gravel), and the consistency of cohesive soils (plastic silt and clay). All disturbed samples were visually identified and described in the field, sealed to retain moisture, and returned to our laboratory for additional examination.

SPT N-values can be significantly affected by several factors, including the efficiency of the hammer used. Automatic hammers generally have higher energy transfer efficiencies than cathead-driven (manual) hammers. For reference, cathead hammers are typically assumed to have an average energy efficiency of 60 percent. Based on information we received from Western States, the energy efficiency of the hammer used on the rubber track-mounted CME-55LC drill rig averaged 85.0 percent. All N-values presented in this report are in blows per foot, as counted in the field. No corrections have been applied.

An SPT was considered to have met refusal where more than 50 blows were required to drive the sampler 6 inches. If refusal was encountered in the first 6-inch interval (for example, 50 for 1.5 inches), the count is reported as 50/1st 1.5". If refusal was encountered in the second 6-inch interval (for example, 48, 50 for 1.5 inches), the count is reported as 50/1.5". If refusal was encountered in the last 6-inch interval (for example, 39, 48, 50 for 1.5 inches), the count is reported as 98/7.5".

B.3.2 Relatively Undisturbed Sampling

Relatively undisturbed samples were attempted in 3-inch O.D. thin-wall Shelby tubes, which were hydraulically pushed into the soil at the bottom of the boreholes. The recovered soils exposed at the ends of the tube were examined and described in the field. After examination, the tubes were sealed to preserve the natural moisture of the samples. The sealed tubes were stored in the upright position, and care was taken to avoid shock and vibration during their transport to and storage in our laboratory.

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B.4 MATERIAL DESCRIPTIONS

In the field, soil samples were described and identified visually in general accordance with ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Consistency, color, relative moisture, degree of plasticity, peculiar odors, and other distinguishing characteristics of the samples were noted. Once returned to the laboratory, soil samples were re-examined, various laboratory tests were performed, and the field descriptions and identifications were modified as necessary. We refined our visual-manual soil descriptions and identifications based on the results of the laboratory tests, using elements of the Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), ASTM D2487. However, ASTM D2487 was not followed in full because it requires a suite of tests to be performed to classify a single sample. Select soil descriptions were also updated on the boring logs based on the additional round of laboratory testing that was performed on select samples remaining in storage for the dike setback design. The specific terminology used in the soil classifications is defined in the Soil Description and Log Key, Figure B1. The specific terminology used in the rock classifications is defined in the Rock Classification and Log Key, Figure B2.

B.5 LOGS OF BORINGS

The summary logs of Shannon & Wilson geotechnical borings are presented in Figures B3 through B9. Material descriptions and interfaces on the logs are interpretive, and actual changes may be gradual. The left-hand portion of the boring logs provides descriptions, identifications, and geotechnical unit designations for the materials encountered in the boring. The right-hand portion of the boring logs shows a graphic log, sample locations and designations, backfill details, and a graphical representation of N-values, natural water contents, percent passing the #200 sieve, and sample recovery.

B.6 BOREHOLE ABANDONMENT

Upon completion of drilling, the borings were backfilled with bentonite chips or bentonite/cement grout in accordance with Oregon Department of Water Resources regulations. Cold patch asphalt pavement was used to seal the surface of the borings performed on Sandlake Road.

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Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT[1]	FINE-GRAINED SOILS (50% or more fines)[2]	COARSE-GRAINED SOILS (less than 50% fines)[2]
Major	*Silt, Lean Clay, Elastic Silt,* or *Fat Clay*[3]	*Sand* or *Gravel*[4]
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: *Sandy* or *Gravelly*[4]	More than 12% fine-grained: *Silty* or *Clayey*[3]
Minor Follows major constituent	15% to 30% coarse-grained: *with Sand* or *with Gravel*[4]	5% to 12% fine-grained: *with Silt* or *with Clay*[3]
	30% or more total coarse-grained and lesser coarse-grained constituent is 15% or more: *with Sand* or *with Gravel*[4]	15% or more of a second coarse-grained constituent: *with Sand* or *with Gravel*[4]

[1] All percentages are by weight of total specimen passing a 3-inch sieve.
[2] The order of terms is: *Modifying Major with Minor*.
[3] Determined based on behavior.
[4] Determined based on which constituent comprises a larger percentage.
[5] Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.

NOTE: Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

PARTICLE SIZE DEFINITIONS

DESCRIPTION	SIEVE NUMBER AND/OR APPROXIMATE SIZE
FINES	< #200 (0.075 mm = 0.003 in.)
SAND Fine Medium Coarse	#200 to #40 (0.075 to 0.4 mm; 0.003 to 0.02 in.) #40 to #10 (0.4 to 2 mm; 0.02 to 0.08 in.) #10 to #4 (2 to 4.75 mm; 0.08 to 0.187 in.)
GRAVEL Fine Coarse	#4 to 3/4 in. (4.75 to 19 mm; 0.187 to 0.75 in.) 3/4 to 3 in. (19 to 76 mm)
COBBLES	3 to 12 in. (76 to 305 mm)
BOULDERS	> 12 in. (305 mm)

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS		COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 4	Very loose	< 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
> 50	Very dense	15 - 30	Very stiff
		> 30	Hard

WELL AND BACKFILL SYMBOLS

	Bentonite Cement Grout		Surface Cement Seal
	Bentonite Grout		Asphalt or Cap
	Bentonite Chips		Slough
	Silica Sand		Inclinometer or Non-perforated Casing
	Gravel		Vibrating Wire Piezometer
	Perforated or Screened Casing		

PERCENTAGES TERMS [1,2]

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

[1] Gravel, sand, and fines estimated by mass. Other constituents, such as organics, cobbles, and boulders, estimated by volume.

[2] Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

SOIL DESCRIPTION AND LOG KEY

May 2025 112880

SHANNON & WILSON

FIG. B1
Sheet 1 of 3

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2013_SD RING_CLASS1_112880.GPJ S&W2013L.GDT 5/5/25

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488)

MAJOR DIVISIONS			GROUP SYMBOL	GRAPHIC SYMBOL	TYPICAL IDENTIFICATIONS
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Gravel *(less than 5% fines)*	GW		Well-Graded Gravel; Well-Graded Gravel with Sand
			GP		Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel *(more than 12% fines)*	GM		Silty Gravel; Silty Gravel with Sand
			GC		Clayey Gravel; Clayey Gravel with Sand
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Sand *(less than 5% fines)*	SW		Well-Graded Sand; Well-Graded Sand with Gravel
			SP		Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand *(more than 12% fines)*	SM		Silty Sand; Silty Sand with Gravel
			SC		Clayey Sand; Clayey Sand with Gravel
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML		Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL		Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL		Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH		Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH		Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH		Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT		Peat or other highly organic soils (see ASTM D4427)
FILL	Placed by humans, both engineered and nonengineered. May include various soil materials and debris.				The Fill graphic symbol is combined with the soil graphic that best represents the observed material

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.
2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.
3. The soil graphics above represent the various USCS identifications (i.e., *GP*, *SM*, etc.) and may be augmented with additional symbology to represent differences within USCS designations. *Sandy Silt (ML)*, for example, may be accompanied by the *ML* soil graphic with sand grains added. Non-USCS materials may be represented by other graphic symbols; see log for descriptions.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

SOIL DESCRIPTION AND LOG KEY

May 2025 112880

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FIG. B1
Sheet 2 of 3

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2013_SDRWG_CLASS2_112880.01.01 SW2015_BRANYPOX.GLB SWILSON.CTT 5/5/25

GRADATION TERMS

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

CEMENTATION TERMS[1]

Weak	Crumbles or breaks with handling or slight finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

PLASTICITY[2]

DESCRIPTION	VISUAL-MANUAL CRITERIA	APPROX. PLASITICTY INDEX RANGE
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.	< 4%
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 to 10%
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 to 20%
High	It take considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	> 20%

ADDITIONAL TERMS

Mottled	Irregular patches of different colors.
Bioturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

PARTICLE ANGULARITY AND SHAPE TERMS[1]

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

ACRONYMS AND ABBREVIATIONS

ATD	At Time of Drilling
approx.	Approximate/Approximately
Diam.	Diameter
Elev.	Elevation
ft.	Feet
FeO	Iron Oxide
gal.	Gallons
Horiz.	Horizontal
HSA	Hollow Stem Auger
I.D.	Inside Diameter
in.	Inches
lbs.	Pounds
MgO	Magnesium Oxide
mm	Millimeter
MnO	Manganese Oxide
NA	Not Applicable or Not Available
NP	Nonplastic
O.D.	Outside Diameter
OW	Observation Well
pcf	Pounds per Cubic Foot
PID	Photo-Ionization Detector
PMT	Pressuremeter Test
ppm	Parts per Million
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
rpm	Rotations per Minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
q_u	Unconfined Compressive Strength
VWP	Vibrating Wire Piezometer
Vert.	Vertical
WOH	Weight of Hammer
WOR	Weight of Rods
Wt.	Weight

STRUCTURE TERMS[1]

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

[1]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.
[2]Adapted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

SOIL DESCRIPTION AND LOG KEY

May 2025 112880

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FIG. B1
Sheet 3 of 3

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BASED ON INTERNATIONAL SOCIETY FOR ROCK MECHANICS (ISRM) ROCK CLASSIFICATION METHODS[a]

FABRIC TERMS

SEDIMENTARY ROCKS

MASSIVE - Rock without significant structure

BEDDED - Regular layering from sedimentation

FISSILE - Tendency to break along laminations

METAMORPHIC ROCKS

FOLIATED - Parallel arrangement or distribution of minerals

SCHISTOSE - Parallel arrangement of tabular minerals giving a planar fissility

GNEISSOSE - Segregation of minerals into bands

CLEAVAGE - Tendency to split along secondary, planar textures or structures

STRENGTH

GRADE	DESCRIPTION	FIELD IDENTIFICATION	APPROXIMATE RANGE OF UNIAXIAL COMPRESSIVE STRENGTH (MPa)	(psi)
R0	Extremely Weak Rock	Indented by thumbnail	0.25 to 1	36 to 145
R1	Very Weak Rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1 to 5	145 to 700
R2	Weak Rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5 to 25	700 to 3,600
R3	Medium Strong Rock	Cannot be scraped or peeled by a pocket knife, specimen can be fractured with single firm blow of geological hammer	25 to 50	3,600 to 7,200
R4	Strong Rock	Specimen requires more than one blow of geological hammer to fracture it	50 to 100	7,200 to 14,500
R5	Very Strong Rock	Specimen requires many blows of geological hammer to fracture it	100 to 250	14,500 to 36,250
R6	Extremely Strong Rock	Specimen can only be chipped with geological hammer	>250	>36,250

VESCULARITY

Slightly Vesicular	1 to 10%
Moderately Vesicular	10 to 30%
Highly Vesicular	30 to 50%
Scoriaceous	>50%

JOINT ROUGHNESS

SMALL SCALE	INTERMEDIATE SCALE
Rough	Stepped
Smooth	Undulating
Slickensided	Planar

WEATHERING

TERM	DESCRIPTION
Fresh	No visible signs of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.
Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and somewhat weaker than in its fresh condition.
Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
Highly Weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely Weathered	All rock is decomposed and/or disintegrated to soil. The original mass is still largely intact.
Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

DISCONTINUITY TERMS

FRACTURE - Collective term for any natural break excluding shears, shear zones, and faults

JOINT (JT) - Planar break with little or no displacement

FOLIATION JOINT (FJ) or BEDDING JOINT (BJ) - Joint along foliation or bedding

INCIPIENT JOINT (IJ) or INCIPIENT FRACTURE (IF) - Joint or fracture not evident until wetted and dried; breaks along existing surface

RANDOM FRACTURE (RF) - Natural, very irregular fracture that does not belong to a set

BEDDING PLANE SEPARATION or PARTING - A separation along bedding after extraction from stress relief or slaking

FRACTURE ZONE (FZ) - Planar zone of broken rock without gouge

MECHANICAL BREAK (MB) - Breaks due to drilling or handling; drilling break (DB), hammer break (HB)

SHEAR (SH) - Surface of differential movement evident by presence of slickensides, striations, or polishing

SHEAR ZONE (SZ) - Zone of gouge and rock fragments bounded by planar shear surfaces

FAULT (FT) - Shear zone of significant extent; differentiation from shear zone may be site-specific

STRUCTURE SPACING TERMS

STRATIGRAPHIC	SPACING	DISCONTINUITY *
Extremely Thick	> 20 ft. (> 6 m)	Extremely Wide
Very Thick	6 to 20 ft. (2 to 6 m)	Very Wide
Thick	2 to 6 ft. (0.6 to 2 m)	Wide
Medium	8 to 24 in. (0.2 to 0.6 m)	Moderate
Thin	2.5 to 8 in. (60 to 200 mm)	Close
Very Thin	1 to 2.5 in. (20 to 60 mm)	Very Close
Laminated: Thickly	0.25 to 1 in. (6 to 20 mm)	Extremely Close
Laminated: Thinly	<0.25 in. (<6 mm)	Extremely Close

* Refers to apparent spacing along core axis unless measured orthogonal to discontinuity; should then report for each set

[a] Reference: Brown, E.T., ed., 1981, Rock Characterization Testing and Monitoring ISRM Suggested methods: New York, International Society for Rock Mechanics (ISRM).

DRAFT

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

ROCK CLASSIFICATION AND LOG KEY

May 2025 112880

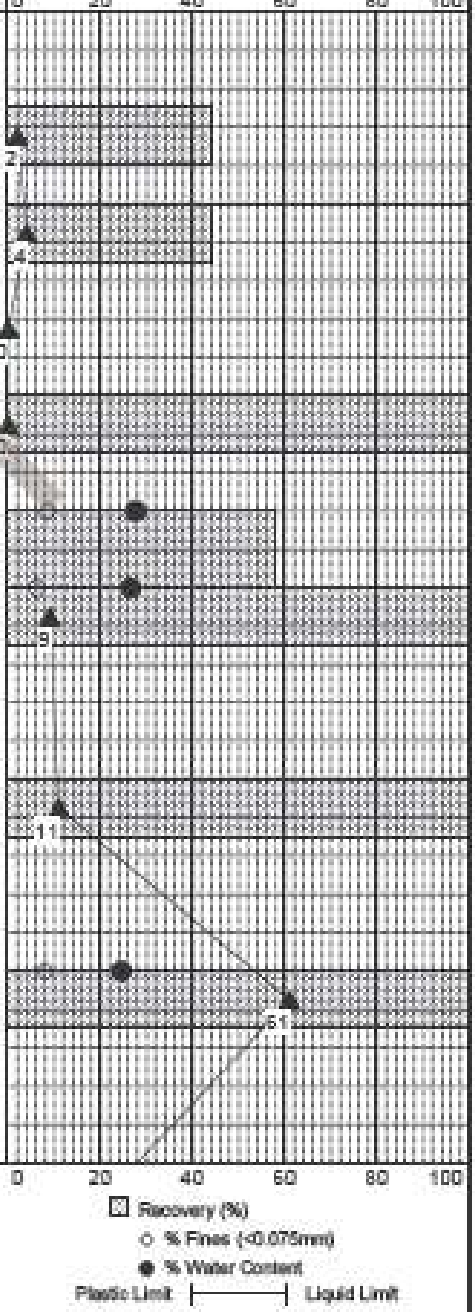
SHANNON & WILSON FIG. B2

Filename: C:\S & W Tech Forms\Rock Classification - LogKey.dwg Layout: Layout1 Date: 05-12-2008 Login: Seth Sonnier

Total Depth:	76.5 ft.	Northing:	~ 601,333 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,095 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. / Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
Gravel Pavement					
Very loose to loose, light brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coarse, subangular gravel; fine to medium sand; low plasticity fines; micaceous; slight iron oxide staining. **EMBANKMENT FILL**	12.5 2.0		S-1 S-2 S-3		
Very soft, dark gray, *Silty Sand (SM)*; wet; fine to coarse sand; low plasticity fines; trace organics; shell fragments; micaceous.	4.5 10.0		S-4		
Loose to very dense, gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace fine, subrounded gravel; fine to medium sand; nonplastic fines; micaceous. **FLUVIAL AND ESTUARIAN DEPOSITS**	2.0 12.5		S-5 S-6 S-7 S-8		

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND
- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-1

January 2026 | 112880

SHANNON & WILSON | FIG. B3 Sheet 1 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2015.GRAYPDX.GLB SHANNON_PDX.GDT 3/31/26 Log: SCF Rev: SCS Typ: SCF

Total Depth:	78.5 ft.	Northing:	~ 601,333 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,095 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. (ft.) | Depth | Symbol | Samples | Ground Water | Depth, ft

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

0 20 40 60 80 100

Continued:

Loose to very dense, gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace fine, subrounded gravel; fine to medium sand; nonplastic fines; micaceous.

-18.0 / 32.5

FLUVIAL AND ESTUARIAN DEPOSITS

Gray, *Elastic Silt (MH)*; wet; fine to medium sand; low plasticity; micaceous.

-21.0 / 35.5

Harder drilling action at 35.5 feet.

Dense, red-brown mottled, *Silty Gravel with Sand (GM)*; moist; fine to coarse, subangular to subrounded; fine to coarse sand; low plasticity fines; disturbed texture; heavy iron oxide staining.

-25.5 / 40.0

ALLUVIAL FAN DEPOSITS

Dense, Yellow-brown to dark gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coarse, subangular to subrounded gravel; fine to medium sand; low plasticity fines; micaceous; moderate iron oxide staining.

FLUVIAL AND ESTUARIAN DEPOSITS

-38.0 / 52.5

Medium dense to very dense, brown to gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coarse, subangular to subrounded gravel; fine to medium sand; nonplastic fines; micaceous.

S-9 S-10A S-10B S-11 S-12 S-13 S-14

22 25 38 51 54 76

35 40 45 50 55

CONTINUED NEXT SHEET

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-1

January 2026 112880

SHANNON & WILSON

FIG. B3
Sheet 2 of 3

REV 3

DRAFT

MASTER LOG_E 112880.GPJ SHN2015.GPJ PDX_DLIB SHANNA_PDX.GDT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	76.5 ft.	Northing:	~ 601,333 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,095 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft	PENETRATION RESISTANCE, N (blows/ft.) ▲ Hammer Wt. & Drop: 140 lbs / 30 inches
Continued: Medium dense to very dense, brown to gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coarse, subangular to subrounded gravel; fine to medium sand; nonplastic fines; micaceous. *Gravel interbeds inferred from drilling action.* **FLUVIAL AND ESTUARIAN DEPOSITS**			S-15 S-16		65	20 15
Medium dense to dense, gray to gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; low plasticity fines; stratified with interbeds of *Silty Sand (SM)*; micaceous.	-53.0 67.5		S-17 S-18		70 75	33 26
Completed: March 31, 2025	-62.0 76.5				80 85	

0 20 40 60 80 100

0 20 40 60 80 100

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-1

January 2026 112880

SHANNON & WILSON

FIG. B3
Sheet 3 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2015.GRAY.PDX.GLB SHANNA_PDX.LOG.DT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	66.5 ft.	Northing:	~ 601,319 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 14 ft.	Easting:	~ 7,312,217 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

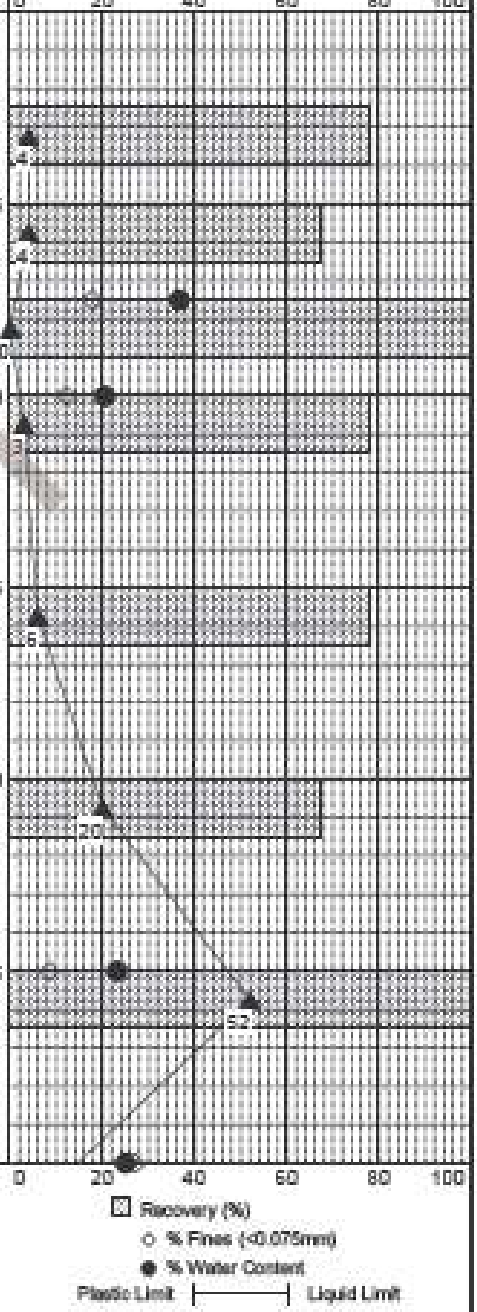
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. / Depth (ft.)	Soil Description
	Gravel Pavement
11.5 / 2.5	Very loose to loose, brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium; low plasticity fines; micaceous; pockets of *Silty Sand (SM)*. **EMBANKMENT FILL**
6.5 / 7.5	Very loose, gray, *Silty Sand (SM)*; wet; fine to medium; nonplastic fines; micaceous.
4.5 / 9.5	Very loose to loose, gray, *Silty Sand (SM)*; wet; fine sand; nonplastic fines; micaceous. **FLUVIAL AND ESTUARIAN DEPOSITS**
-3.5 / 17.5	Medium dense to very dense, gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coarse, subangular to subrounded gravel; fine to medium sand; nonplastic fines; stratified with interbeds of *Poorly Graded Gravel with Silt and Sand (GP-GM)*; micaceous.
-13.5 / 27.5	Loose, gray to brown, *Silty Sand (SM)*; wet; few to little, fine to coarse, subangular to subrounded gravel; fine to medium sand.

Symbol | Samples: S-1, S-2, S-3, S-4, S-5, S-6, S-7 | Ground Water | Depth, ft

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 Inches



CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered

I Standard Penetration Test

Recovery (%)

○ % Fines (<0.075mm)

● % Water Content

Plastic Limit ⊢——⊣ Liquid Limit

DRAFT

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-2

January 2026 | 112880

SHANNON & WILSON | FIG. B4 | Sheet 1 of 3

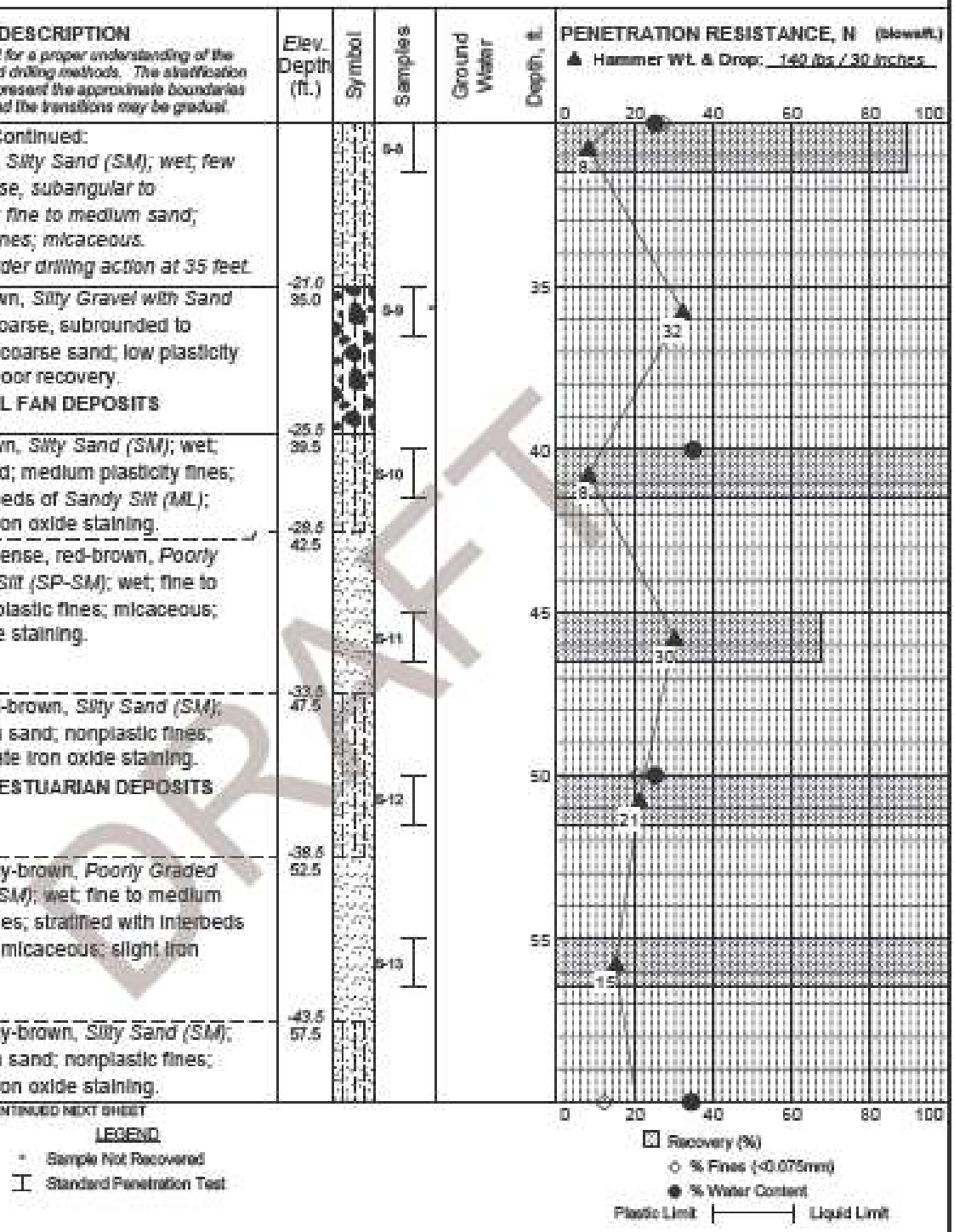
REV 3

MASTER LOG_E 112880.GPJ SW2013_BRARY.PDX.GLB SHANNON_PDX.GLB DT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	58.5 ft.	Northing:	~ 601,319 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 14 ft.	Easting:	~ 7,312,217 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. / Depth (ft.)
Continued: Loose, gray-brown, *Silty Sand (SM)*; wet; few to little, fine to coarse, subangular to subrounded gravel; fine to medium sand; medium plasticity fines; micaceous. *Harder drilling action at 35 feet.*	
Dense, gray to brown, *Silty Gravel with Sand (GM)*; wet; fine to coarse, subrounded to subangular; fine to coarse sand; low plasticity fines; micaceous; poor recovery. **ALLUVIAL FAN DEPOSITS**	-21.0 35.0
Loose, gray to brown, *Silty Sand (SM)*; wet; fine to medium sand; medium plasticity fines; stratified with interbeds of *Sandy Silt (ML)*; micaceous; slight iron oxide staining.	-25.5 39.5
Medium dense to dense, red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; micaceous; moderate iron oxide staining.	-28.5 42.5
Medium dense, red-brown, *Silty Sand (SM)*; wet; fine to medium sand; nonplastic fines; micaceous; moderate iron oxide staining. **FLUVIAL AND ESTUARIAN DEPOSITS**	-33.5 47.5
Medium dense, gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; stratified with interbeds of *Silty Sand (SM)*; micaceous; slight iron oxide staining.	-38.5 52.5
Medium dense, gray-brown, *Silty Sand (SM)*; wet; fine to medium sand; nonplastic fines; micaceous; slight iron oxide staining.	-43.5 57.5

CONTINUED NEXT SHEET



LEGEND

* Sample Not Recovered

Standard Penetration Test

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-2

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SHANNON & WILSON

FIG. B4
Sheet 2 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2015.GRAYPDX.GLB SHANNON_ROLLOUT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	88.5 ft.	Northing:	~ 601,319 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 14 ft.	Easting:	~ 7,312,217 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft	PENETRATION RESISTANCE, N (blows/ft.) ▲ Hammer Wt. & Drop: 140 lbs / 30 Inches
Continued: Medium dense, gray-brown, *Silty Sand (SM)*; wet; fine to medium sand; nonplastic fines; micaceous; slight iron oxide staining.	-48.5 62.5		S-14		65	21
Medium dense, gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; stratified with interbeds of *Silty Sand (SM)*; micaceous; slight iron oxide staining.			S-15			21
FLUVIAL AND ESTUARIAN DEPOSITS			S-16		70	25
			S-17		75	24
Dense, gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; micaceous; moderate iron oxide staining.	-68.5 82.5				80	
			S-18		85	33
Completed: April 1, 2025	-72.5 86.5					

0 20 40 60 80 100

LEGEND
- * Sample Not Recovered
- I Standard Penetration Test
- ☒ Recovery (%)
- ○ % Fines (<0.075mm)
- ● % Water Content
- Plastic Limit ⊢—⊣ Liquid Limit

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-2

January 2026 112880

SHANNON & WILSON

FIG. B4
Sheet 3 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2015.GRAYPDX.GLB SHANNON_PDX.GDT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	~ 600,349 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,476 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.) | Symbol | Samples | Ground Water | Depth, ft

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 Inches

0 | 20 | 40 | 60 | 80 | 100

15.0 — Asphalt: 4 inches thick

14.7 — Base Aggregate

13.8 — Very soft, dark brown, *Organic Soil (OH)*; wet; trace, fine, subrounded to rounded gravel; trace to few, fine to coarse sand; high plasticity; few to little organics.

MARSH DEPOSITS

7.8 — Very loose, yellow-brown, *Silty Gravel (GM)*; wet; fine to coarse, subrounded to rounded gravel; fine to coarse sand; low to medium plasticity fines; micaceous.

5.8 — Gray, *Elastic Silt (MH)*; wet; trace, fine to coarse sand; medium to high plasticity; slight iron oxide staining.

4.3 — Loose to medium dense, gray to brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines; micaceous.

FLUVIAL AND ESTUARIAN DEPOSITS

-4.7 — Medium dense to very dense, brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace, fine to coarse, subrounded to rounded gravel; fine to coarse sand; medium plasticity fines; micaceous.

S-1 | S-2 | S-3 | S-4 | S-5 | S-6 | S-7

0 | 5 | 10 | 15 | 20 | 25

2 | 30 | 6 | 25 | 13 | 28

CONTINUED NEXT SHEET

LEGEND

I Standard Penetration Test

☒ Recovery (%)

○ % Fines (<0.075mm)

● % Water Content

Plastic Limit ⊢——⊣ Liquid Limit

DRAFT

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-3

January 2026 | 112880

SHANNON & WILSON | FIG. B5 Sheet 1 of 3

REV 3

MASTER LOG_E 112880.GPJ SHN2015.GPJ SHANNON_ROLLOUT 3/31/26 Log: SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	~ 600,349 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,476 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft	PENETRATION RESISTANCE, N (blows/ft.)
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.						▲ Hammer Wt. & Drop: 140 lbs / 30 Inches
Continued: Medium dense to very dense, brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse sand; medium plasticity fines; micaceous.			S-8			56
			S-9		35	57
FLUVIAL AND ESTUARIAN DEPOSITS			S-10		40	32
			S-11		45	16
			S-12		50	20
Dense, brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine, subrounded to rounded gravel; fine to coarse sand; nonplastic fines; micaceous; heavy iron oxide staining.	-37.2 52.5		S-13		55	13

0 20 40 60 80 100

CONTINUED NEXT SHEET

LEGEND

I Standard Penetration Test

☒ Recovery (%)

○ % Fines (<0.075mm)

● % Water Content

Plastic Limit ⊢——⊣ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-3

January 2026 112880

SHANNON & WILSON

FIG. B5
Sheet 2 of 3

REV 3

DRAFT

MASTER LOG_E 112880.GPJ SHN2015.GRAY.PDX.GLB SHANNON_ROLLOUT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	~ 600,349 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,476 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.) | Symbol | Samples | Ground Water | Depth, ft | PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 Inches

Continued:

Dense, brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few fine, subrounded to rounded gravel; fine to coarse; nonplastic fines; micaceous; heavy iron oxide staining.

-48.7 / 64.0

FLUVIAL AND ESTUARIAN DEPOSITS

Harder drilling action at 64 feet.

SILTSTONE/SANDSTONE: very weak to weak (R1-R2); moderately weathered; joint structure indiscernible in SPT sample.

ALSEA FORMATION

-54.8 / 70.1

Completed: March 24, 2025

S-14: 39

S-15: 50/1st 3"

S-16: 50/1st 1"

0 20 40 60 80 100

65 70 75 80 85

LEGEND

I Standard Penetration Test

☒ Recovery (%)

○ % Fines (<0.075mm)

● % Water Content

Plastic Limit ⊢—⊣ Liquid Limit

DRAFT

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-3

January 2026 | 112880

SHANNON & WILSON | FIG. B5 Sheet 3 of 3

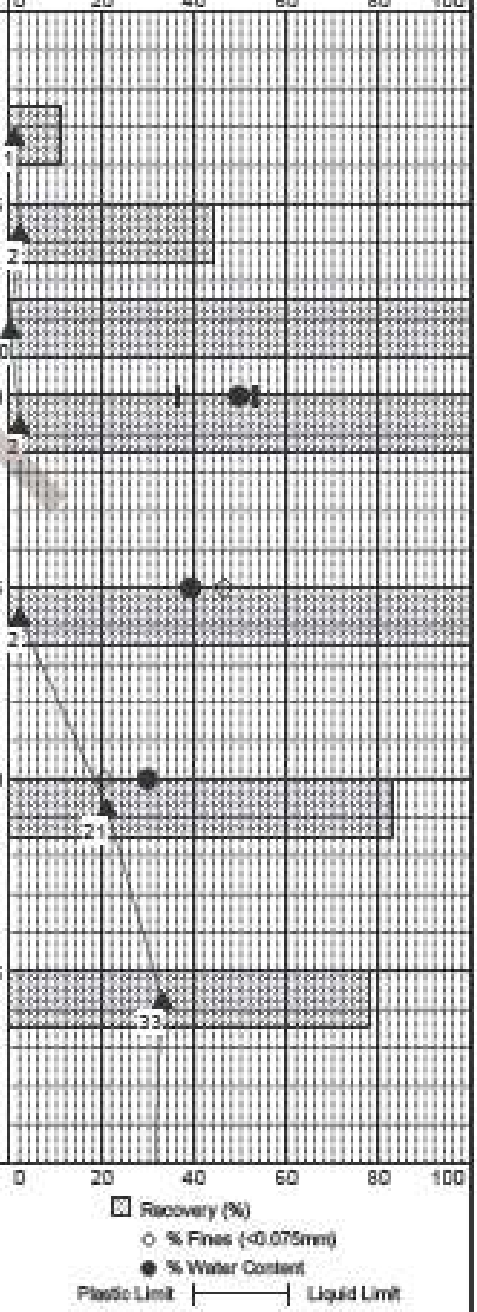
REV 3

MASTER_LOG_E 112880.GPJ SW2013_BRARY.PDX.GLB SHANNA_PDX.GLB 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	~ 600,357 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,606 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft
Asphalt Concrete	14.7 0.3				
Base Aggregate	13.5 1.5				
Very soft, dark brown, *Organic Soil (OH)*; wet; fine to coarse sand; high plasticity; few to little organics. **MARSH DEPOSITS**			S-1 S-2 S-3		
Very soft to soft, light gray to gray-brown, *Elastic Silt (MH)*; wet; trace fine sand; medium to high plasticity; micaceous.	-5.0 10.0		S-4		
Very loose, blue-gray, *Silty Sand (SM)*; wet; fine to medium sand; high plasticity fines; micaceous.	-3.0 12.0				
Medium dense, yellow-brown to red-brown, *Silty Sand with Gravel (SM)*; wet; fine to coarse, subangular to subrounded; gravel; fine to coarse sand; low plasticity fines; micaceous; slight iron oxide staining. **FLUVIAL AND ESTUARIAN DEPOSITS**	-1.3 16.3		S-5 S-6		
Dense, yellow-brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse sand; nonplastic fines; heavy iron oxide staining.	-10.0 25.0		S-7		

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND
- Sample Not Recovered
- Standard Penetration Test

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-4

January 2026 | 112880

SHANNON & WILSON | FIG. B6 Sheet 1 of 3

REV 3

DRAFT

MASTER LOG_E 112880.GPJ SHN2015.GRAYPDX.GLB SHANNON_PDX.GLB 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	~ 600,357 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,606 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft	PENETRATION RESISTANCE, N (blows/ft.)
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.						▲ Hammer Wt. & Drop: 140 lbs / 30 Inches
Continued: Dense to very dense, red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse sand; nonplastic fines; heavy iron oxide staining.			S-8 *			31
	-19.0 34.0				35	
Medium dense to very dense, gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; trace nonplastic fines; micaceous, shell fragments.			S-9			68
FLUVIAL AND ESTUARIAN DEPOSITS			S-10		40	24
	-27.5 42.5					
Medium dense, brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse sand; low plasticity fines; micaceous; slight iron oxide staining.			S-11		45	13
			S-12		50	14
	-37.5 52.5					
Medium dense, brown to yellow-brown and red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse, subangular to subrounded, gravel; fine to coarse sand; low plasticity fines; micaceous; slight iron oxide staining.			S-13		55	16

CONTINUED NEXT SHEET

LEGEND

* Sample Not Recovered

Standard Penetration Test

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-4

January 2026 | 112880

SHANNON & WILSON | FIG. B6 Sheet 2 of 3

REV 3

DRAFT

MASTER LOG_E 112880.GPJ SHN2015.GRAY.FOX.GLB SHANNON_ROLLOUT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	70.1 ft.	Northing:	~ 600,357 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 15 ft.	Easting:	~ 7,312,606 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft	PENETRATION RESISTANCE, N (blows/ft.)
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.						▲ Hammer Wt. & Drop: 140 lbs / 30 inches
Continued: Medium dense, red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse, subangular to subrounded, gravel; fine to coarse sand; low plasticity fines; micaceous; slight iron oxide staining.			S-14			25
FLUVIAL AND ESTUARIAN DEPOSITS	-49.0 64.0				65	
Harder drilling action at 64 feet.			S-15			50/3"
SILTSTONE/SANDSTONE: extremely weak to weak (R0-R2); moderately weathered; joint structure indiscernible in SPT sample.						
ALSEA FORMATION	-55.1 70.1		S-16		70	50/1st 1"
Completed: March 25, 2025					75	
					80	
					85	

0 20 40 60 80 100

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-4

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SHANNON & WILSON

FIG. B6
Sheet 3 of 3

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2013_BRARY.PDX.GLB SHANNON_PDX.GLB Log: SCF Rev: SCS Typ: SCF 3/31/26

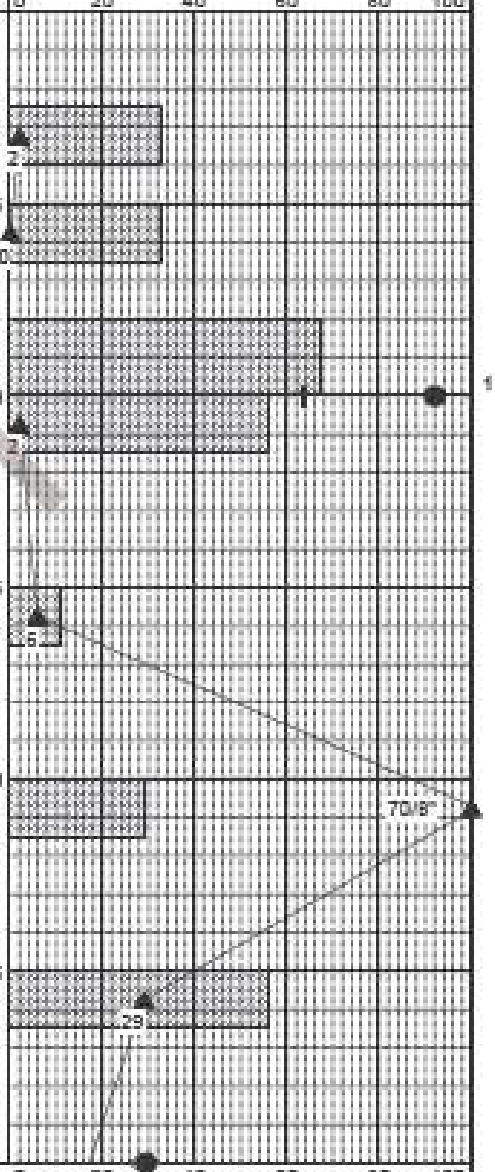
Total Depth:	74.1 ft.	Northing:	~ 698,630 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 32 ft.	Easting:	~ 7,311,955 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)
Asphalt; 10 inches thick.	31.5
Base Aggregate	0.9 30.4
Very soft to soft, dark brown to red-brown, *Elastic Silt with Sand (MH)*; moist; fine to medium sand; low plasticity; micaceous; heavy iron oxide staining. **ROADWAY FILL**	2.0
Very soft to soft, Dark brown, *Organic Soil (OH)*; moist; trace to few, fine to coarse sand; high plasticity; few to little organics. **MARSH DEPOSITS** *S-3: Pocket Pen = 0.7; Torvane = 0.8.*	23.9 8.5
Loose, gray, *Silty Gravel with Sand (GM)*; moist; fine to coarse, subangular to subrounded, gravel; fine to coarse sand; low to medium plasticity fines; micaceous. **ALLUVIAL FAN DEPOSITS**	19.9 12.5
Medium dense to very dense, brown to red-brown, *Silty Gravel with Sand, with Cobbles (GM)*; wet; fine to coarse, subangular to subrounded, gravel; fine to coarse sand; low to medium plasticity fines; disturbed texture; heavy iron oxide staining; basalt fragments.	14.4 18.0
Medium dense, gray to gray-brown, *Silty Sand (SM)*; wet; trace, fine, angular to subrounded gravel.	4.9 27.5

CONTINUED NEXT SHEET

Symbol | Samples: S-1, S-2, S-3, S-4, S-5, S-6, S-7 | Ground Water | Depth, ft

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND
- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-5

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SHANNON & WILSON | FIG. B7 Sheet 1 of 3

REV 3

DRAFT

MASTER LOG_E 112880.GPJ SHN2015.GPJ SHANNON_ROLLOUT 3/31/26 Log: SCF Rev: SCS Typ: SCF

Total Depth:	74.1 ft.	Northing:	~ 699,630 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 32 ft.	Easting:	~ 7,311,955 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. (ft.) / Depth

Continued:

Medium dense, gray to gray-brown, *Silty Sand (SM)*; wet; trace, fine, angular to subrounded gravel; fine to medium sand; low plasticity fines; micaceous; heavy iron oxide staining; weakly cemented.

-0.1 / 32.5

Soft to medium stiff, yellow-brown, *Lean Clay with Sand (CL)*; wet; fine to medium sand; medium plasticity; micaceous.

-5.1 / 37.5

Medium dense, red-brown, *Silty Gravel with Sand (GM)*; wet; fine to coarse, subangular to subrounded, gravel; fine to coarse sand; medium plasticity fines; micaceous.

ALLUVIAL FAN DEPOSITS

-10.1 / 42.5

Very dense, red-brown, *Silty Sand (SM)*; wet; fine to medium sand; nonplastic fines; moderate iron oxide staining.

FLUVIAL AND ESTUARIAN DEPOSITS

-15.1 / 47.5

Dense to very dense, gray and red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; low plasticity fines; micaceous; stratified with interbeds of *Poorly Graded Gravel with Sand (GP)*; moderate iron oxide staining.

Symbol / Samples / Ground Water / Depth, ft

S-8 S-9 S-10 S-11 S-12 S-13

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

0 20 40 60 80 100

15 8 21 63 91 41

35 40 45 50 55

0 20 40 60 80 100

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit ⊢—⊣ Liquid Limit

CONTINUED NEXT SHEET

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-5

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SHANNON & WILSON

FIG. B7
Sheet 2 of 3

REV 3

DRAFT

MASTER LOG_E 112880.GPJ SHN2015.GDT 3/31/26 Log: SCF Rev: SCS Typ: SCF

Total Depth:	74.1 ft.	Northing:	~ 699,630 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 32 ft.	Easting:	~ 7,311,955 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft	PENETRATION RESISTANCE, N (blows/ft.)
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.						▲ Hammer Wt. & Drop: 140 lbs / 30 Inches
Continued: Dense to very dense, gray and red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium; low plasticity fines; micaceous; stratified with interbeds of *Poorly Graded Gravel with Sand (GP)*; moderate iron oxide staining. **FLUVIAL AND ESTUARIAN DEPOSITS** *Harder drilling action at 67 feet.*	-34.6 67.0		S-14 S-15		65	32 40
SILTSTONE/SANDSTONE: very weak to weak (R1-R2); slightly to moderately weathered; joint structure indiscernible in SPT sample. **ALSEA FORMATION**			S-16		70	50/1st 3"
Completed: March 28, 2025	-41.7 74.1		S-17		75	50/1st 1"
					80	
					85	

0 20 40 60 80 100

LEGEND
- * Sample Not Recovered
- Standard Penetration Test
- 3" O.D. Shelby Tube
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-5

January 2026 112880

SHANNON & WILSON

FIG. B7
Sheet 3 of 3

REV 3

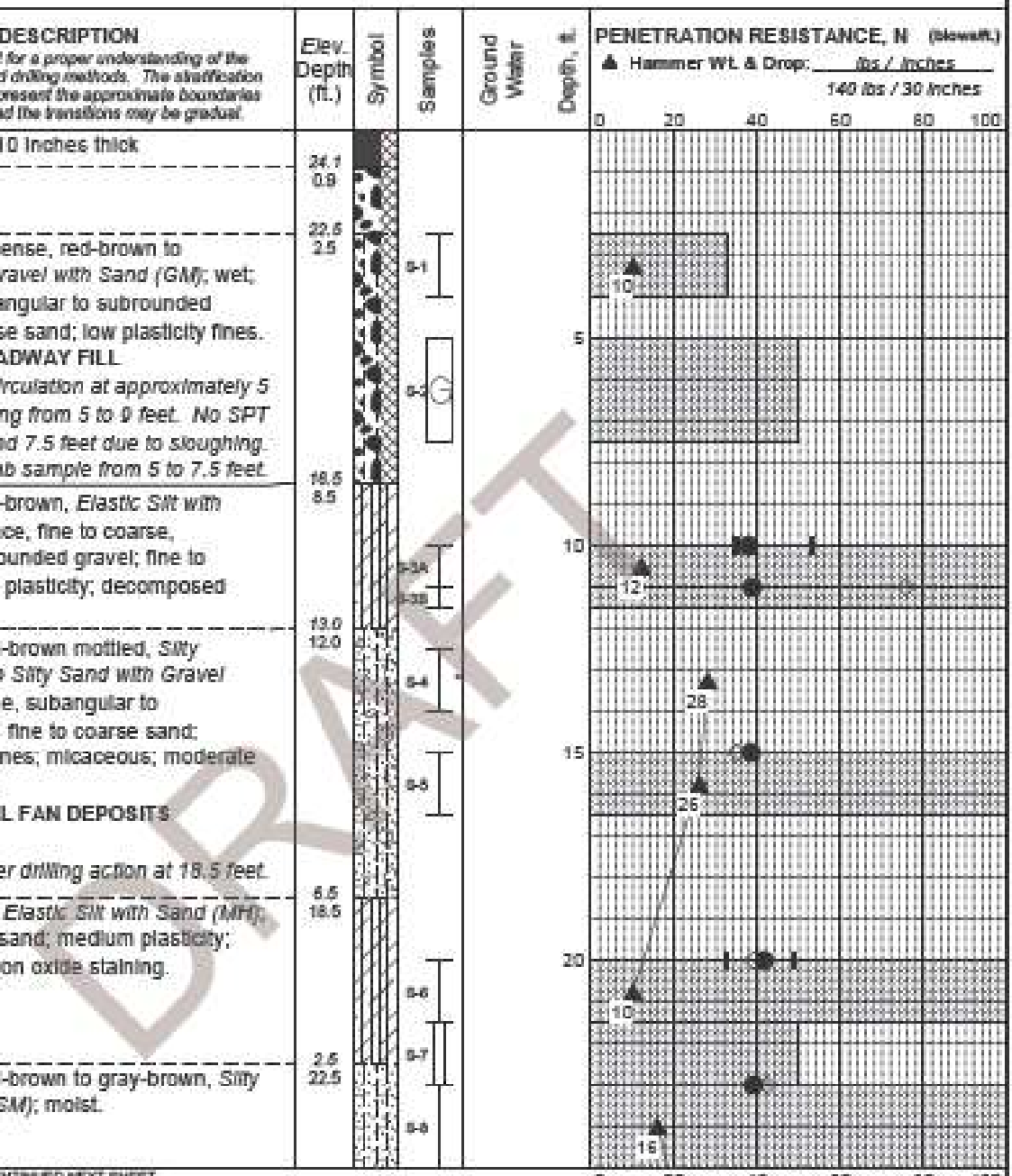
DRAFT

MASTER_LOG_E 112880.GPJ SHN2015.GRAYPDX.GLB SHANNA_PDX.GLB IDOT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	90.1 ft.	Northing:	~ 699,213 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 25 ft.	Easting:	~ 7,311,470 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)
Asphalt Concrete; 10 inches thick	24.1
Base Aggregate	0.9 22.6
Loose to medium dense, red-brown to gray-brown, *Silty Gravel with Sand (GM)*; wet; fine to coarse, subangular to subrounded gravel; fine to coarse sand; low plasticity fines. **ROADWAY FILL** *Lost drilling fluid circulation at approximately 5 feet. Hole sloughing from 5 to 9 feet. No SPT samples at 5.0 and 7.5 feet due to sloughing. Cuttings grab sample from 5 to 7.5 feet.*	2.5 16.5
Stiff, gray to yellow-brown, *Elastic Silt with Sand (MH)*; wet; trace, fine to coarse, subangular to subrounded gravel; fine to medium sand; high plasticity; decomposed gravel clasts.	8.5 13.0
Medium dense, red-brown mottled, *Silty Gravel with Sand to Silty Sand with Gravel (GM/SM)*; moist; fine, subangular to subrounded gravel; fine to coarse sand; medium plasticity fines; micaceous; moderate iron oxide staining. **ALLUVIAL FAN DEPOSITS** *Softer drilling action at 18.5 feet.*	12.0 6.5 18.5
Stiff, yellow-brown, *Elastic Silt with Sand (MH)*; wet; fine to coarse sand; medium plasticity; micaceous; slight iron oxide staining.	2.5
Medium dense, red-brown to gray-brown, *Silty Sand with Gravel (SM)*; moist.	22.5

CONTINUED NEXT SHEET



LEGEND

- Sample Not Recovered
- Standard Penetration Test
- Grab Sample
- 3" O.D. Shelby Tube

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-6

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SHANNON & WILSON | FIG. B8 Sheet 1 of 4

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2015.GRAYPDX.GLB SHANNON_PDX.GLB 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	90.1 ft.	Northing:	~ 699,213 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 25 ft.	Easting:	~ 7,311,470 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.)

Symbol

Samples

Ground Water

Depth, ft

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: lbs / inches

140 lbs / 30 inches

0 20 40 60 80 100

Continued:

Medium dense, red-brown to gray-brown, *Silty Sand with Gravel (SM)*; moist; interbedded with gray-brown sandy SILT (ML); moist; medium plasticity fines; sand fine to medium; fine, subangular to subrounded gravel; fine to coarse; low to medium plasticity; micaceous; disturbed texture; heavy iron oxide staining.

ALLUVIAL FAN DEPOSITS

-2.5
27.5

Dense to very dense, gray-brown to red-brown, *Silty Sand (SM)*; wet; fine to medium sand; medium plasticity fines; micaceous.

-7.5
32.5

Medium dense to very dense, gray-brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; moist to wet; fine to coarse sand; nonplastic fines; micaceous; slight to moderate iron oxide staining.

FLUVIAL AND ESTUARIAN DEPOSITS

S-9

S-10

S-11

S-12

30 35 40 45

33 50 59 55

0 20 40 60 80 100

CONTINUED NEXT SHEET

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- Grab Sample
- 3" O.D. Shelby Tube

Recovery (%)

% Fines (<0.075mm)

% Water Content

Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-6

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SHANNON & WILSON

FIG. B8
Sheet 2 of 4

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2015.GRAYPDX.GLB SHANNA_PDX.GLB IPDX.GDT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	90.1 ft.	Northing:	– 699,213 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	– 25 ft.	Easting:	– 7,311,470 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	–	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	–	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION	Elev. Depth (ft.)	Symbol	Samples	Ground Water Depth, ft.	PENETRATION RESISTANCE, N (blows/ft.) ▲ Hammer Wt. & Drop: lbs / inches
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.					140 lbs / 30 inches
Continued: Medium dense to very dense, gray-brown to red-brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to coarse; nonplastic fines; micaceous; moderate iron oxide staining.			S-13		21
FLUVIAL AND ESTUARIAN DEPOSITS			S-14	55	22
Harder drilling action at 60 feet.					
SILTSTONE: very weak to weak (R1-R2); slightly to moderately weathered; joint structure indiscernible in SPT sample.	-35.0 60.0		S-15	60	50/4"
ALSEA FORMATION			S-16	65	50/1st 2"
			S-17	70	50/1st 1"

0 20 40 60 80 100

CONTINUED NEXT SHEET

LEGEND

- Sample Not Recovered
- Standard Penetration Test
- Grab Sample
- 3" O.D. Shelby Tube

Recovery (%)
% Fines (<0.075mm)
% Water Content
Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-6

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SHANNON & WILSON

FIG. B8
Sheet 3 of 4

REV 3

DRAFT

MASTER_LOG_E 112880.GPJ SHN2015_BRARY.PDX.GLB SHANNON_PDX.GDT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	90.1 ft.	Northing:	~ 699,213 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 25 ft.	Easting:	~ 7,311,470 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft	PENETRATION RESISTANCE, N (blows/ft.)
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.						▲ Hammer Wt. & Drop: lbs / inches 140 lbs / 30 inches
Continued: SILTSTONE: very weak to weak (R1-R2); slightly to moderately weathered; joint structure indiscernible in SPT sample.			S-18		80	50/1st 2"
ALSEA FORMATION			S-19			
			S-20		85	50/1st 2"
Completed: March 27, 2025	-65.1 90.1		S-21		90	50/1st 1"
					95	

0 20 40 60 80 100

0 20 40 60 80 100

☒ Recovery (%)
○ % Fines (<0.075mm)
● % Water Content
Plastic Limit ⊢—⊣ Liquid Limit

LEGEND
- Sample Not Recovered
- Standard Penetration Test
- Grab Sample
- 3" O.D. Shelby Tube

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-6

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SHANNON & WILSON

FIG. B8
Sheet 4 of 4

REV 3

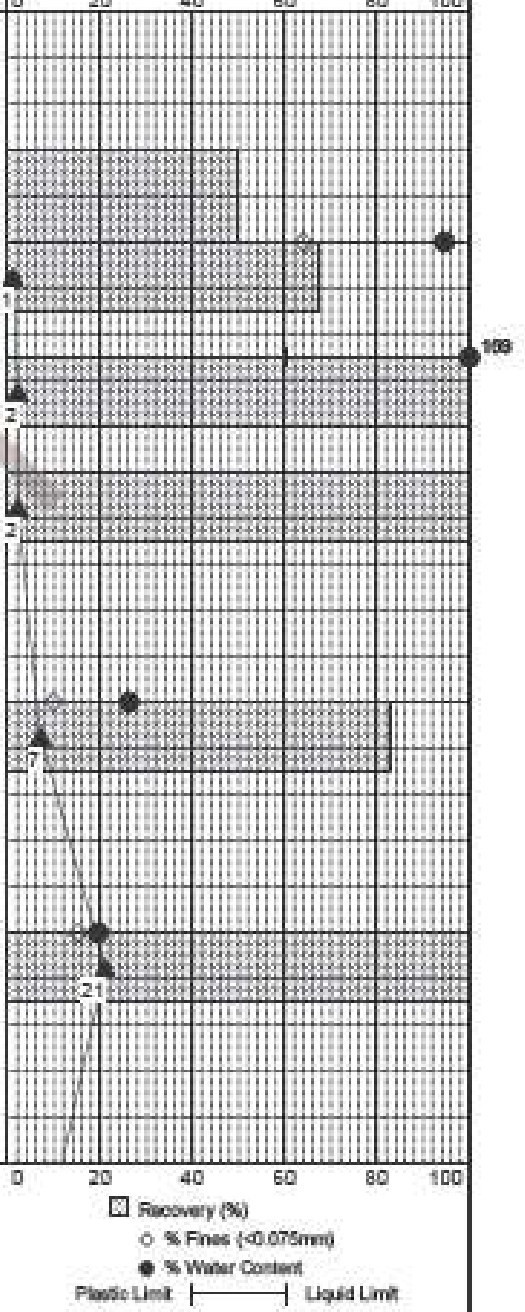
DRAFT

MASTER_LOG_E 112880.GPJ SW2013L.GRAYPDX.GLB SHANNA_PDX.GLOBT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	41.5 ft.	Northing:	~ 698,697 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 11 ft.	Easting:	~ 7,311,104 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-65LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NF	Offset:	~	Other Comments:	Hammer Efficiency = 65%		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)	Symbol	Samples	Ground Water Depth, ft.	PENETRATION RESISTANCE, N (blows/ft.) ▲ Hammer Wt. & Drop: 140 lbs / 30 inches
Asphalt Concrete; 9 inches thick	9.8				
Base Aggregate	0.8				
Dark brown to olive brown, *Elastic Silt with Gravel (MH)*; wet; fine, subangular gravel; trace to few, fine to medium sand; medium to high plasticity; trace to few organics.	8.6 2.0		S-1		
S-1: Pocket Pen = 0.4; Torvane = 0.5	6.1 4.5				
Very soft, dark brown, *Gravelly Organic Soil with Sand (OH)*; wet; fine, subangular to subrounded gravel; fine to medium sand; high plasticity; some organics; stratified with interbeds of peat.	3.6 7.0		S-2 S-3		
MARSH DEPOSITS					
Soft, dark brown, *Organic Soil (OH)*; wet; trace, fine to medium sand; high plasticity; some organics; stratified with interbeds of peat.	-0.6 11.2		S-4		
Loose, gray, *Silty Sand (SM)*; wet; fine to medium sand; low plasticity fines; micaceous.					
FLUVIAL AND ESTUARIAN DEPOSITS	-3.5 14.0				
Loose, brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; fine to medium sand; nonplastic fines.			S-5		
Medium dense, yellow-brown to brown, *Silty Sand (SM)*; wet; trace to few, fine to coarse, subrounded gravel; fine to coarse sand; nonplastic fines; micaceous; slight iron oxide staining.	-7.0 17.5		S-6		
Medium dense, yellow-brown to brown, *Poorly Graded Sand with Silt (SP-SM)*; wet.	-12.0 22.5				

CONTINUED NEXT SHEET



LEGEND

- 3" O.D. Shelby Tube
- Standard Penetration Test

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-7

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FIG. B9
Sheet 1 of 2

REV 3

DRAFT

MASTER LOG_E 112880.GPJ SHN2015.GRAYPDX.GLB SHANNON_PDX.GLB EDIT 3/31/26
Log: SCF Rev: SCS Typ: SCF

Total Depth:	41.5 ft.	Northing:	~ 698,697 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~ 11 ft.	Easting:	~ 7,311,104 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-55LC	Hammer Type:	Automatic
Horiz. Datum:	OR83-NWF	Offset:	~	Other Comments:	Hammer Efficiency = 85%		

SOIL DESCRIPTION	Elev. Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft	PENETRATION RESISTANCE, N (blows/ft.)
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.						▲ Hammer Wt. & Drop: 140 lbs / 30 Inches
Continued: Medium dense, yellow-brown to brown, *Poorly Graded Sand with Silt (SP-SM)*; wet; trace to few, fine to coarse, subrounded gravel; fine to coarse sand; nonplastic fines; micaceous; slight iron oxide staining; stratified with interbeds of *Poorly Graded Gravel with Silt and Sand (GP-GM)*.			S-7			10
			S-8		30	15
FLUVIAL AND ESTUARIAN DEPOSITS			S-9		35	14
			S-10		40	18
Completed: March 25, 2025	-31.0 41.5				45	

0 20 40 60 80 100

LEGEND

- 3" O.D. Shelby Tube
- Standard Penetration Test
- Recovery (%)
- % Fines (<0.075mm)
- % Water Content
- Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

LOG OF BORING B-7

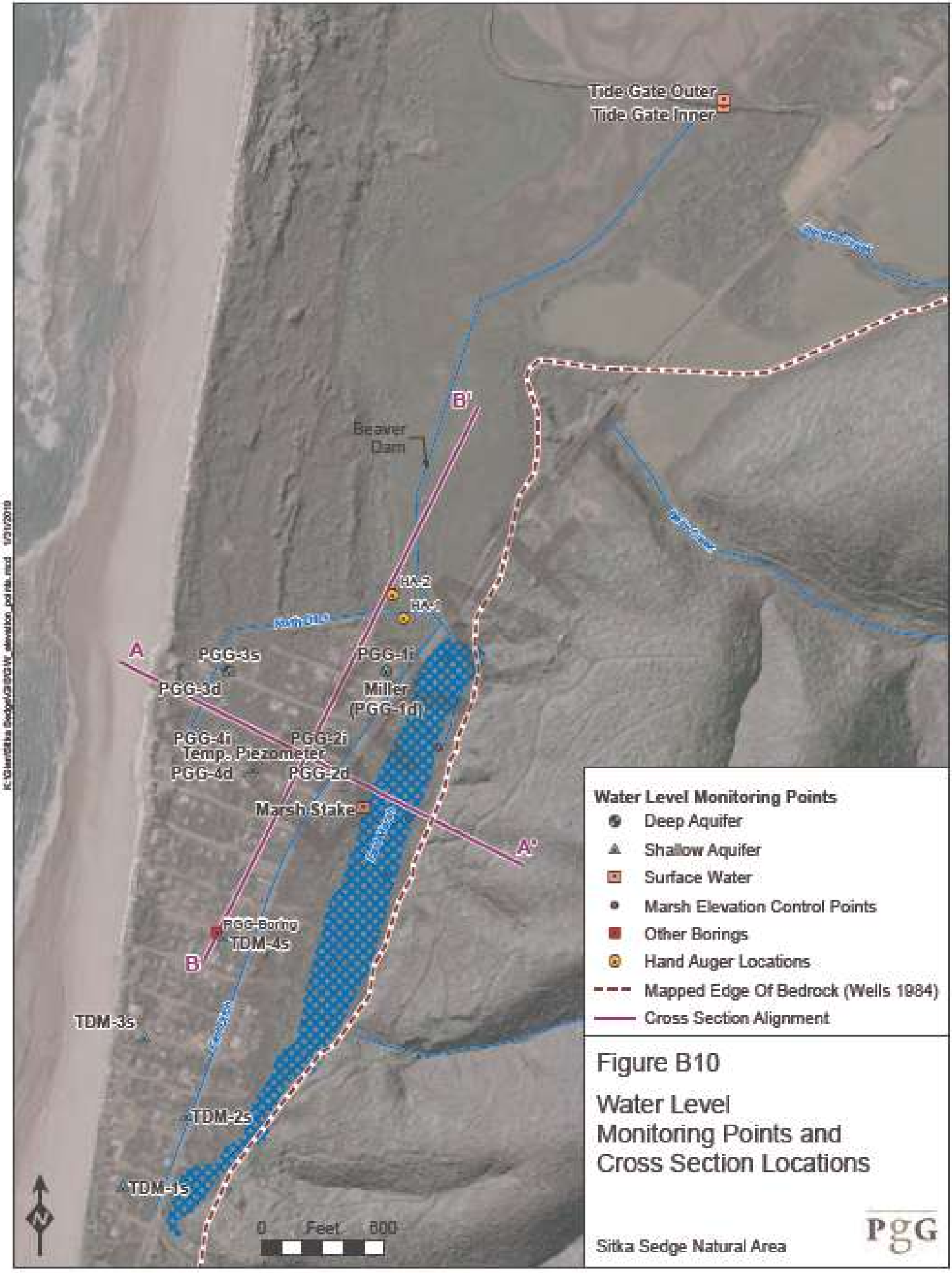
January 2026 | 112880

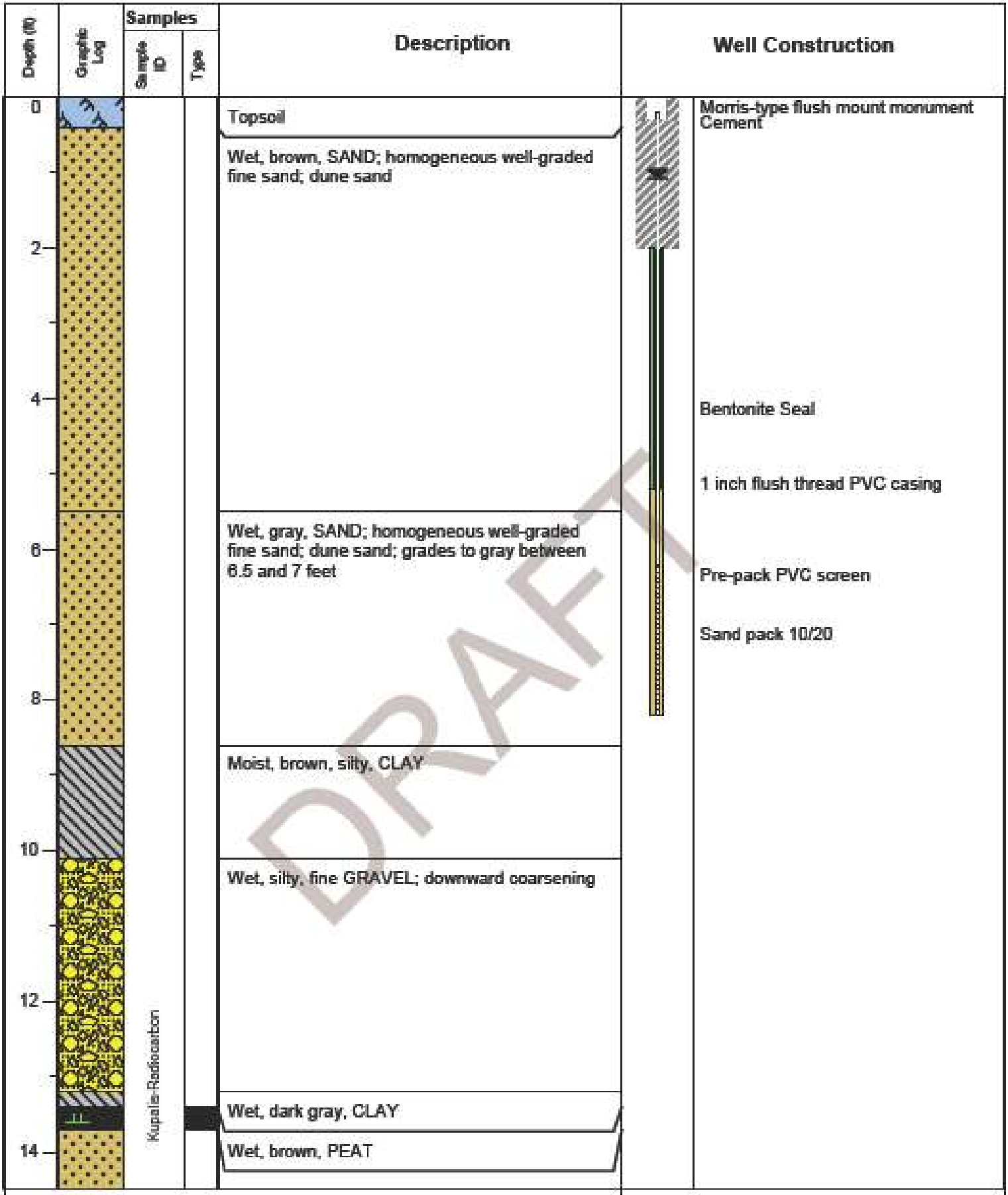
SHANNON & WILSON | FIG. B9 Sheet 2 of 2

REV 3

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MASTER_LOG_E 112880.GPJ SHN2015.GRAYPDX.GLB SHANNA_PDX.GLOBT 3/31/26
Log: SCF Rev: SCS Typ: SCF


Tide Gate Outer
Tide Gate Inner
B'
Beaver Dam
HA-2
HA-1
A
PGG-3s
PGG-1i
PGG-3d
Miller (PGG-1d)
PGG-4i
Temp. Piezometer
PGG-2i
PGG-4d
PGG-2d
Marsh Stake
A'
PGG-Boring
TDM-4s
B
TDM-3s
TDM-2s
TDM-1s
0
Feet
800
N
Water Level Monitoring Points
Deep Aquifer
Shallow Aquifer
Surface Water
Marsh Elevation Control Points
Other Borings
Hand Auger Locations
Mapped Edge Of Bedrock (Wells 1984)
Cross Section Alignment
Figure B10
Water Level Monitoring Points and Cross Section Locations
Sitka Sedge Natural Area
PgG



Address: Roma Ave and Sand Lake Rd
City, State: Tierra Del Mar, OR
Horizontal Loc.: N 200000.0 ft, E 1000000 ft
Consulting Firm: Pacific Groundwater Group
Logged by: Glen Wallace
Date: 2/14/18

Drilling Firm: Holocene Drilling
Drilling Method: Direct Push
Well ID: L106639
DTW: 1.06 ft bmp
MP Elevation: 00.00 ft
Datum: NAVD 88

Figure B11
Boring Log and As-Built
PGG-1i

OPRD Sitka Sedge
Hydrogeologic Investigation
JG1704

PgG

Depth (ft)	Graphic Log	Samples: Sample ID	Samples: Type	Description	Well Construction
				Wet, gray, fine-SAND; trace roots; well graded	

DRAFT

Address: Roma Ave and Sand Lake Rd
City, State: Tierra Del Mar, OR
Horizontal Loc.: N 200000.0 ft, E 1000000 ft
Consulting Firm: Pacific Groundwater Group
Logged by: Glen Wallace
Date: 2/14/18

Drilling Firm: Holocene Drilling
Drilling Method: Direct Push
Well ID: L106639
DTW: 1.06 ft bmp
MP Elevation: 00.00 ft
Datum: NAVD 88

Figure B11
Boring Log and As-Built
PGG-1i

OPRD Sitka Sedge
Hydrogeologic Investigation
JG1704

PgG

APPENDIX C: LABORATORY TEST RESULTS

Appendix C

Laboratory Test Results

CONTENTS

C.1 Introduction 1
C.2 Soil Testing 1
C.2.1 Moisture (Natural Water) Content 1
C.2.2 Atterberg Limits 2
C.2.3 Particle-Size Analysis 2
C.2.4 Consolidation Testing 2
C.2.5 Corrosivity Testing 3

Figures

Figure C1: Atterberg Limits Results (2025)
Figure C2: Grain Size Distribution (2025)
Figure C3: Grain Size Distribution (2026)
Figure C4: One-Dimensional Consolidation, Soil Sample at 9 Feet in Boring B-5 (2026)
Figure C5: One-Dimensional Consolidation, Soil Sample at 3 Feet in Boring B-7 (2026)

Attachments

Cooper Testing Laboratory, Inc., Corrosivity Tests Summary, dated May 2, 2025

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C.1 INTRODUCTION

Soil samples obtained during the field explorations were described and identified in the field in general accordance with the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), ASTM D2488. The samples were reviewed in the Shannon & Wilson laboratory. The physical characteristics of the samples were noted, and the field descriptions and identifications were modified where necessary in accordance with the terminology presented in Appendix B, Figures B1 and B2. Select samples were chosen for various laboratory tests. We refined our visual-manual soil descriptions and identifications based on the results of the laboratory tests, using elements of the Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), ASTM D2487. The refined descriptions and identifications were then incorporated into the Logs of Borings, presented in Appendix B. Note that ASTM D2487 was not followed in full because it requires that a suite of tests be performed to fully classify a single sample.

The initial soil testing program for bridge and culvert design included natural moisture content, Atterberg Limits, particle size analyses, and corrosivity testing. In addition to the laboratory testing performed for the bridges and culvert, laboratory testing was performed on select samples remaining in storage for the dike setback design, including particle size analyses and consolidation testing. Laboratory testing was performed by Shannon & Wilson in our in-house laboratories in Lake Oswego, Oregon, and Seattle, Washington, and Cooper Testing Laboratory of Palo Alto, California. All test procedures were performed in accordance with applicable ASTM International standards.

Analytical testing was performed on a composite sample of investigation-derived waste from the upper 10 feet of the borings for disposal. Analytical testing was performed by Pace Analytical National (Pace) of Mount Juliet, Tennessee, in accordance with applicable Environmental Protection Agency (EPA) standards. The results of this testing are not included in the report.

C.2 SOIL TESTING

C.2.1 Moisture (Natural Water) Content

Natural moisture content determinations were performed in accordance with ASTM D2216 on selected soil samples. The natural moisture content is a measure of the amount of moisture in the soil at the time of exploration. It is defined as the ratio of the weight of

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water to the dry weight of the soil, expressed as a percentage. The results of moisture content determinations are presented on the Logs of Borings in Appendix B.

C.2.2 Atterberg Limits

Atterberg Limits were determined for select samples in accordance with ASTM D4318. This analysis yields index parameters of the soil that are useful in soil identification, as well as in a number of analyses, including liquefaction analysis. An Atterberg Limits test determines a soil's liquid limit (LL) and plastic limit (PL). These are the maximum and minimum moisture contents at which the soil exhibits plastic behavior. A soil's plasticity index (PI) can be determined by subtracting PL from LL. The LL, PL, and PI of the tested samples are presented in Figure C1, Atterberg Limits Results. The results of the Atterberg Limits tests are also presented in the Logs of Borings in Appendix B. For the purposes of soil description, Shannon & Wilson uses the term nonplastic to refer to soils with a PI less than 4, low plasticity for soils with a PI range of 4 to 10, medium plasticity for soils with a PI range of 10 to 20, and high plasticity for soils with a PI greater than 20. Oven-dried liquid limits were performed on samples B-3 S-2 at 5.0 feet, and B-7 S-3 at 7.5 feet.

C.2.3 Particle-Size Analysis

Particle-size analyses were conducted on select samples to determine their grain-size distributions. Sieve analyses were conducted in accordance with ASTM D6913 and sieve with hydrometer analyses were conducted in accordance with ASTM D422. A wet sieve analysis was performed to determine the percentage (by weight) of each sample passing the No. 200 (0.075 mm) sieve. The material retained on the No. 200 sieve was shaken through a series of sieves to determine the distribution of the plus No. 200 fraction (ASTM D6913). Several particle-size analyses included hydrometer analysis by ASTM D422, which yields the grain-size distribution of the sample fraction finer than the No. 200 sieve. For some analyses, only the percent passing the No. 200 sieve was determined (ASTM D1140). Results of the particle-size analyses performed for bridges and culvert design are presented in Figure C2. Results of the additional particle-size analyses that were performed for dike setback design are presented in Figure C3. The percentage of each sample passing the No. 200 sieve is presented on the Logs of Borings in Appendix B.

C.2.4 Consolidation Testing

One-dimensional consolidation tests were performed for dike setback design on selected samples in accordance with ASTM D2435. Consolidation tests measure the volume change and rate of volume change of soil samples under predetermined loads. Each specimen was trimmed and fit into the testing apparatus. Vertical (normal) loads were applied while the sample was inundated with water. The specimen was then permitted to consolidate under

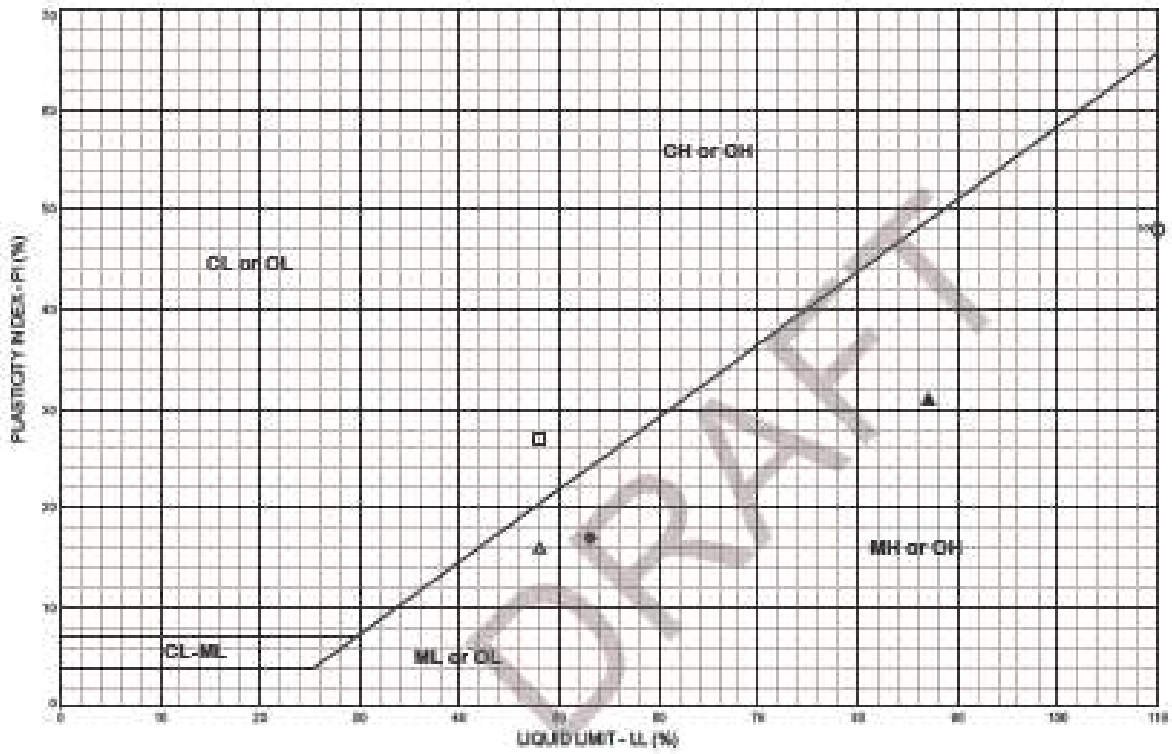
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the applied normal loads, and the amounts of consolidation were measured at specified time intervals. We note that the consolidation test results should be used with caution, as the tests were performed on samples that were in storage for a number of months. The one-dimensional consolidation test results are presented in Figures C4 and C5.

C.2.5 Corrosivity Testing

Corrosivity testing was performed on select specimens to evaluate the corrosivity potential of the soil at the site. The corrosivity test suite included redox potential, soil conductivity (resistivity), soluble chloride and sulfate concentration, sulfide tests, and soil pH.
A discussion of the corrosivity results is included in the main report text in Section 6.4. Analytical testing was performed by Cooper Testing Laboratory, and the results of the corrosivity testing are attached to the end of this appendix.

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NOTES

1) Atterberg limits tests were performed in general accordance with ASTM D4318 unless otherwise noted in the report.

2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

3) Plasticity adjectives used in sample descriptions correspond to plasticity index as follows:
- Nonplastic (NP) (< 4%)
- Low Plasticity (4 to 10%)
- Medium Plasticity (10 to 20%)
- High Plasticity (> 20%)

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL*	GROUP NAME*	LL %	PL %	PI %	NAT. W.C. %	PASS #200 %
B-1, S-5	13.0	SP-SM	*Poorly Graded Sand with Silt*	NP	NP	NP	20	8
B-2, S-3	7.5	SM	*Silty Sand*	NP	NP	NP	37	18
▲ B-3, S-2	5.0	OH	*Organic Soil (Oven Dried LL = 47)*	87	56	31	83	
◆ B-4, S-4	10.0	MH	*Elastic Silt*	53	36	17	50	
○ B-5, S-4	10.0	OH	*Organic Soil with Gravel*	112	64	48	82	
□ B-5, S-9	35.0	CL	*Lean Clay with Sand*	48	21	27		
△ B-5, S-10	40.0	GM	*Silty Gravel with Sand*	48	32	16	36	38

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Tillamook County, Oregon

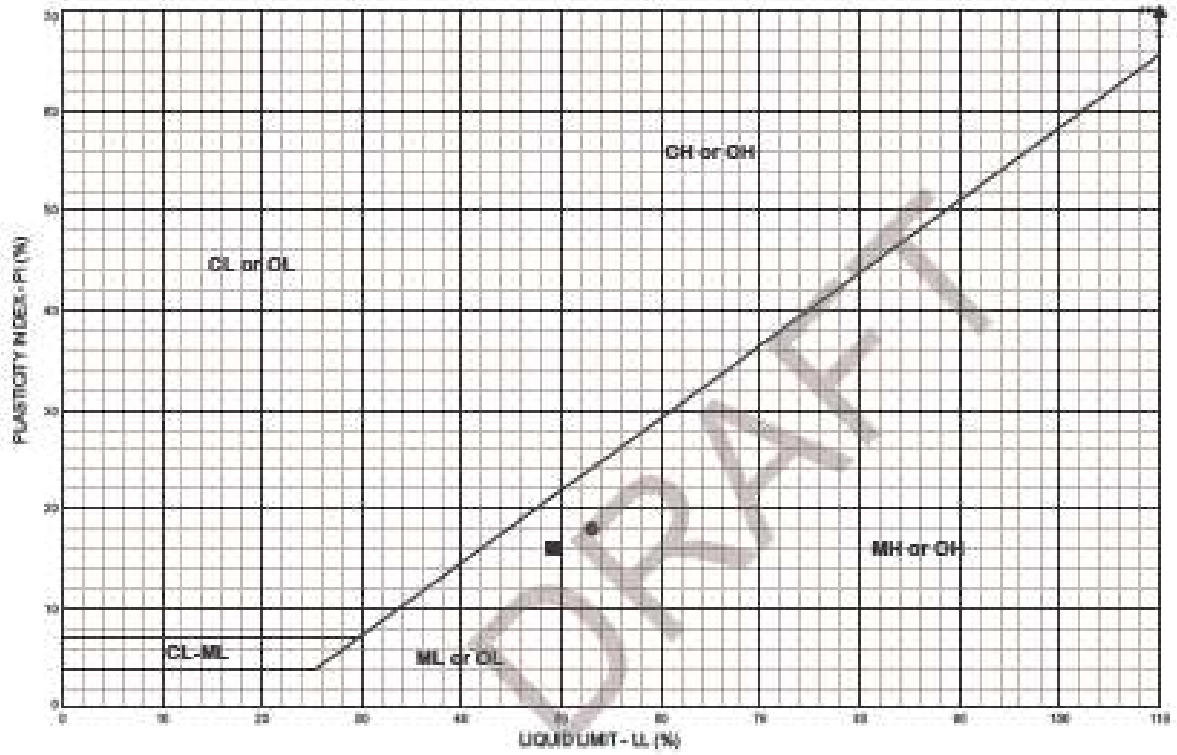
ATTERBERG LIMITS RESULTS

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FIG. C1
Sheet 1 of 2

FIG. C1



NOTES

1) Atterberg limits tests were performed in general accordance with ASTM D4318 unless otherwise noted in the report.

2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

3) Plasticity adjectives used in sample descriptions correspond to plasticity index as follows:
- Nonplastic (NP) (< 4%)
- Low Plasticity (4 to 10%)
- Medium Plasticity (10 to 30%)
- High Plasticity (> 30%)

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL*	GROUP NAME*	LL %	PL %	PI %	NAT. W.C. %	FINES %
● B-6, S-2	10.0	MH	*Elastic Silt with Sand*	53	35	18	30	
■ B-6, S-5	30.0	MH	*Elastic Silt with Sand*	49	33	16	42	36
▲ B-7, S-3	7.5	OH	*Organic Soil (Oven Dried LL = 58)*	153	83	80	109	69

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

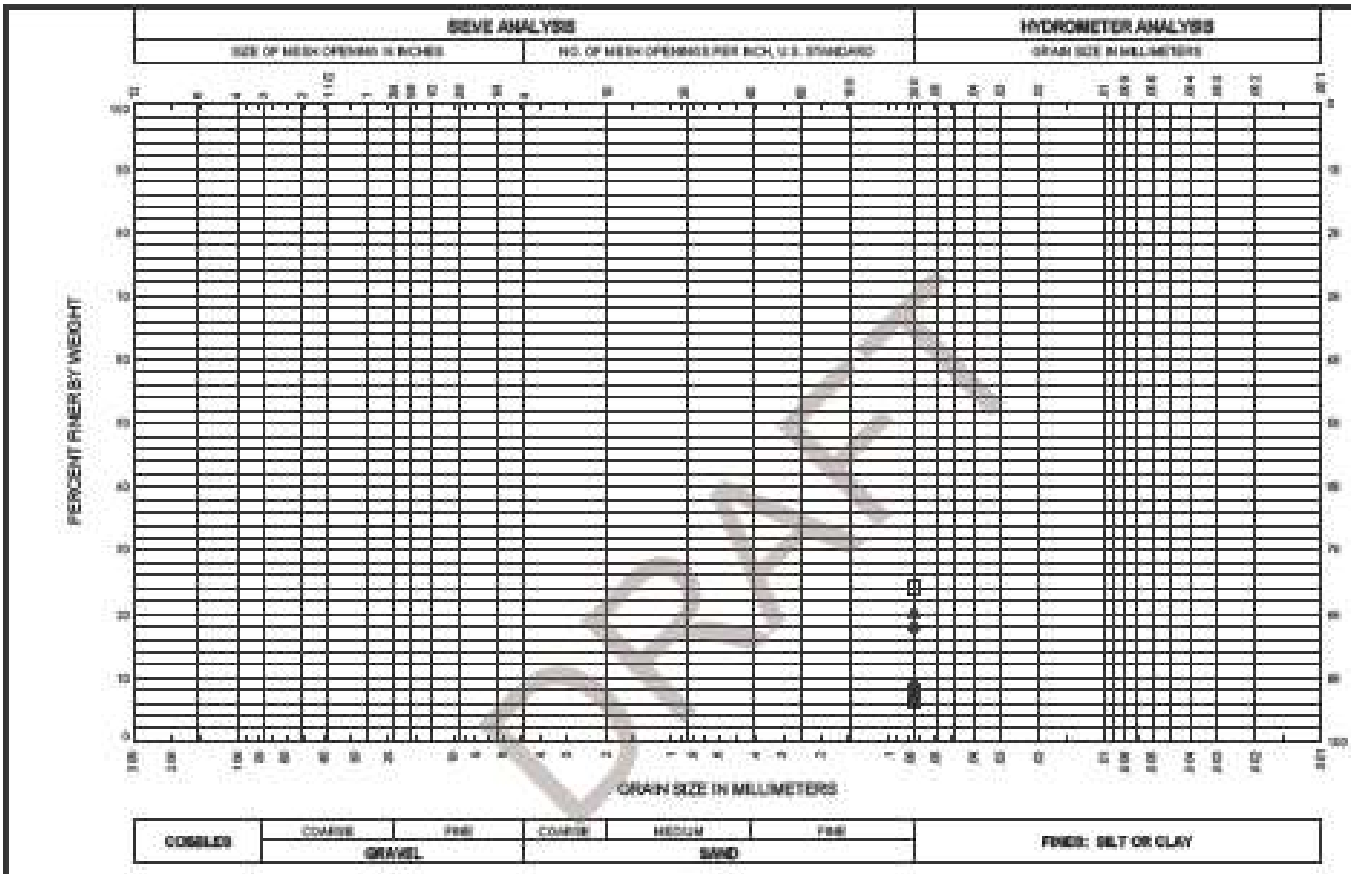
ATTERBERG LIMITS RESULTS

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FIG. C1
Sheet 2 of 2

FIG. C1



NOTES:
1) Sieve analysis were performed in general accordance with ASTM D6913. sieve with hydrometer analysis were performed in general accordance with ASTM D422, and amount that pass #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Name and Group Symbol are in accordance with ASTM D2487 where appropriate laboratory tests are performed.

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[1]	GROUP NAME[2]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-1, S-5	13.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	6	29	
■ B-1, S-6	15.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	6	27	
▲ B-1, S-9	25.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	6	25	
◆ B-2, S-3	7.5	SM	*Silty Sand*	-	-	19	37	
○ B-2, S-7	25.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	6	23	
□ B-2, S-10	40.0	SM	*Silty Sand*	-	-	24	35	
△ B-2, S-12	53.0	SM	*Silty Sand*	-	-	30	28	

Silka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

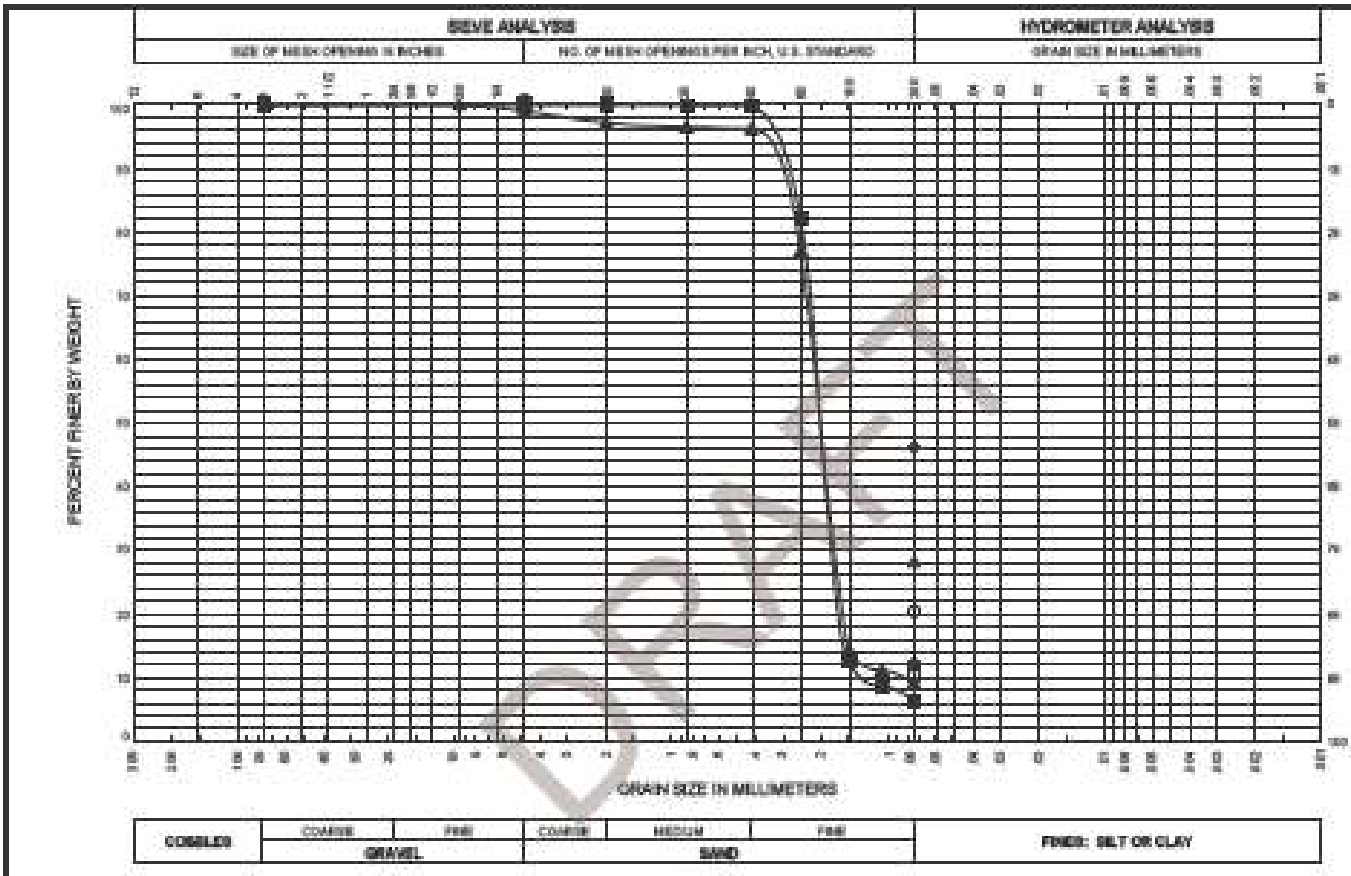
GRAIN SIZE DISTRIBUTION

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FIG. C2
Sheet 1 of 3

FIG. C2



NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913. sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount that pass #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Name and Group Symbol are in accordance with ASTM D2487 where appropriate laboratory tests are performed.

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[1]	GROUP NAME[1]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-2, S-14	63.0	SM	*Silty Sand*	-	-	12	34	
■ B-3, S-9	15.0	SP-SM	*Poorly Graded Sand with Silt*	6	94	6	27	
▲ B-3, S-7	25.0	SP-SM	*Poorly Graded Sand with Silt*	1	90	9	27	
◆ B-4, S-5	15.0	SM	*Silty Sand*	-	-	45	39	
○ B-4, S-8	33.0	SM	*Silty Sand with Gravel*	-	-	33	30	
□ B-4, S-9	35.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	10	29	
△ B-5, S-8	33.0	SM	*Silty Sand*	-	-	28	30	

Silka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

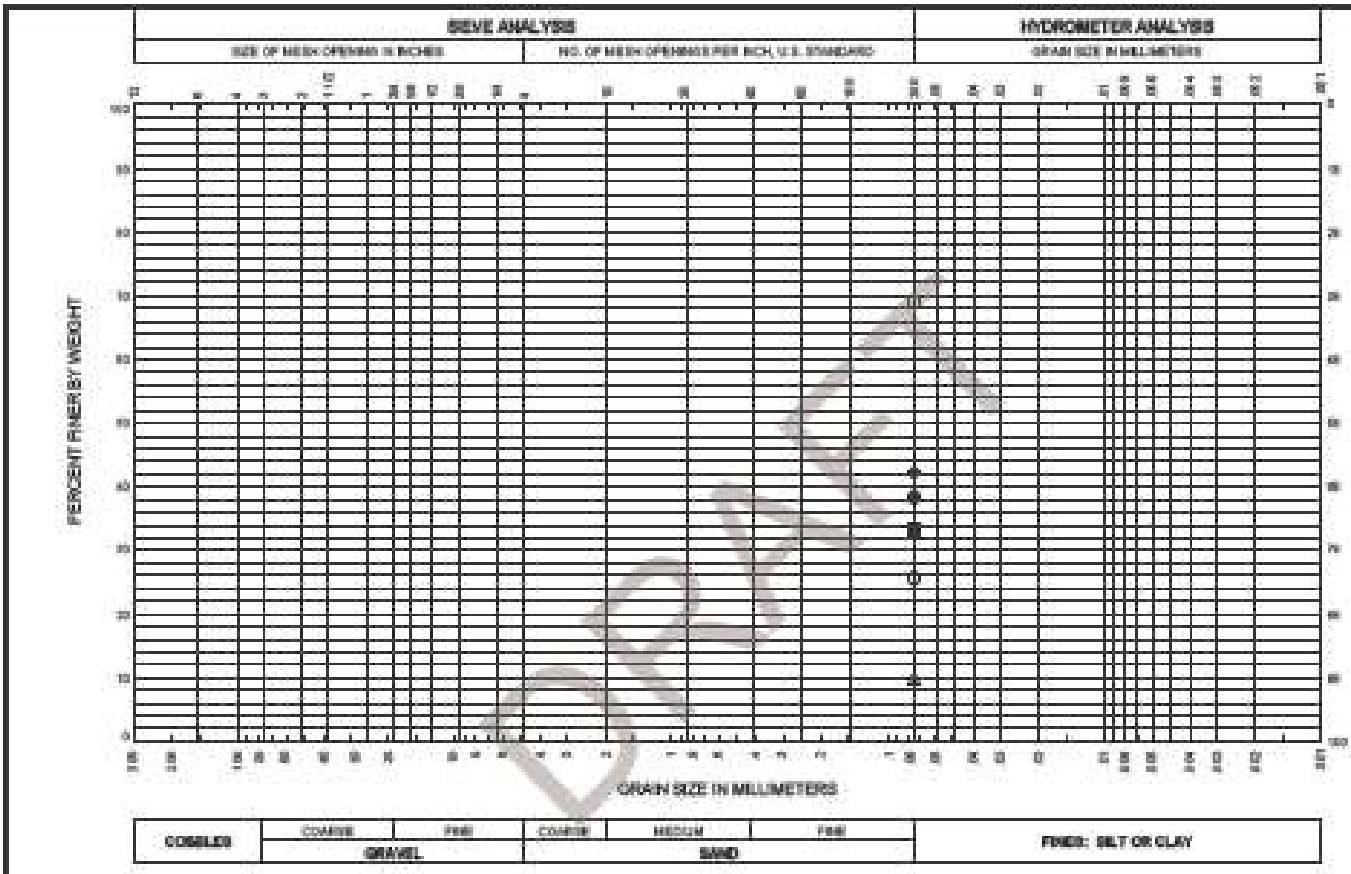
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FIG. C2
Sheet 2 of 3

FIG. C2



NOTES:
1) Sieve analysis were performed in general accordance with ASTM D6913, sieve with hydrometer analysis were performed in general accordance with ASTM D422, and amount finer than #200 sieve analysis were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Name and Group Symbol are in accordance with ASTM D2487 where appropriate laboratory tests are performed.

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[1]	GROUP NAME[1]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-6, S-10	43.0	GM	*Silty Gravel with Sand*	-	-	39	36	
■ B-6, S-4	15.0	GM	*Silty Gravel with Sand*	-	-	33	38	
▲ B-6, S-5	23.0	MH	*Elastic Silt with Sand*	-	-	59	42	
◆ B-6, S-7	33.0	SM	*Silty Sand*	-	-	42	39	
○ B-6, S-8	38.0	SM	*Silty Sand*	-	-	38	33	
□ B-7, S-3	7.5	OH	*Organic Silt (Oven Dried LL = 58)*	-	-	69	109	
△ B-7, S-5	15.0	SP-SM	*Poorly Graded Sand with Silt*	-	-	10	26	

Silva Sedge Tidal Wetland Restoration
Tillamook County, Oregon

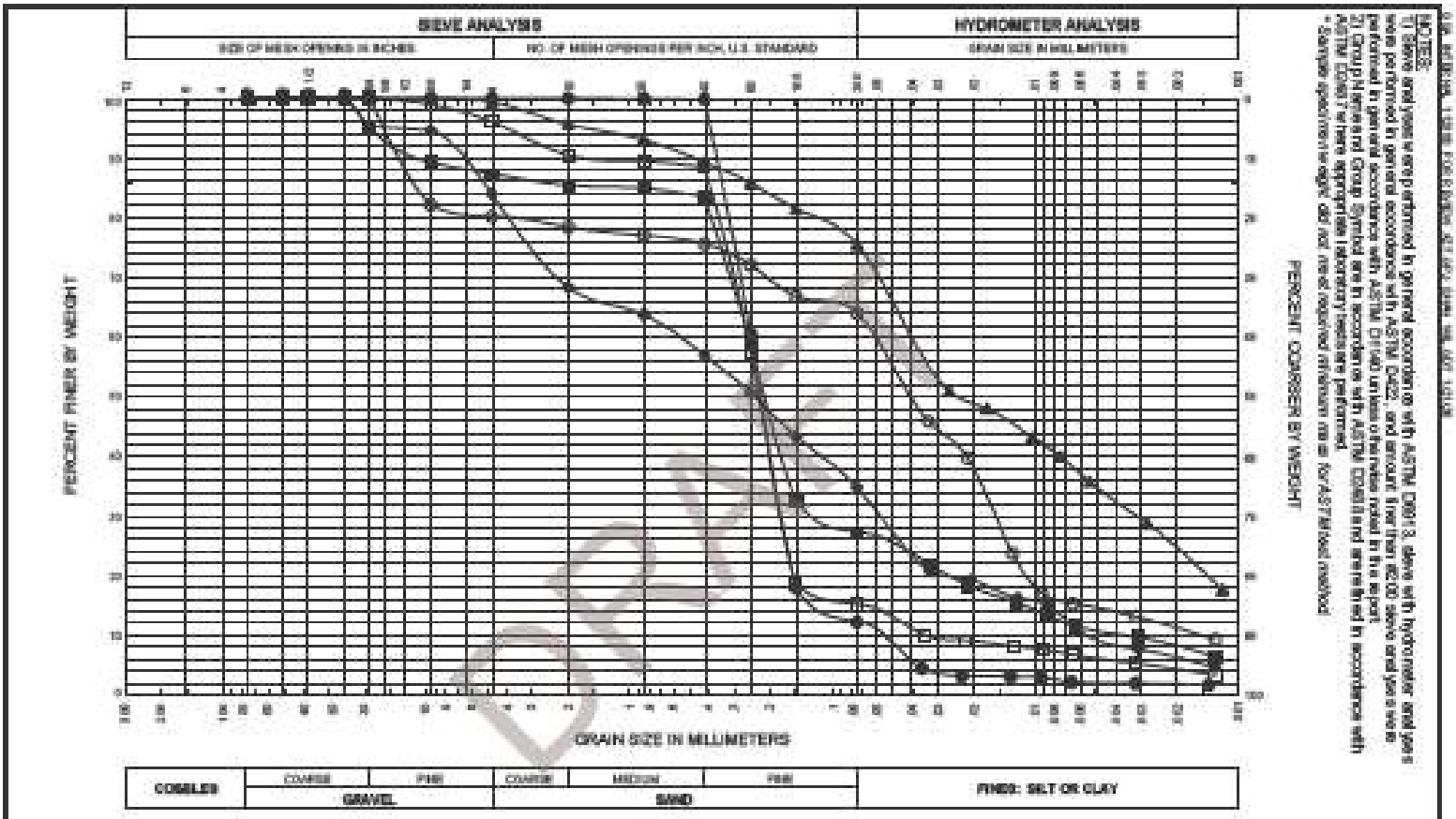
GRAIN SIZE DISTRIBUTION

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FIG. C2
Sheet 3 of 3

FIG. C2



NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913, sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount finer than #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in this report.
2) Group Name and Group Symbol are in accordance with ASTM D2487 and are based on laboratory testing where laboratory testing was performed.
* Sample specimen weight did not meet required minimum mass for ASTM test method.

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL*	GROUP NAME	C_c	C_u	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-2, S-4	10.0	SM	*Silty Sand*	2.0	4.1	0.0	87.8	12.2	21	
■ B-2, S-8	30.0	SM	*Silty Sand*	12.8	73.5	12.7	60.2	27.2	25	
▲ B-6, S-3B	11.0	MH	*Elastic Silt with Sand*			0.7	33.8	75.5	39	
◆ B-6, S-5*	15.0	SM	*Silty Sand with Gravel*	1.1	115.8	16.0	49.1	34.9	36	
○ B-7, S-2	5.0	OH	*Gravelly Organic Soil with Sand*	2.6	41.0	19.8	16.2	64.0	95	
□ B-7, S-6	20.0	SM	*Silty Sand*	3.5	7.8	3.6	80.9	15.1	20	

Silka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

GRAIN SIZE DISTRIBUTION

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FIG. C3

FIG. C3

Silka Sedge Tidal Wetland Restoration
Cloverdale, Oregon

TEST SUMMARY
Boring B-05, Sample S-3, 9 ft

SPECIMEN DATA AND TEST RESULTS

Sample Classification:
Very soft to soft, dark brown, Organic Soil (OH); moist; trace to few, fine to coarse sand; high plasticity; few to little organics. (OH)

Specific Gravity, G_s (Assumed)	2.7
Liquid Limit, LL	—
Plastic Limit, PL	—
Plasticity Index, PI (LL - PL)	—
Fines Content	—
Organic Content	—
Initial Seating Load, g	50
Final Seating Load, g	50
ASTM Test Method	Method B
Coefficient of Consolidation Interpretation	Procedures 1 & 2

	Pre-Inundation	Final Load
Height, in	0.837	0.663
Diameter, in	2.501	2.501
Specimen Volume, in³	4.112	3.257
Wet Unit Weight, pcf	97.6	108.1
Dry Unit Weight, pcf	55.8	70.5
Water Content	75%	53%
Void Ratio	2.02	1.39
Degree of Saturation	100%	100%

Load Increment Number	Applied Stress, tsf	t_{load}, min	t_{50}, min	ΔH_{load}, in	ΔH at t_{50}, in	$\Delta H/H_o$	Void Ratio	a_v, Mpa^{-1}	C_v, cm²/s	k, cm/s	C_c, log(min)$^{-1}$
*	0.00										
1	0.03	1330	0.2138	-0.007	-0.006	0.7%	1.997	7.57E+00	1.71E-02	4.21E-06	1.1E-03
2	0.06	1150	0.1778	-0.006	-0.011	1.3%	1.979	5.76E+00	2.03E-02	3.83E-06	2.0E-03
3	0.13	2830	6.6069	-0.011	-0.022	2.7%	1.939	6.45E+00	5.34E-04	1.14E-07	3.6E-03
4	0.26	1810	4.2658	-0.014	-0.036	4.3%	1.888	4.08E+00	8.01E-04	1.09E-07	2.9E-03
5	0.52	2830	2.3442	-0.022	-0.056	6.7%	1.816	2.94E+00	1.40E-03	1.39E-07	3.3E-03
6	1.03	2410	3.2359	-0.024	-0.082	9.8%	1.723	1.88E+00	9.50E-04	6.21E-08	3.9E-03
7	2.07	1750	1.9055	-0.028	-0.111	13.3%	1.618	1.05E+00	1.49E-03	5.68E-08	4.1E-03
8	4.14	4330	1.9498	-0.036	-0.148	17.7%	1.486	6.68E-01	1.32E-03	3.32E-08	4.1E-03
9	8.27	3790	2.2131	-0.037	-0.187	22.4%	1.343	3.60E-01	1.04E-03	1.48E-08	5.0E-03
10	4.14	310	0.0132	0.001	-0.186	23.3%	1.314	-7.38E-02	1.94E-01	5.98E-07	
11	2.07	1630	0.0324	0.001	-0.183	23%	1.323	4.65E-02	7.72E-02	1.52E-07	
12	1.03	120	0.0169	0.002	-0.180	22.7%	1.333	9.82E-02	1.36E-01	5.63E-07	
13	0.26	120	0.0121	0.002	-0.187	22.3%	1.346	1.76E-01	1.99E-01	1.48E-06	
14	0.06	1020	0.0169	0.004	-0.184	21.9%	1.357	5.79E-01	1.56E-01	3.77E-06	
15	0.03	300	0.0196	0.003	-0.178	21.3%	1.376	6.28E+00	1.46E-01	3.83E-05	

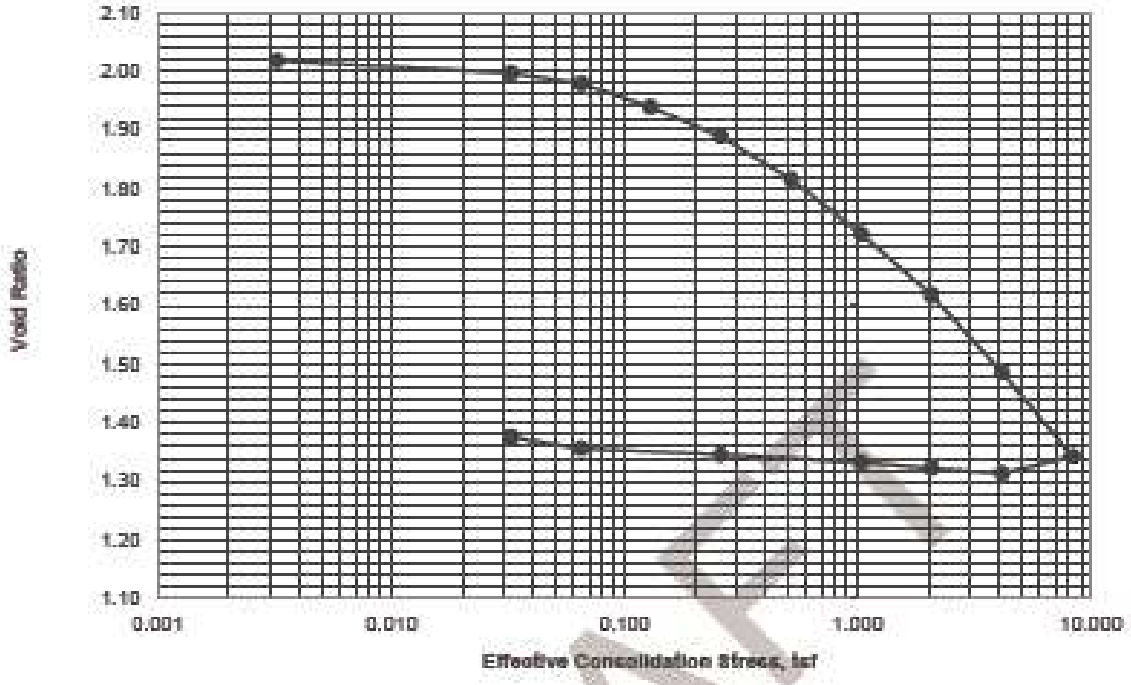
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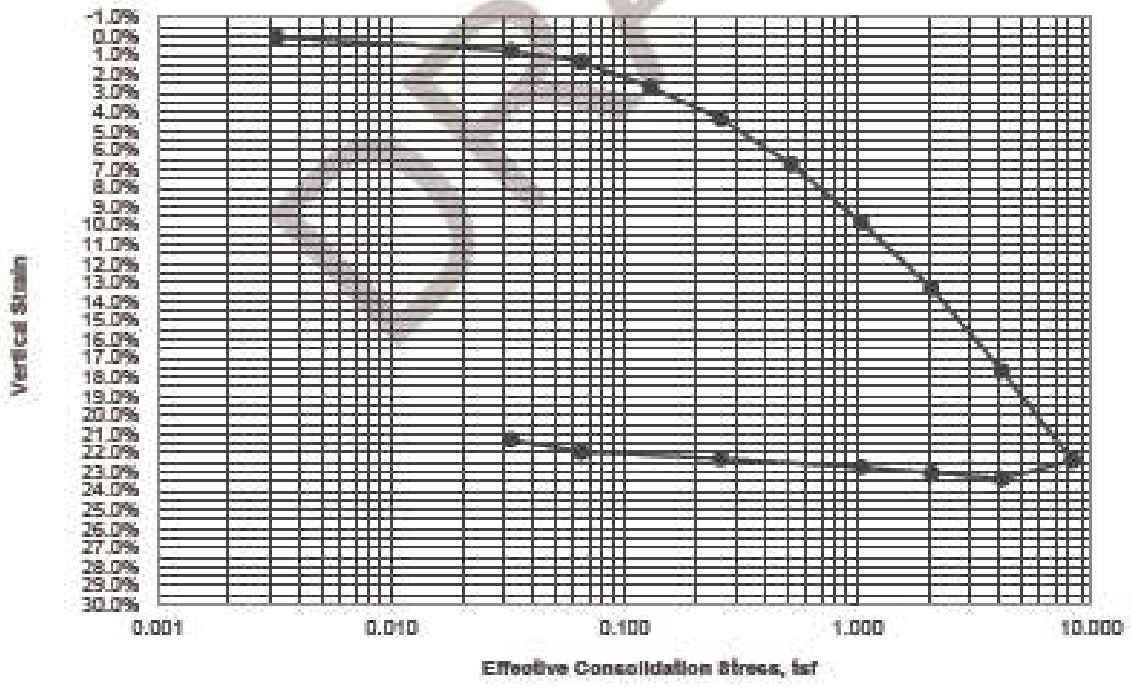
Specimen was remolded.
Tested by DJB in accordance with ASTM D2435-11. Test initiated on 02/05/2026. Finalized by DJB.

FIG. C4

Silka Sedge Tidal Wetland Restoration
Cloverdale, Oregon

CONSOLIDATION CURVES
Boring B-05, Sample S-3, 9 ft





Maximum Applied Effective Consolidation Stress, tsf = 8.27

Specimen was remolded.
Tested by DJB in accordance with ASTM D2435-11. Test initiated on 02/05/2026. Finalized by DJB.

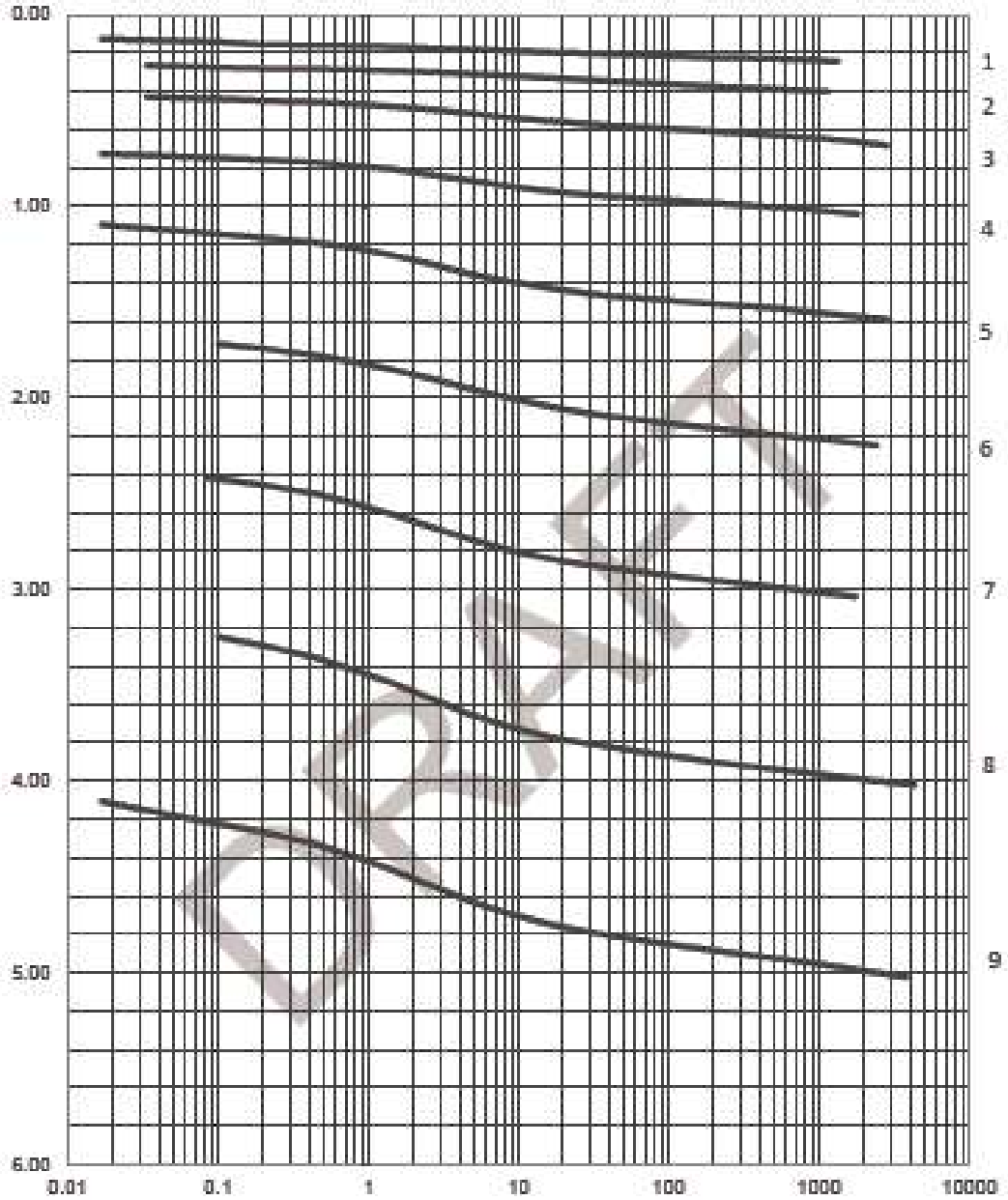
FIG. C4

DRAFT

1123000_S-5_S-3_Consol Rep.t_Dead Weight_GB.xlsm 03/04/26
1123000

Silka Sedge Tidal Wetland Restoration
Cloverdale, Oregon

LOAD INCREMENT TIME-DEFORMATION CURVES
Boring B-05, Sample S-3, 9 ft



Load Sequence, tsf = 0.03, 0.06, 0.13, 0.26, 0.52, 1.03, 2.06, 4.13, 8.26

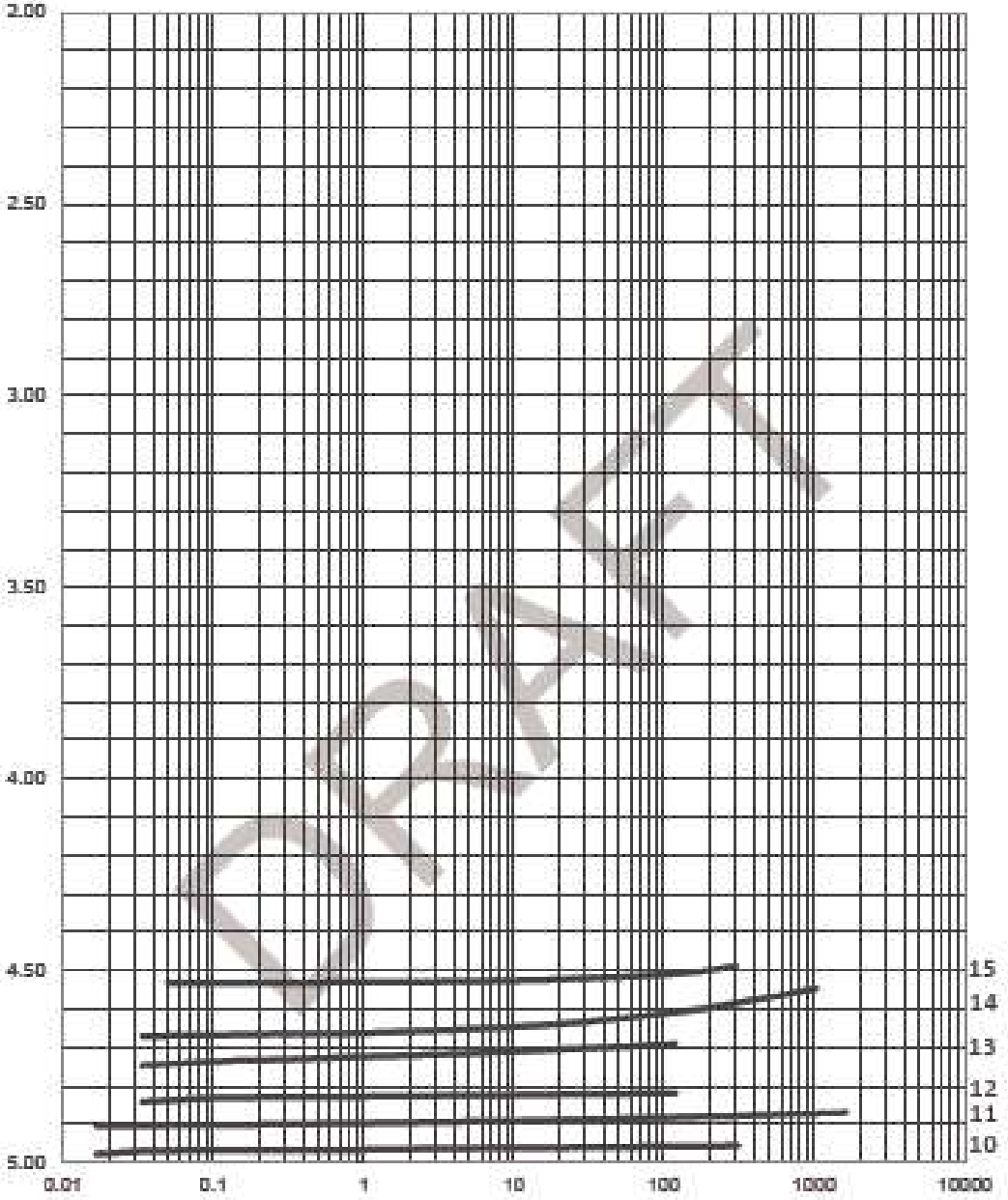
Specimen was remolded.
Tested by DJB in accordance with ASTM D2435-11. Test initiated on 02/05/2026. Finalized by DJB.

FIG. C4

112080_B-5_S-3_Consol Report_Dead Weight_DJB.xlsm 03/04/26
112080

Silka Sedge Tidal Wetland Restoration
Cloverdale, Oregon

UNLOAD INCREMENT TIME-DEFORMATION CURVES
Boring B-05, Sample S-3, 9 ft



DRAFT

UnLoad Sequence, tsf =4.13, 2.06, 1.03, 0.26, 0.06, 0.03

Specimen was remolded.
Tested by DJB in accordance with ASTM D2435-11. Test initiated on 02/05/2026. Finalized by DJB.

FIG. C4

112000_B-5_S-3_Consol Report_Dead Weight_DJB.xlsm 03/04/26
112000

Silka Sedge Tidal Wetland Restoration
Cloverdale, Oregon

TEST SUMMARY
Boring B-07, Sample S-1, 3 ft

SPECIMEN DATA AND TEST RESULTS

Sample Classification:
Soft to medium stiff, dark brown to olive brown, elastic silt with gravel (MH); wet; fine, subangular gravel; trace to few fine to medium sand; medium plasticity; trace to few fine organic debris. (MH)

Specific Gravity, G_s (Assumed)	2.7
Liquid Limit, LL	—
Plastic Limit, PL	—
Plasticity Index, PI (LL - PL)	—
Fines Content	—
Organic Content	—
Initial Seating Load, g	50
Final Seating Load, g	50
ASTM Test Method	Method B
Coefficient of Consolidation Interpretation	Procedures 1 & 2

	Pre-Inundation	Final Load
Height, in	0.856	0.665
Diameter, in	2.501	2.501
Specimen Volume, in^3	4.202	3.265
Wet Unit Weight, pcf	97.8	108.3
Dry Unit Weight, pcf	56.0	72.0
Water Content	75%	50%
Void Ratio	2.01	1.34
Degree of Saturation	100%	100%

Load Increment Number	Applied Stress, tsf	t_{50}, min	t_{90}, min	ΔH_{final}, in	ΔH at t_{100}, in	$\Delta H/H_o$	Void Ratio	a_v, Mpa^{-1}	c_v, cm^2/s	k, cm/s	C_c, log(min)$^{-1}$
*	0.02										
1	0.03	1270	6.026	-0.011	-0.009	1.1%	1.98	2E+01	6E-04	4E-07	2E-03
2	0.06	3130	12.589	-0.008	-0.017	2%	1.95	9E+00	3E-04	9E-08	1E-03
3	0.13	3970	14.125	-0.012	-0.028	3.3%	1.91	6E+00	3E-04	5E-08	2E-03
4	0.26	370	2.951	-0.013	-0.041	4.8%	1.87	4E+00	1E-03	2E-07	3E-03
5	0.52	1331	3.236	-0.019	-0.059	6.9%	1.80	3E+00	1E-03	9E-08	3E-03
6	1.03	1150	2.630	-0.024	-0.083	9.7%	1.72	2E+00	1E-03	7E-08	4E-03
7	2.07	1390	1.698	-0.029	-0.111	13%	1.62	1E+00	2E-03	6E-08	4E-03
8	4.14	3250	1.862	-0.035	-0.145	17%	1.50	6E-01	1E-03	3E-08	4E-03
9	8.27	2830	2.884	-0.038	-0.186	21.7%	1.36	4E-01	9E-04	1E-08	4E-03
10	4.14	2410	0.005	0.002	-0.190	22.3%	1.34	-4E-02	5E-01	9E-07	
11	2.07	1390	0.010	0.009	-0.188	22%	1.35	4E-02	1E-01	2E-07	
12	1.03	1680	0.005	0.002	-0.186	21.7%	1.36	7E-02	5E-01	1E-06	
13	0.52	2950	0.012	0.003	-0.184	21.4%	1.37	3E-01	2E-01	3E-06	
14	0.26	910	0.048	0.002	-0.180	21.1%	1.38	4E-01	7E-02	1E-06	
15	0.13	370	0.012	0.001	-0.178	20.8%	1.38	6E-01	3E-01	6E-06	
16	0.06	2350	0.023	0.003	-0.177	20.7%	1.39	8E-01	1E-01	4E-06	
17	0.03	1990	0.010	0.001	-0.174	20.4%	1.40	3E+00	3E-01	4E-05	

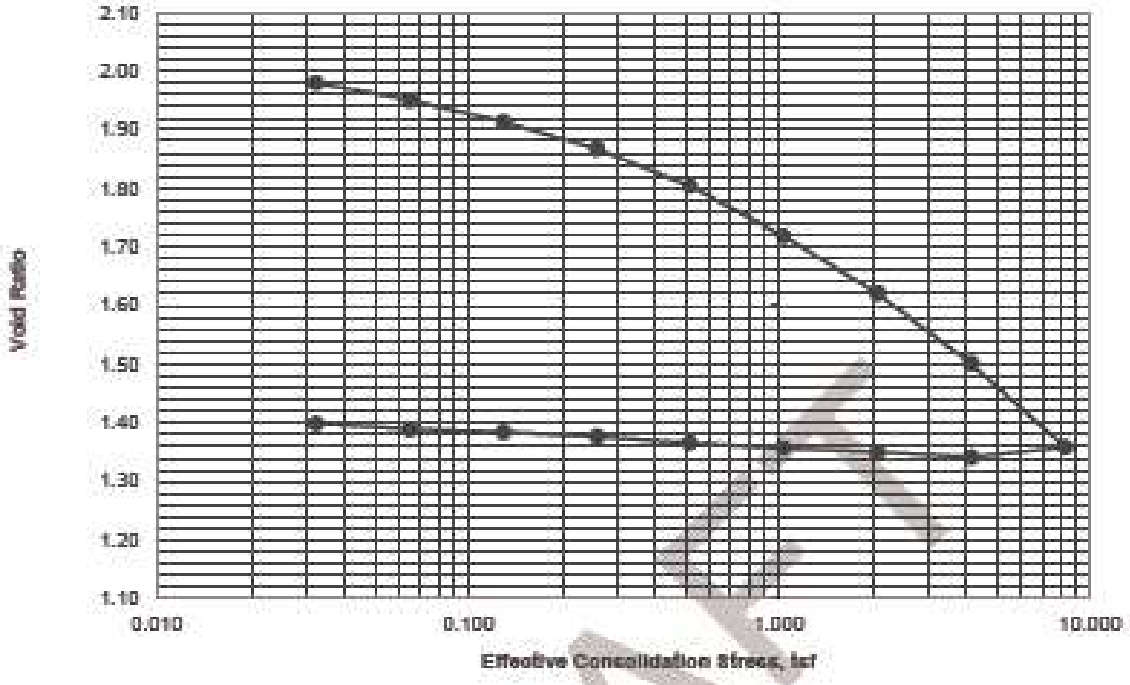
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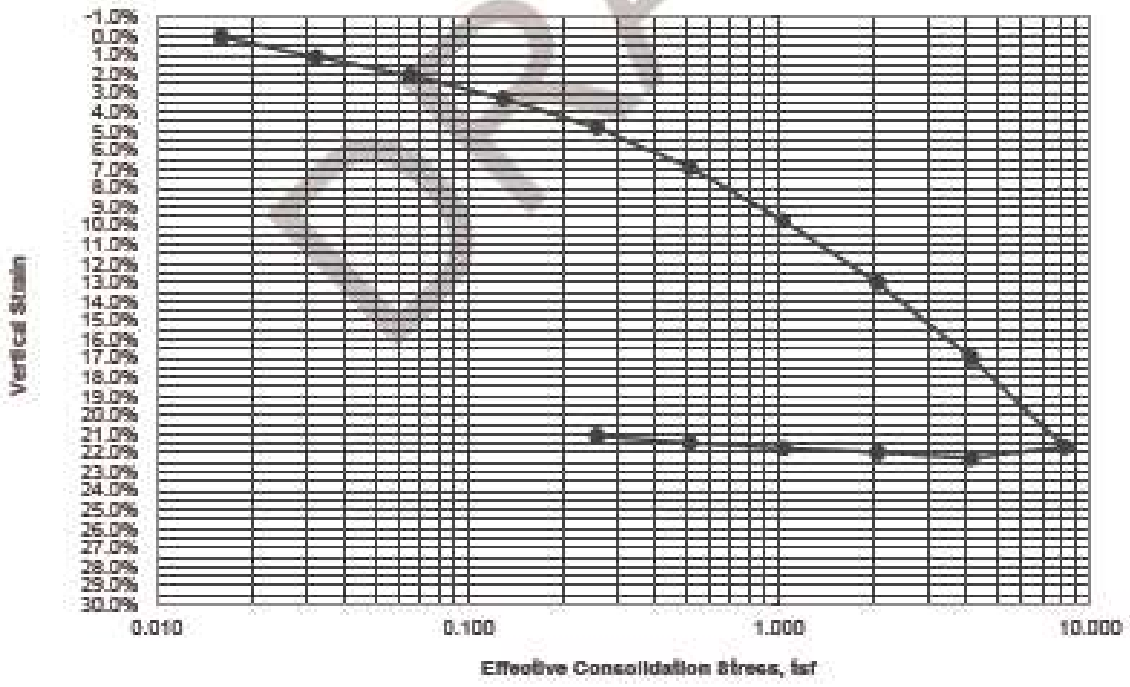
Specimen was remolded.
Tested by DJB in accordance with ASTM D2435-11. Test initiated on 02/03/2026. Finalized by DJB.

FIG. C5

Silka Sedge Tidal Wetland Restoration
Cloverdale, Oregon

CONSOLIDATION CURVES
Boring B-07, Sample S-1, 3 ft





Maximum Applied Effective Consolidation Stress, tsf = 8.27

DRAFT

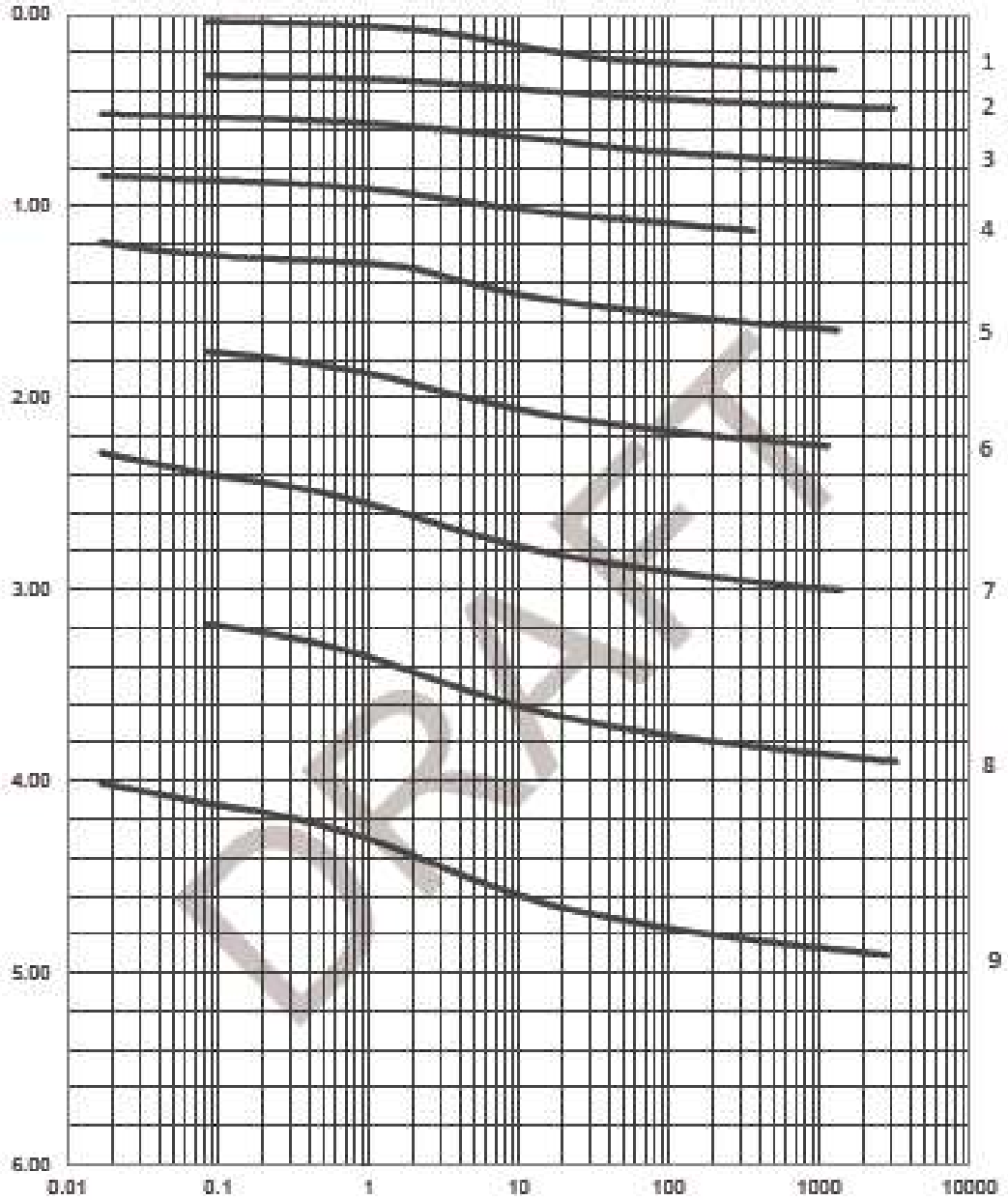
Specimen was remolded.
Tested by DJB in accordance with ASTM D2435-11. Test initiated on 02/03/2026. Finalized by DJB.

FIG. C5

1123000_B-7_S-1_Consol Rept_Dead Weight_GIS.xlsm 03/04/26
1123000

Silka Sedge Tidal Wetland Restoration
Cloverdale, Oregon

LOAD INCREMENT TIME-DEFORMATION CURVES
Boring B-07, Sample S-1, 3 ft



Load Sequence, tsf = 0.03, 0.06, 0.13, 0.26, 0.52, 1.03, 2.06, 4.13, 8.26

Specimen was remolded.
Tested by DJB in accordance with ASTM D2435-11. Test initiated on 02/03/2026. Finalized by DJB.

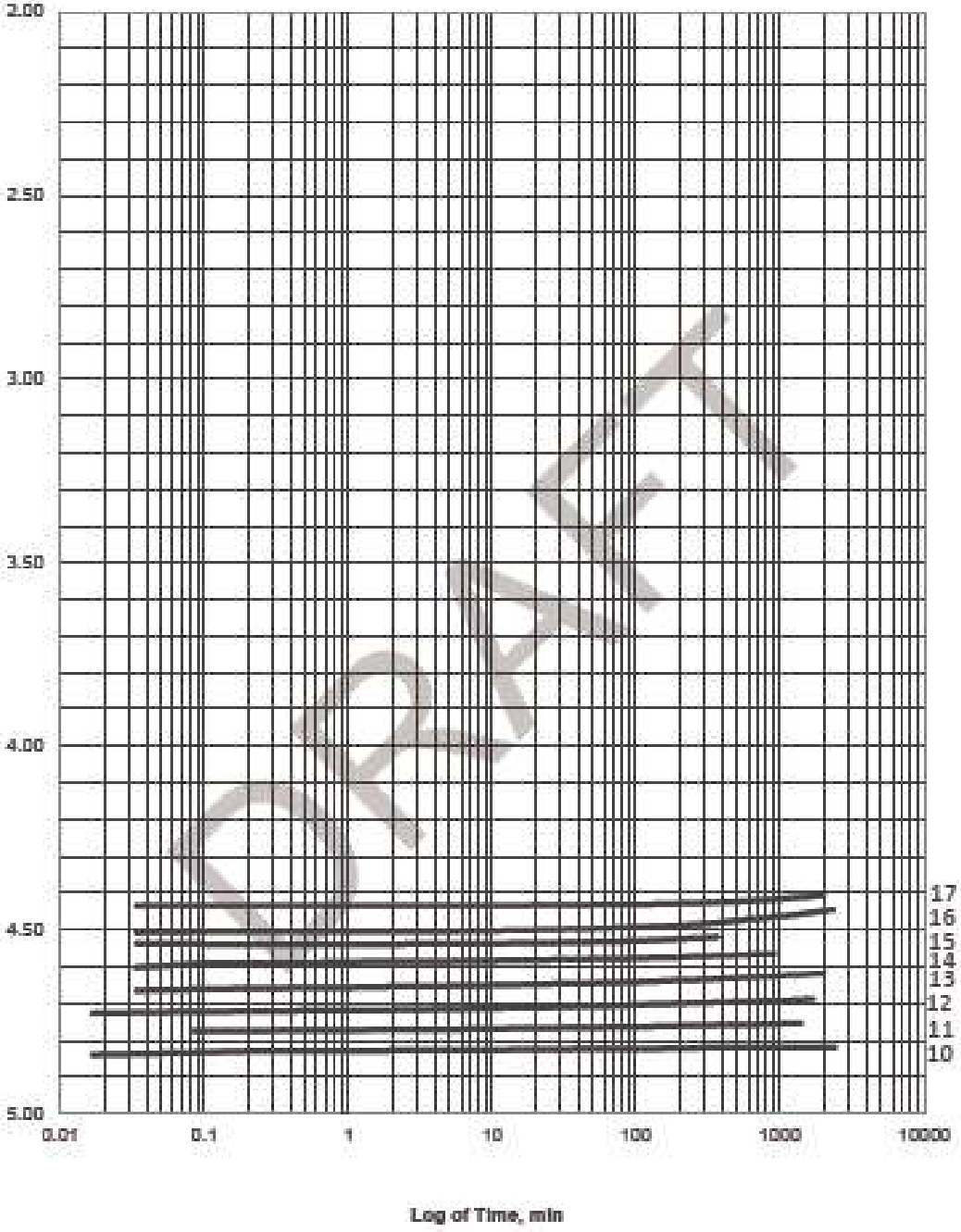
FIG. C5

DRAFT

1123000_B-7_S-1_Consol Rept_Dead Weight_DJB.xlsm 03/04/26
1123000

Silka Sedge Tidal Wetland Restoration
Cloverdale, Oregon

UNLOAD INCREMENT TIME-DEFORMATION CURVES
Boring B-07, Sample S-1, 3 ft



DRAFT

UnLoad Sequence, tsf =4.13, 2.06, 1.03, 0.52, 0.26, 0.13, 0.06, 0.03

Specimen was remolded.
Tested by DJB in accordance with ASTM D2435-11. Test initiated on 02/03/2026. Finalized by DJB.

FIG. C5

COOPER

Corrosivity Tests Summary

CTL #: 040-229 Date: 5/2/2025 Tested By: PJ Checked: PJ

Client: Shannon & Wilson Project: Stike Sedge Proj. No: 112000

Remarks:

Sample Location or ID			Resistivity @ 15.6 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP (Redox)		Sulfide Qualitative by Lead Acetate Paper	Moisture At Test % ASTM D2216	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg Dry Wt.	mg/kg Dry Wt.	% Dry Wt.		E_H (mV) ASTM D1498	At Test Temp °C			
Boring	Sample. No.	Depth, ft.	ASTM G57	Calc'd	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4972	ASTM D1498				
B-1	-	10	183	-	-	3,847	2,070	0.2070	7.1	437	20	-	37.5	Gray Clayey SAND w/ organics
B-3	-	2.5	13,813	-	-	39	155	0.0155	6.6	395	20	-	61.4	Brown Sandy CLAY w/ Gravel

DRAFT

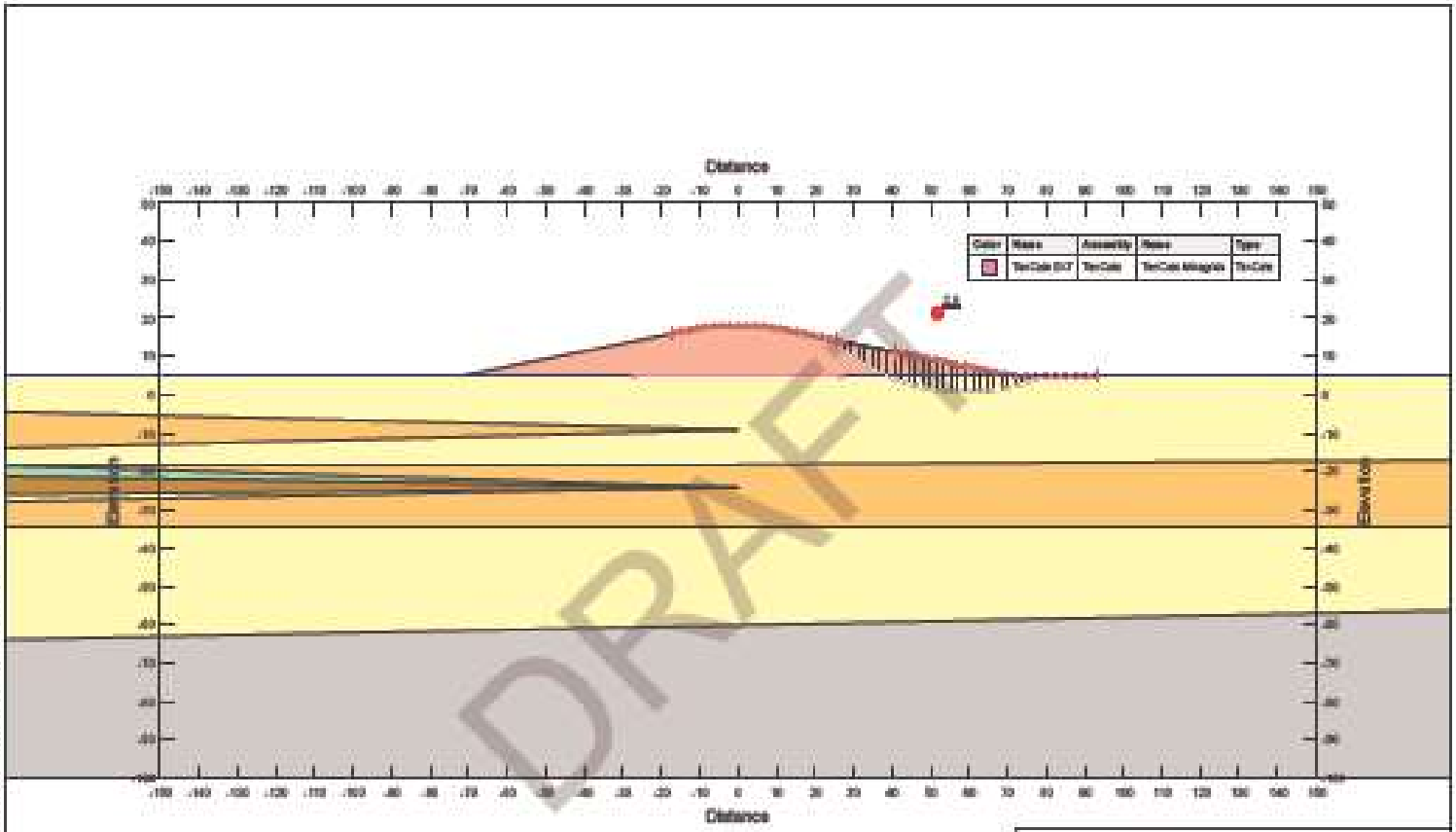
Appendix D

Seepage and Slope Stability Analysis

Figures

Figure D1: Stability Analysis, End of Construction, Landside
Figure D2: Stability Analysis, End of Construction, Waterside
Figure D3: Stability Analysis, Rapid Drawdown
Figure D4: Stability Analysis, Rapid Drawdown with Geogrid
Figure D5: Seepage Analysis, Steady-State
Figure D6: Stability Analysis, Steady Seepage
Figure D7: Stability Analysis, Steady Seepage with Geogrid
Figure D8: Stability Analysis (Sensitivity), End of Construction, Landside with Marsh Deposits
Figure D9: Stability Analysis (Sensitivity), End of Construction, Waterside with Marsh Deposits
Figure D10: Stability Analysis (Sensitivity), Rapid Drawdown, Marsh Deposits
Figure D11: Stability Analysis (Sensitivity), Rapid Drawdown with Geogrid, Marsh Deposits
Figure D12: Seepage Analysis (Sensitivity), Steady-State, Marsh Deposits
Figure D13: Stability Analysis (Sensitivity), Steady Seepage, Marsh Deposits
Figure D14: Stability Analysis (Sensitivity), Steady Seepage with Geogrid, Marsh Deposits
Figure D15: Seepage Analysis (Sensitivity), Transient King Tides, Marsh Deposits
Figure D16: Seepage Analysis (Sensitivity), Transient Smaller Tides, Marsh Deposits
Figure D17: Seepage Analysis (Sensitivity), Transient Smaller Tides, Marsh Deposits, Ground Surface 5 ft
Figure D18: Seepage Analysis (Sensitivity), Transient Smaller Tides, Marsh Deposits, Ground Surface 6 ft
Figure D19: Seepage Analysis (Sensitivity), Transient Smaller Tides, Marsh Deposits, Ground Surface 7 ft
Figure D20: Seepage Analysis (Sensitivity), Transient Smaller Tides, Marsh Deposits, Ground Surface 8 ft
Figure D21: Seepage Analysis (Sensitivity), Transient Smaller Tides, Marsh Deposits, Ground Surface 9 ft
Figure D22: Seepage Analysis (Sensitivity), Transient Smaller Tides, Marsh Deposits, Ground Surface 10 ft

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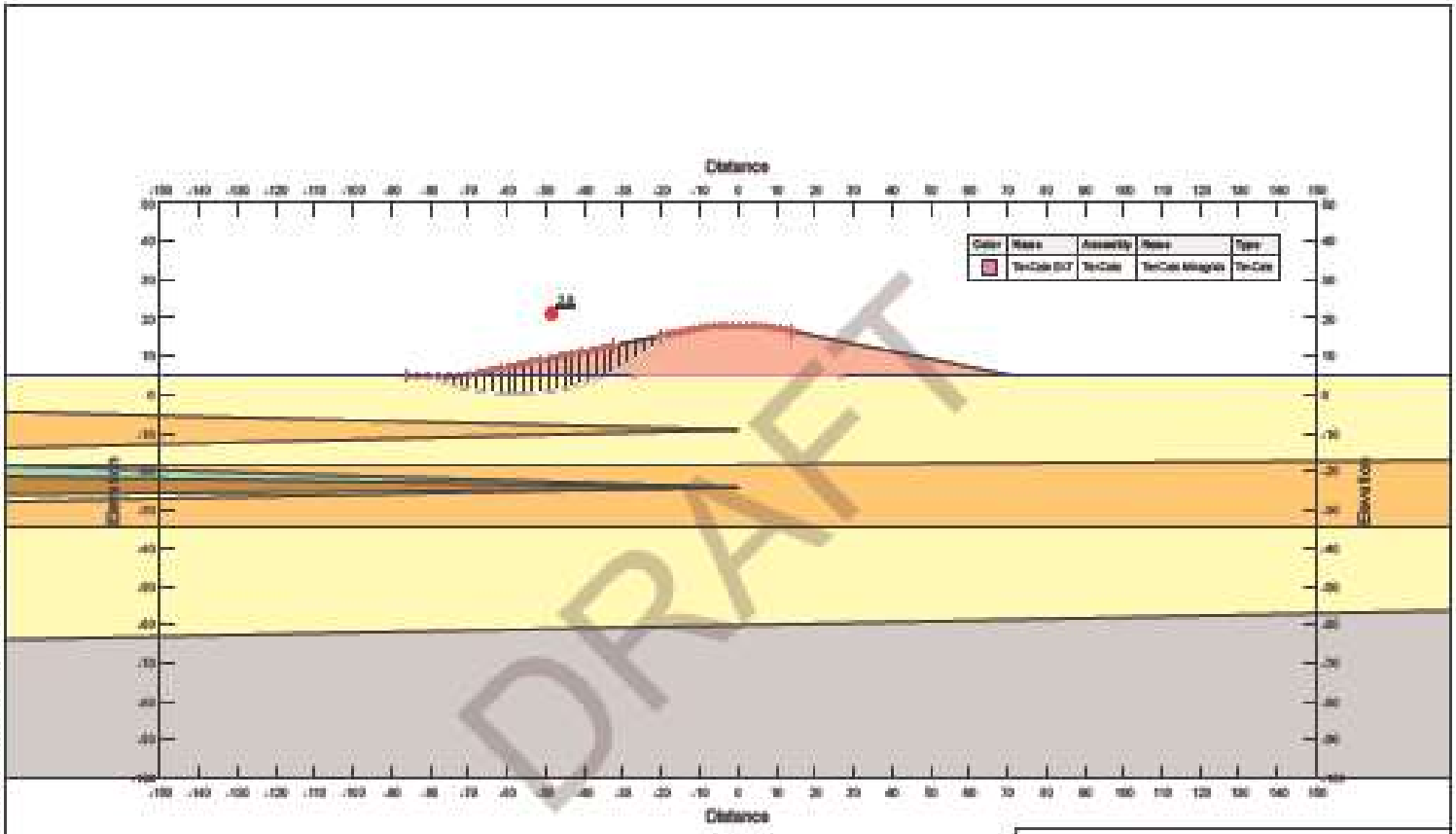
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Stability Analysis
End of Construction
Landside

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FIG. D1



Color	Name	Assembly	Name	Type
	TenCate EX7	TenCate	TenCate MirageXN	TenCate

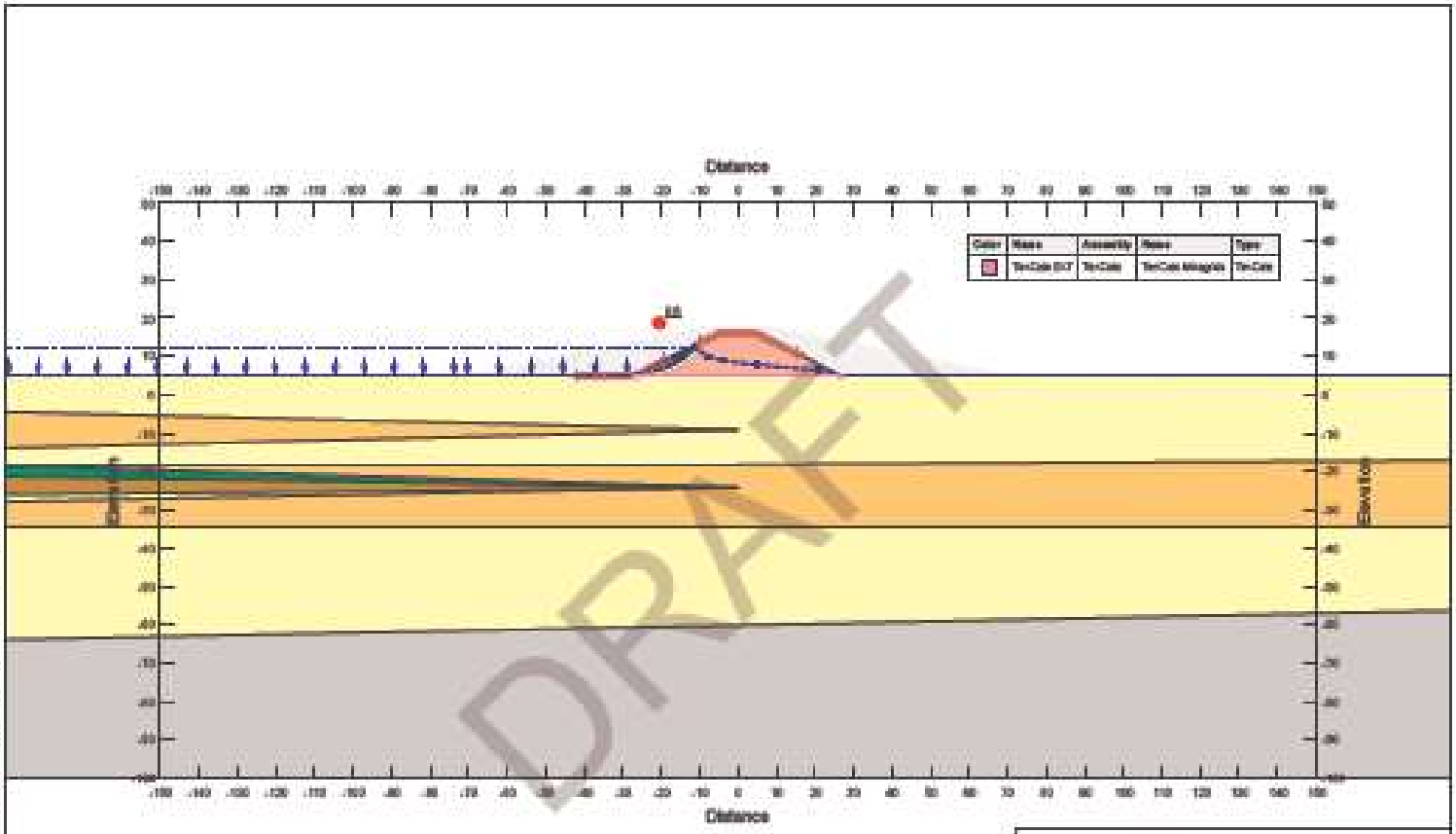
Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Alder Formation	High Strength	140		
	Old	Mohr-Coulomb	130	0	30
	Lava Flt	Mohr-Coulomb	110	0	30
	Mtl Undefined	Mohr-Coulomb	150	200	1
	Qal	Mohr-Coulomb	120	0	32
	Qf Old	Mohr-Coulomb	130	0	30

Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Stability Analysis
End of Construction
Waterside

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Color	Name	Material Behavior	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle (°)	Cohesion Ratio	Ru
	Alder Formation	High Strength	140					
	Old	Mohr-Coulomb	110	Undrained	0	0	1000	0
	Levee Fill	Mohr-Coulomb	110	Undrained	0	0	500	0
	Mh	Mohr-Coulomb	100	Undrained	0	0	500	0
	Qal	Mohr-Coulomb	120	Undrained	0	0	0	0
	Qf-Bm	Mohr-Coulomb	120	Undrained	0	0	0	0

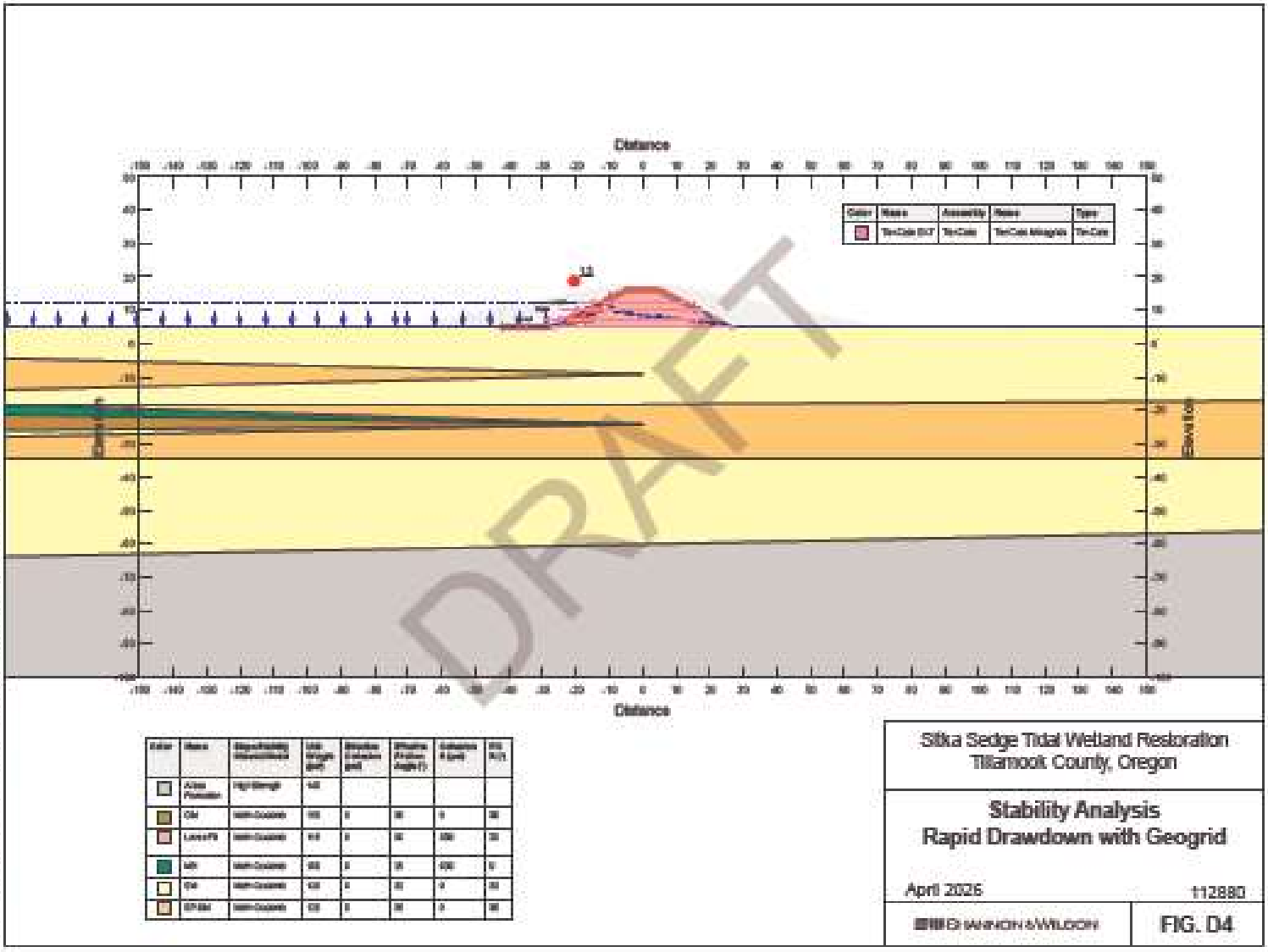
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Stability Analysis
Rapid Drawdown

April 2026 112880

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FIG. D3



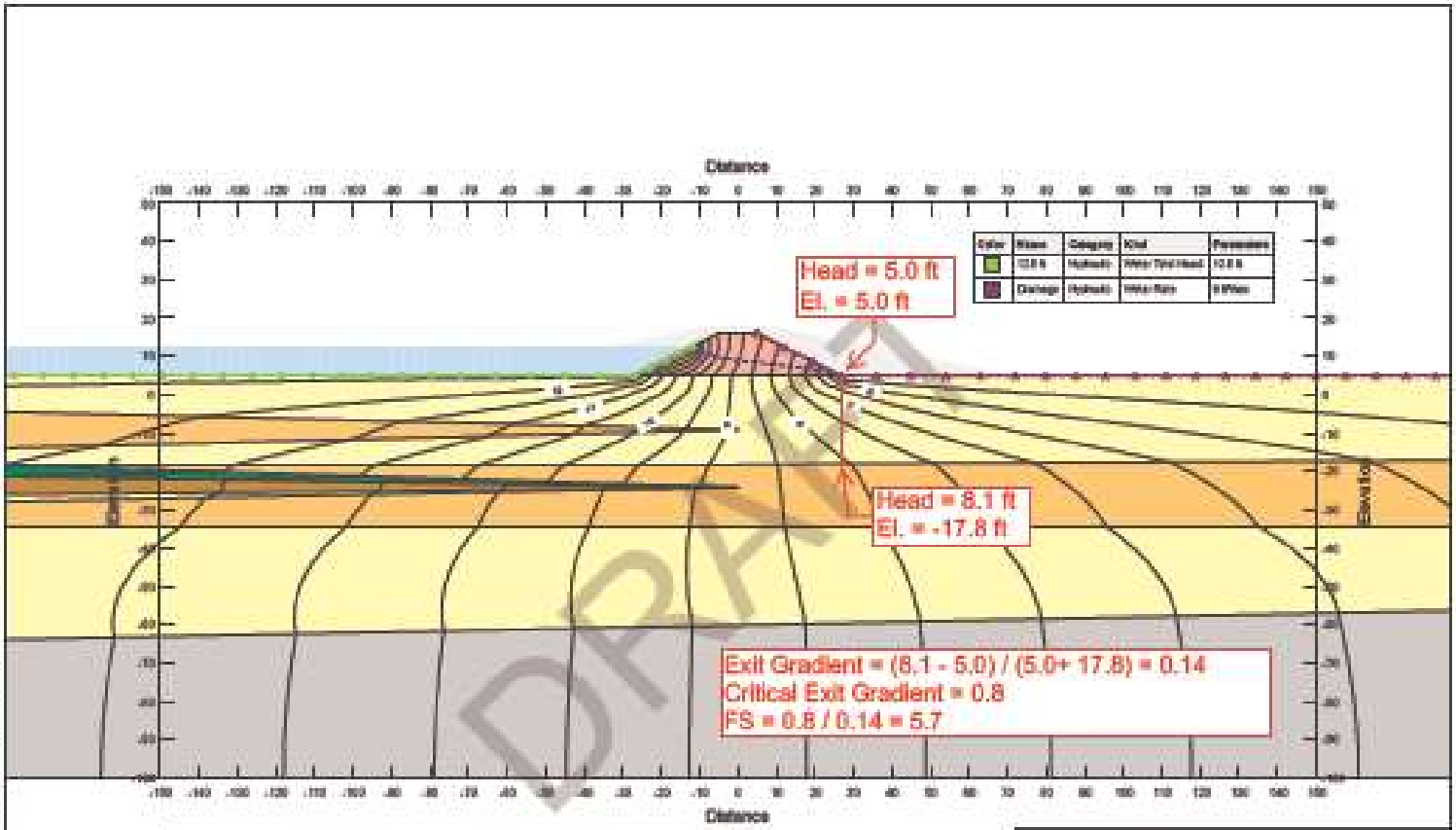
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Stability Analysis
Rapid Drawdown with Geogrid

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FIG. D4



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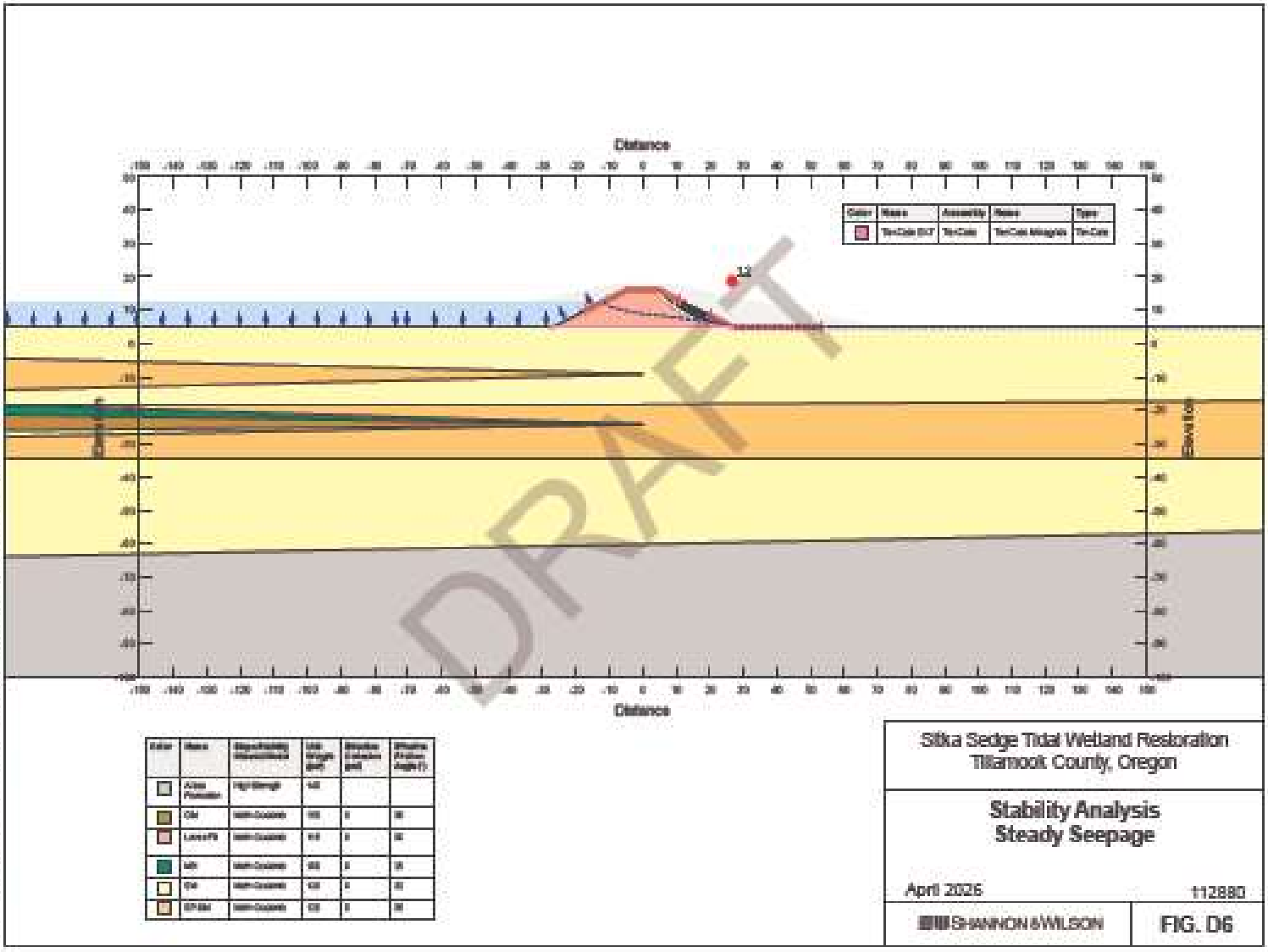
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Seepage Analysis
Steady-State

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FIG. D5



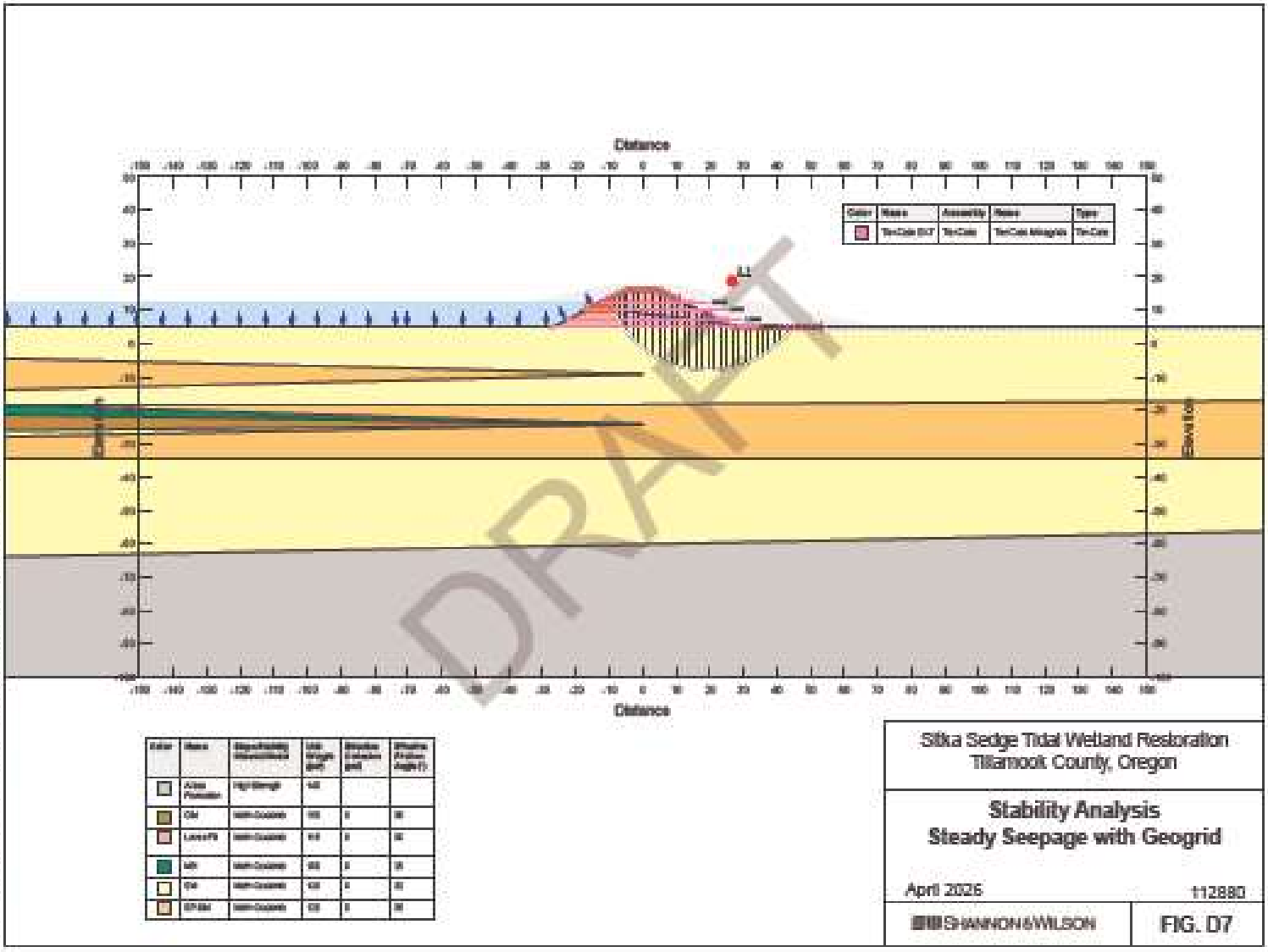
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Stability Analysis
Steady Seepage

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FIG. D6



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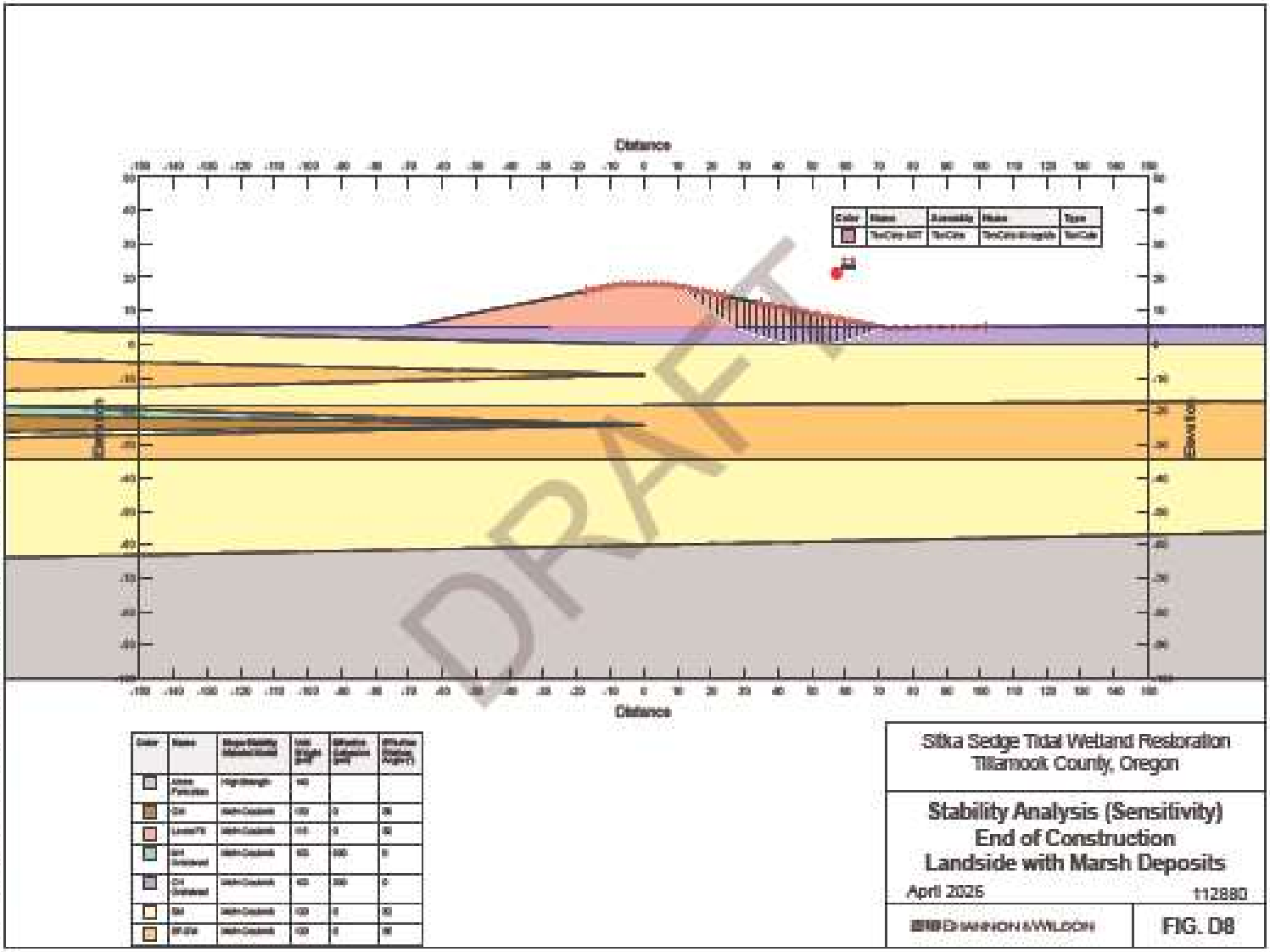
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Stability Analysis
Steady Seepage with Geogrid

April 2026 112880

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FIG. D7



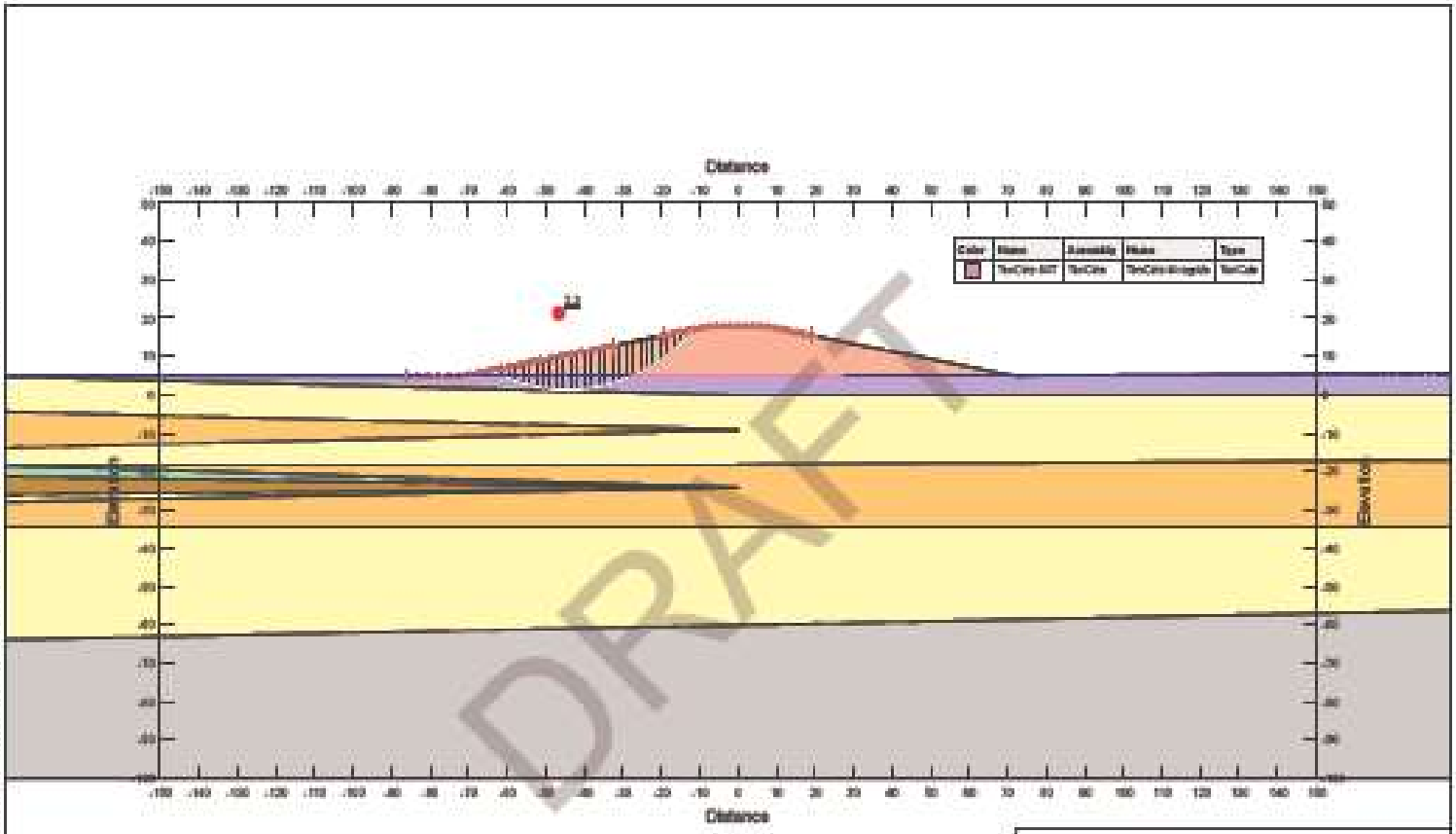
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Stability Analysis (Sensitivity)
End of Construction
Landside with Marsh Deposits

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HDR | DAVIS WILSON

FIG. D8



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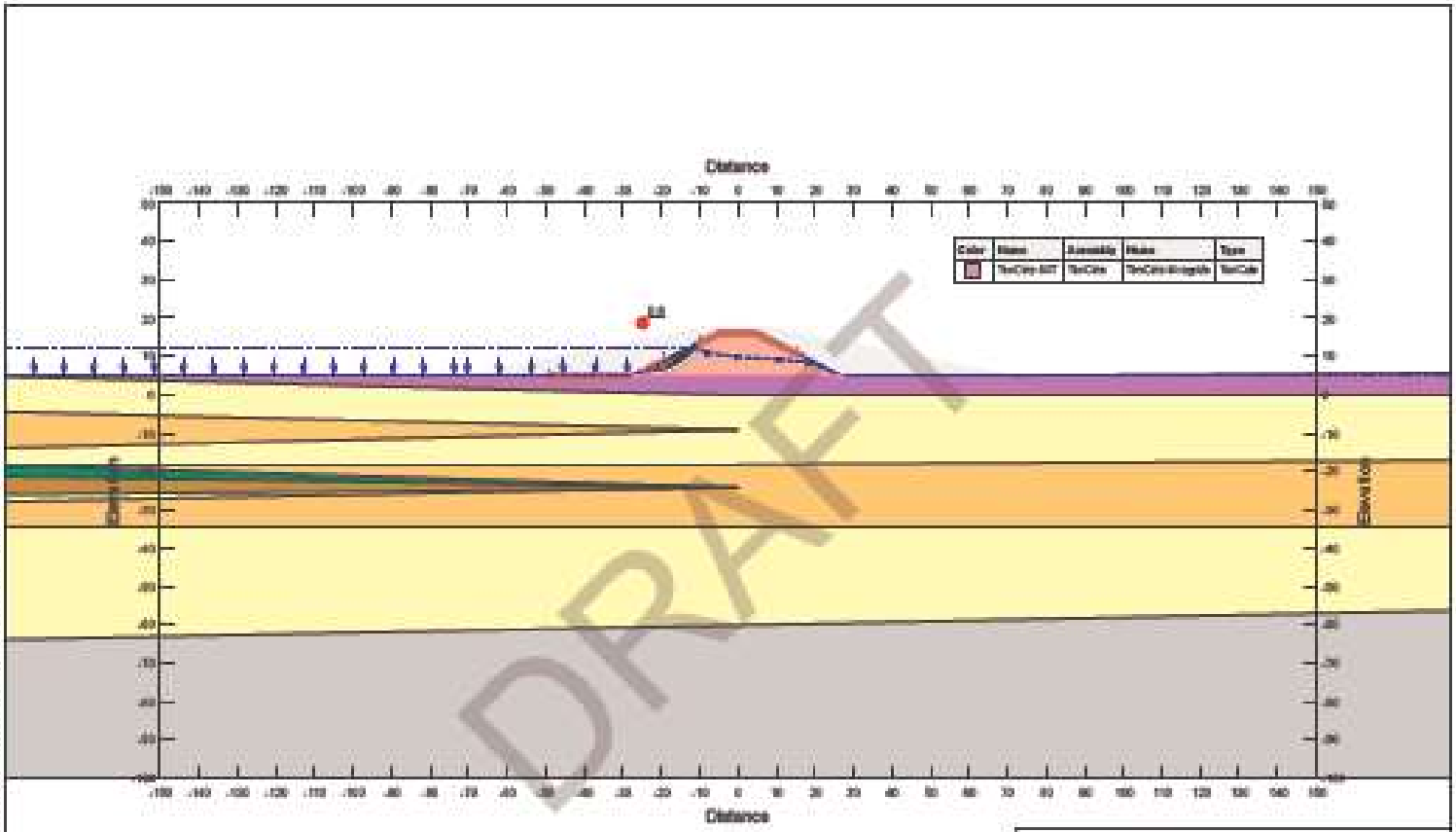
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Stability Analysis (Sensitivity)
End of Construction
Waterside with Marsh Deposits

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FIG. D9



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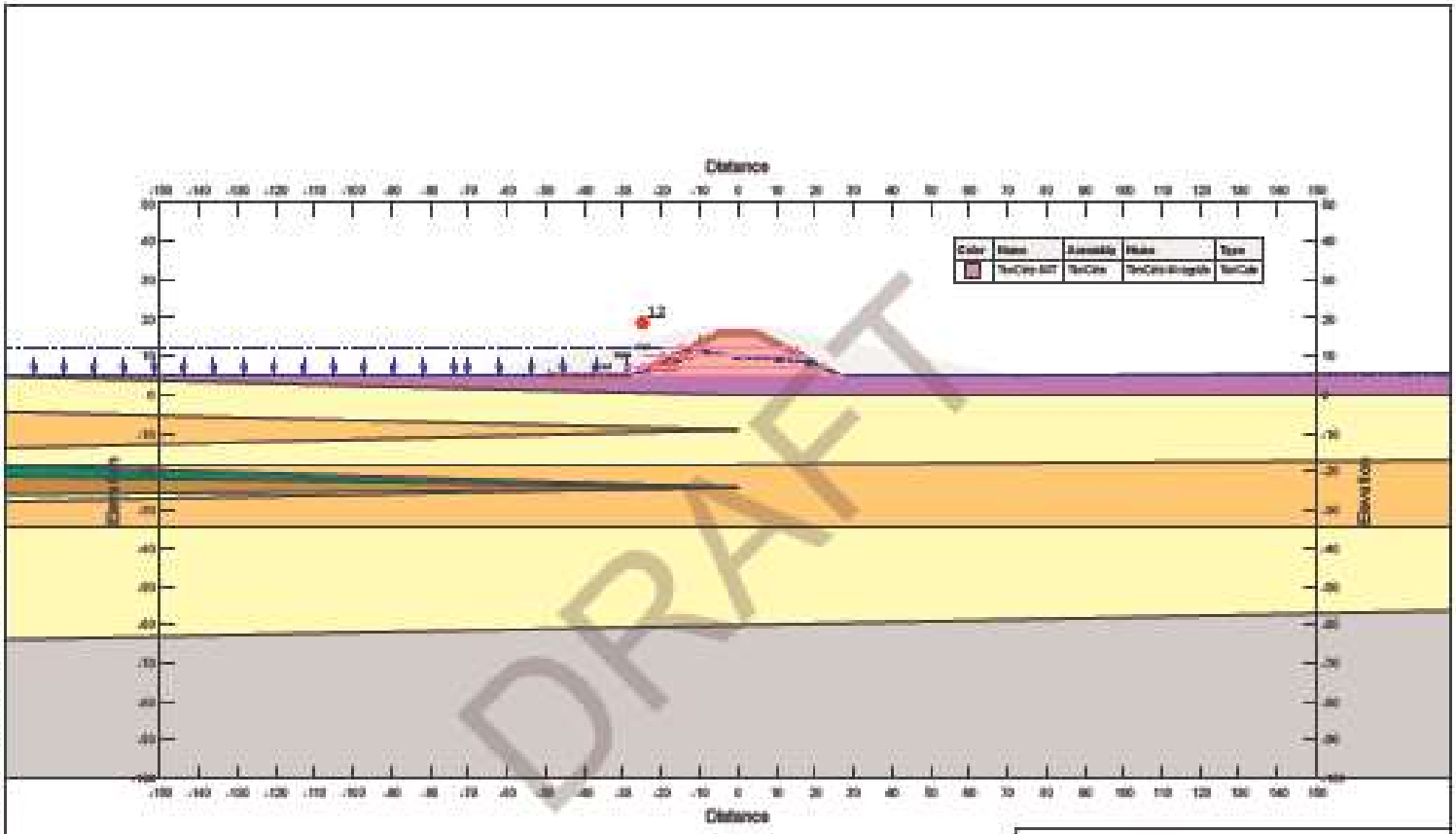
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Stability Analysis (Sensitivity)
Rapid Drawdown
Marsh Deposits

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FIG. D10



DRAFT

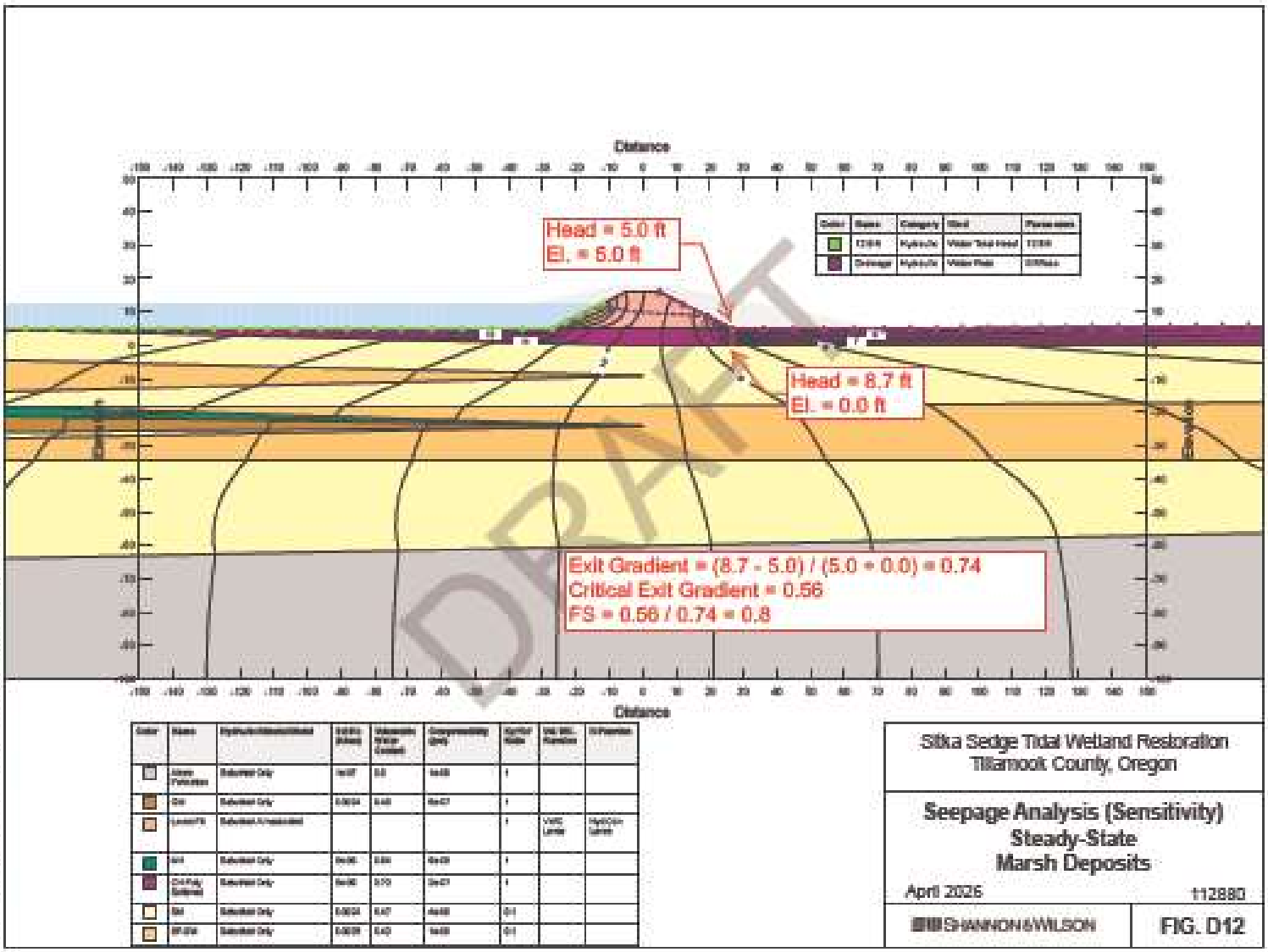
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Stability Analysis (Sensitivity)
Rapid Drawdown with Geogrid
Marsh Deposits

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FIG. D11



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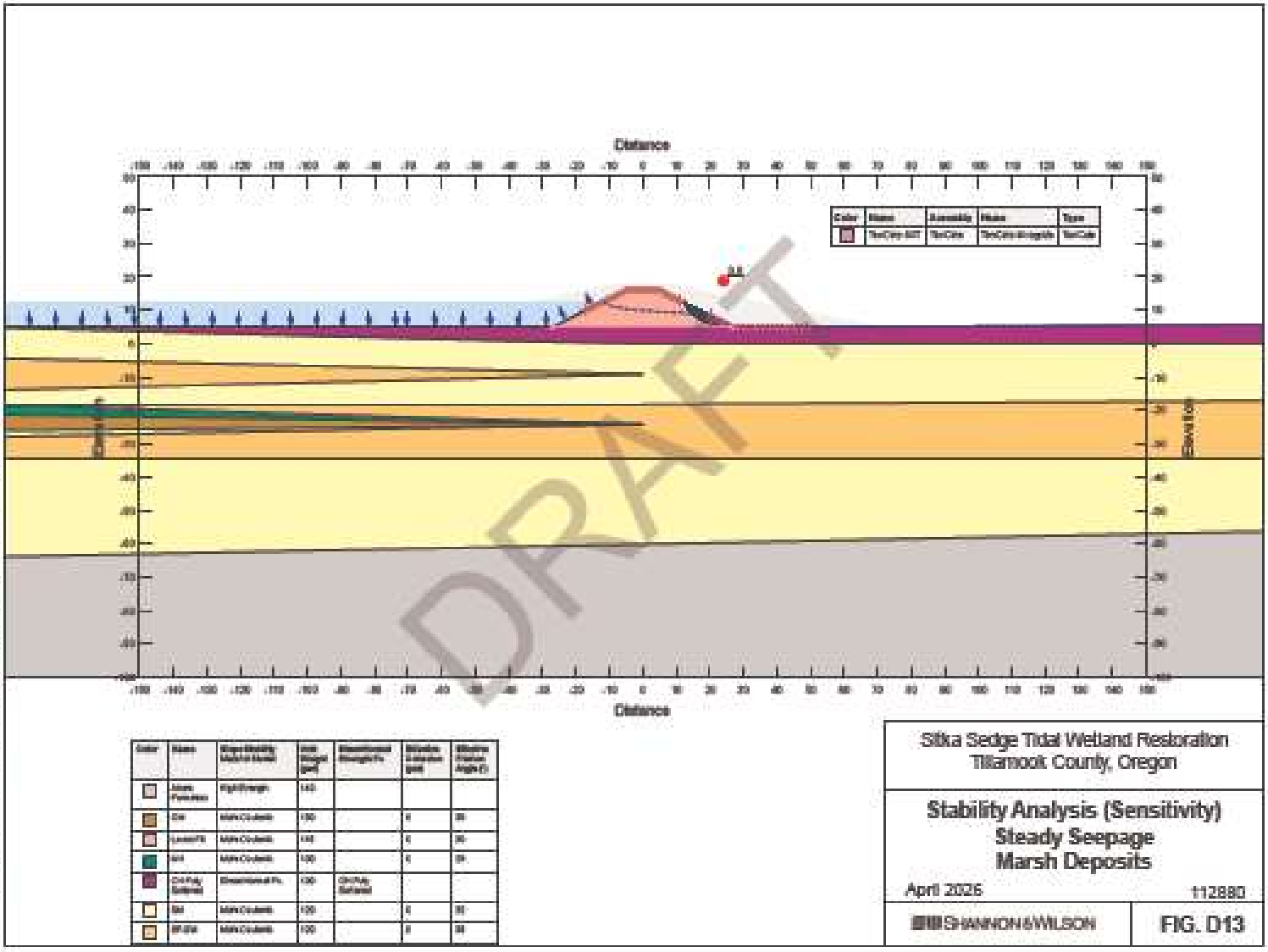
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Seepage Analysis (Sensitivity)
Steady-State
Marsh Deposits

April 2026 112880

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FIG. D12



Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

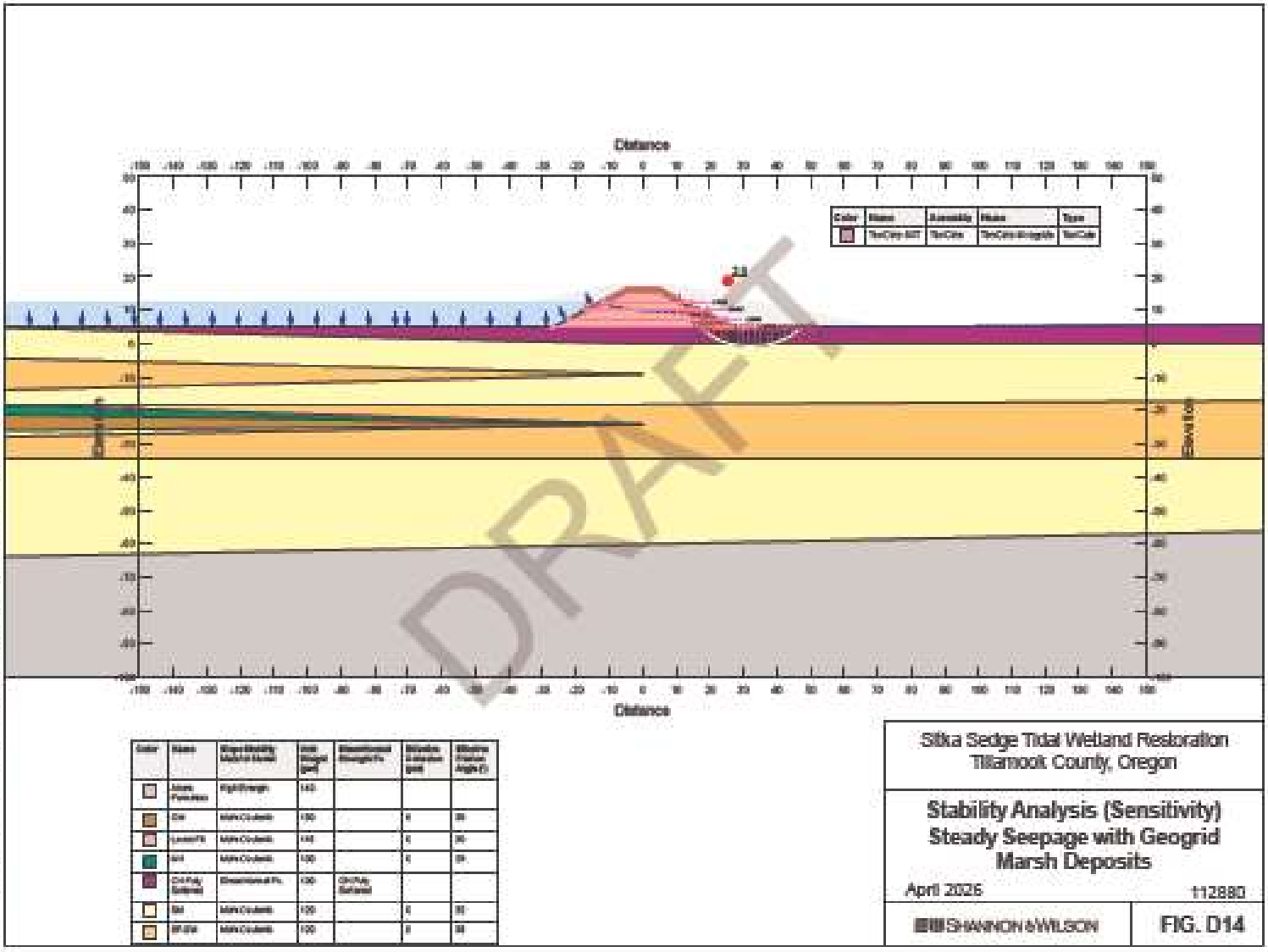
Stability Analysis (Sensitivity)
Steady Seepage
Marsh Deposits

April 2026

112880

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FIG. D13

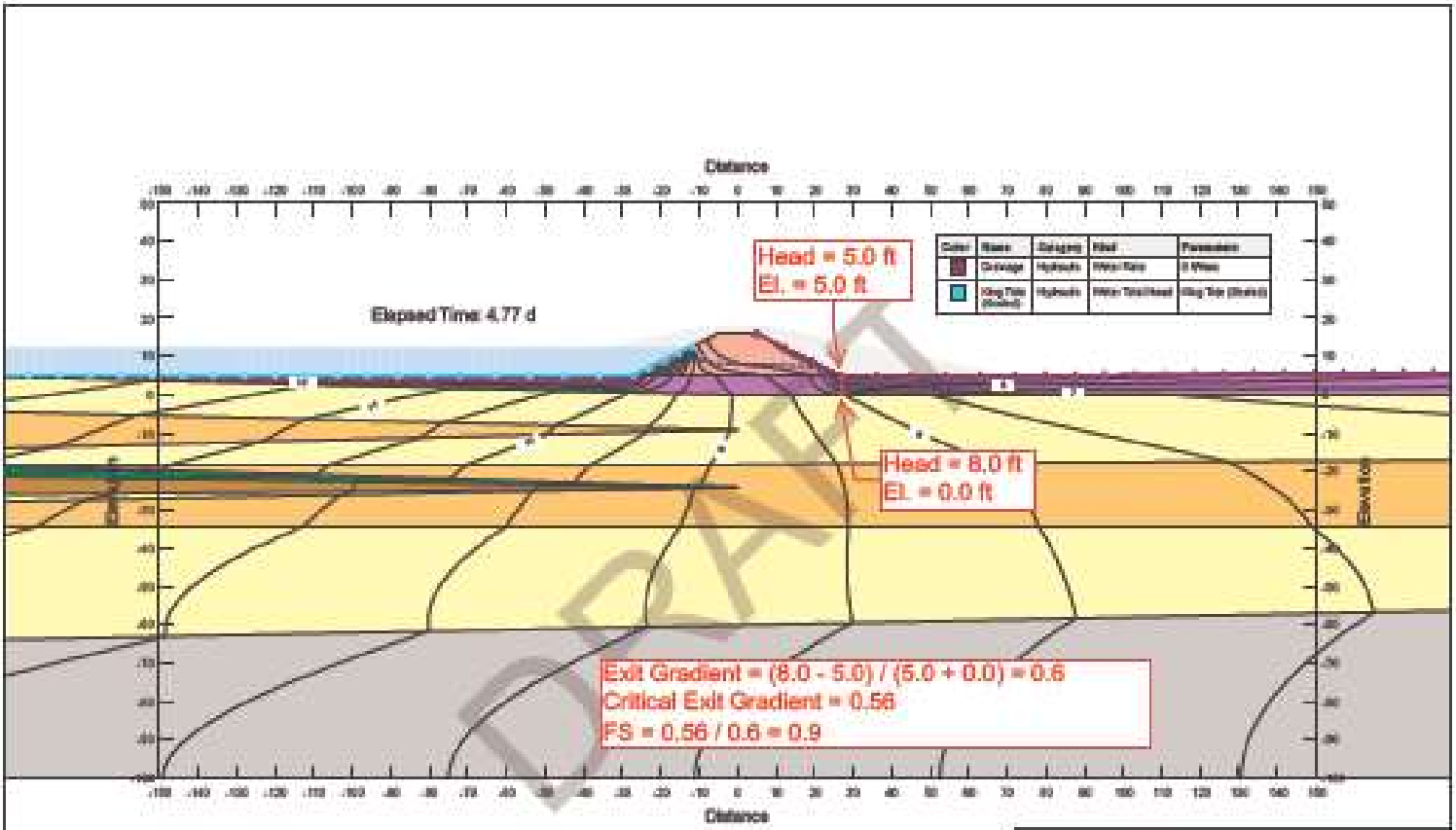


Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Stability Analysis (Sensitivity)
Steady Seepage with Geogrid
Marsh Deposits

April 2026 | 112880

SHANNON & WILSON | FIG. D14



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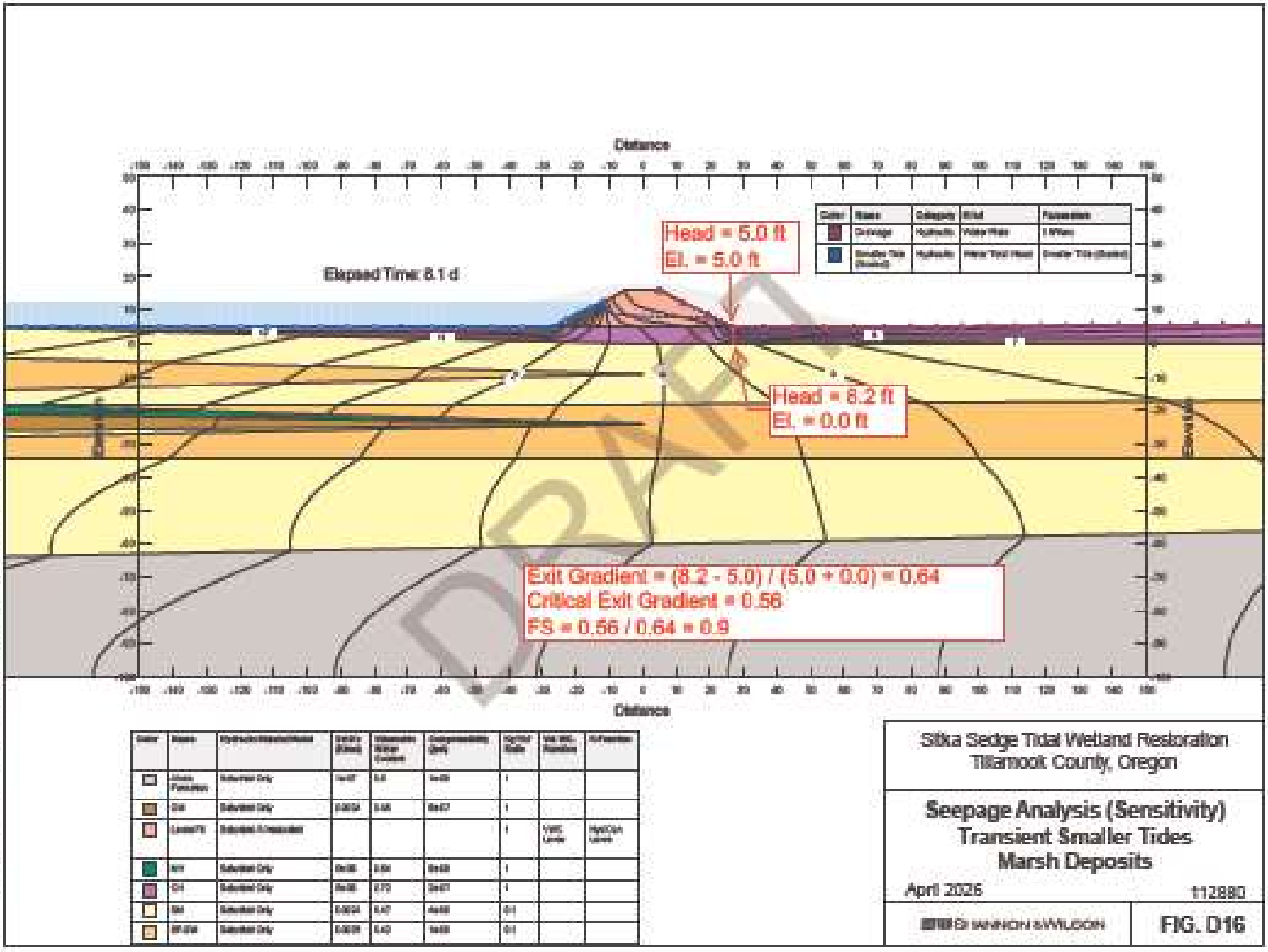
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Seepage Analysis (Sensitivity)
Transient King Tides
Marsh Deposits

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FIG. D15



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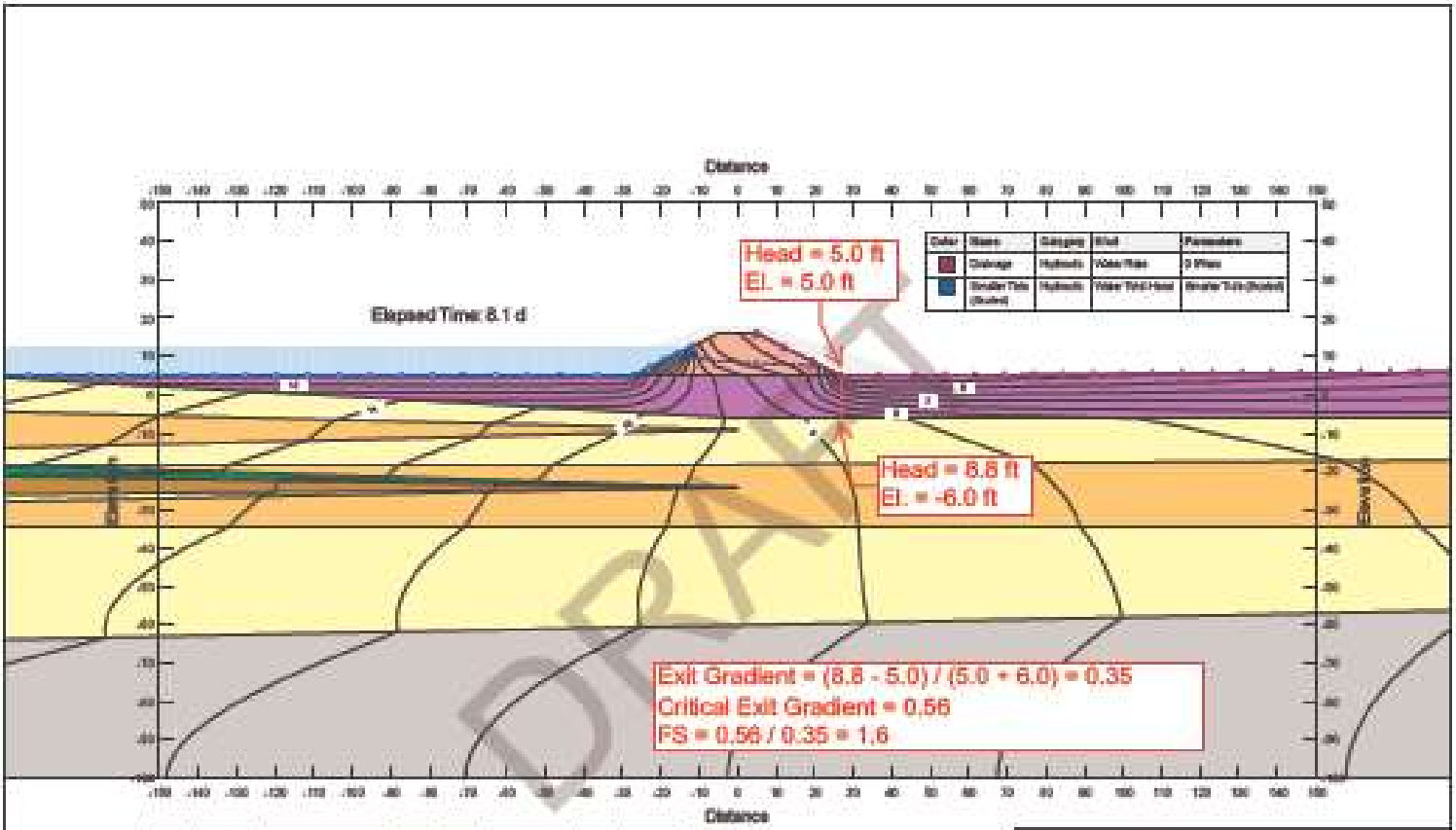
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Seepage Analysis (Sensitivity)
Transient Smaller Tides
Marsh Deposits

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BI SHANNON & WILCOXON

FIG. D16



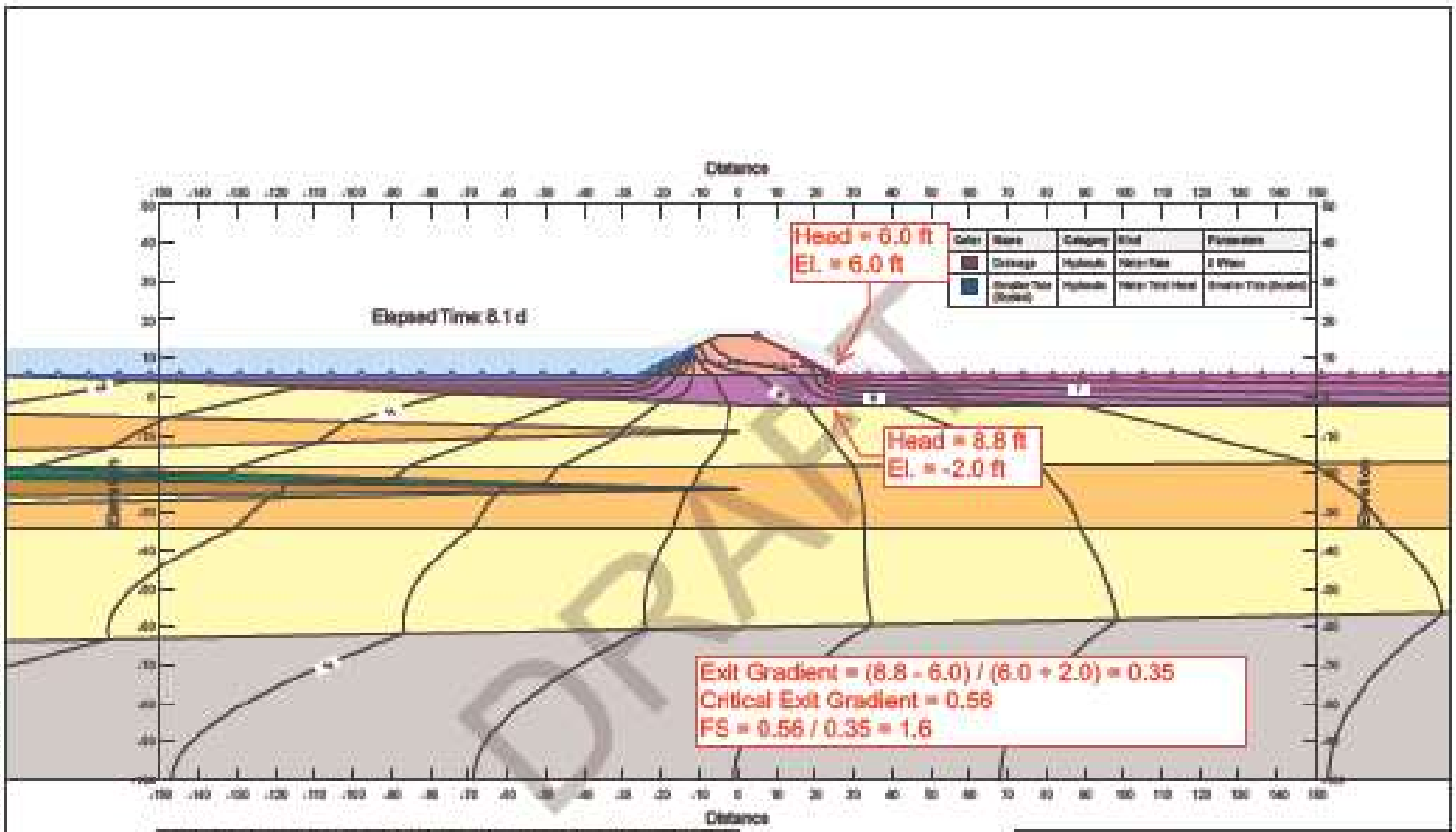
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Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Seepage Analysis (Sensitivity)
Transient Smaller Tides
Marsh Deposits, Ground Surface 5 ft

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Color	Name	Category	Kind	Formation
	Drainage	Hydraulic	Water Rate	2 Water
	Smaller Tides (Scaled)	Hydraulic	Water Total Head	Smaller Tides (Scaled)

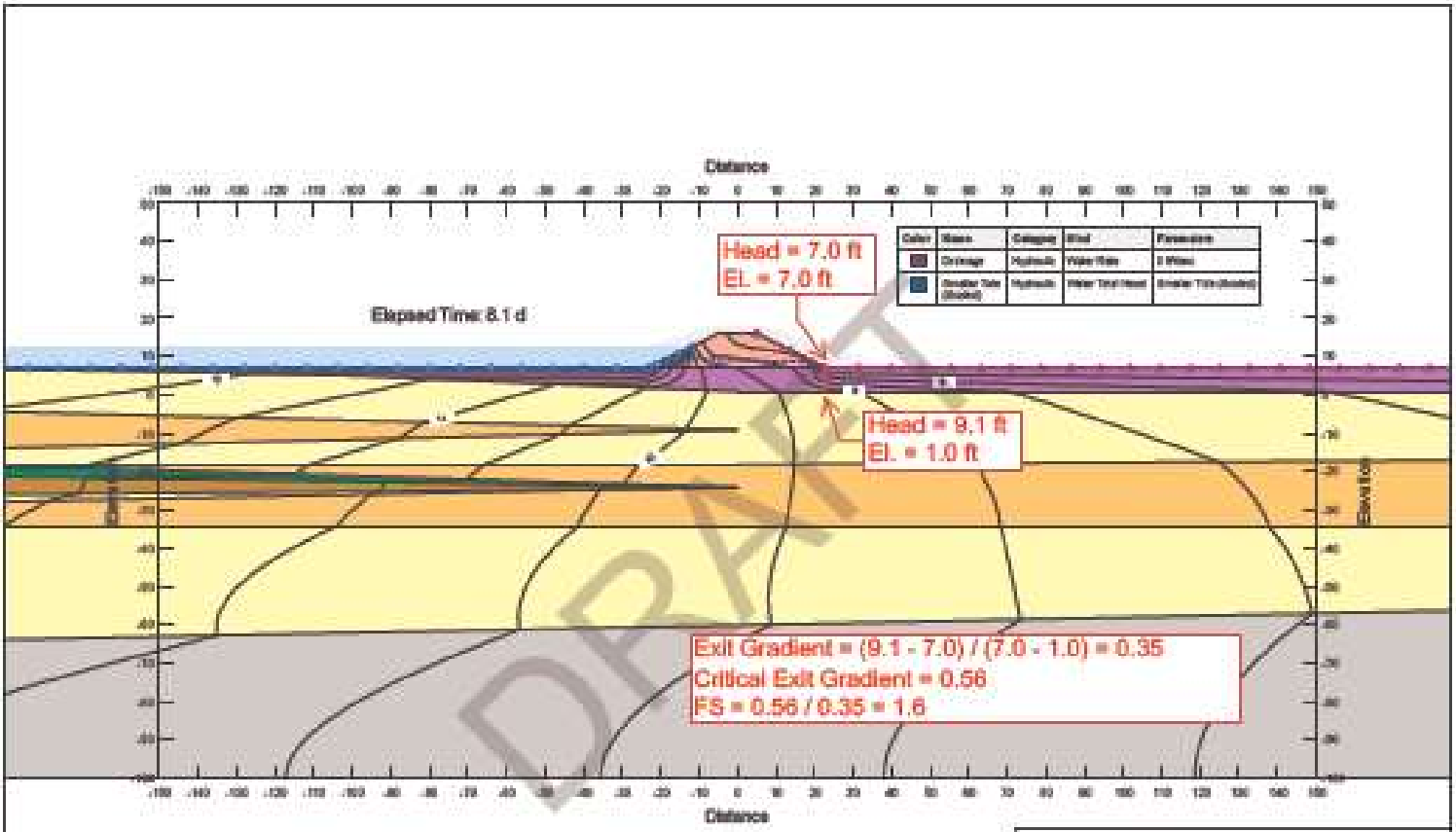
Color	Name	Hydraulic Material Model	Ksat,x (ft/day)	Ksat,y/Ksat,x Ratio	Saturated Water Content	Ky/Kx of Ratio	Ksat,90% Function	Ky Function
	Alsea Formation	Saturated Only	1e-07	0.5	1e-06	1		
	Qm	Saturated Only	0.0001	0.45	1e-03	1		
	Levee Fill	Saturated Unsaturated				1	USDA Default	Default Clone
	MH	Saturated Only	1e-03	0.01	1e-06	1		
	CH	Saturated Only	1e-05	0.13	2e-01	1		
	SW	Saturated Only	0.0001	0.01	4e-04	0.1		
	SP-SM	Saturated Only	0.0001	0.02	1e-04	0.1		

Silka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Seepage Analysis (Sensitivity)
Transient Smaller Tides
Marsh Deposits, Ground Surface 6 ft

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SHANNON & WILSON | FIG. D18



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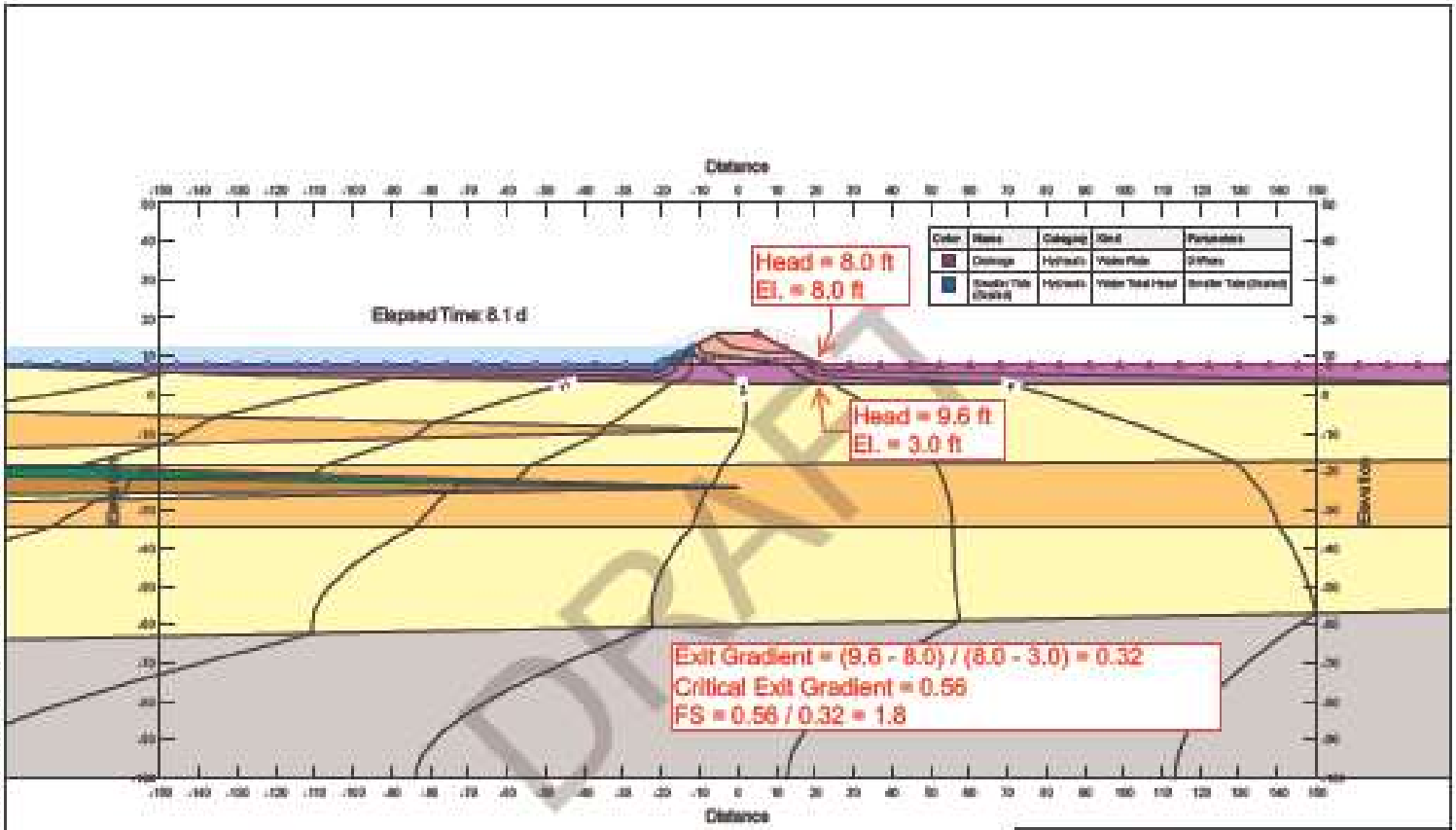
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Seepage Analysis (Sensitivity)
Transient Smaller Tides
Marsh Deposits, Ground Surface 7 ft

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FIG. D19



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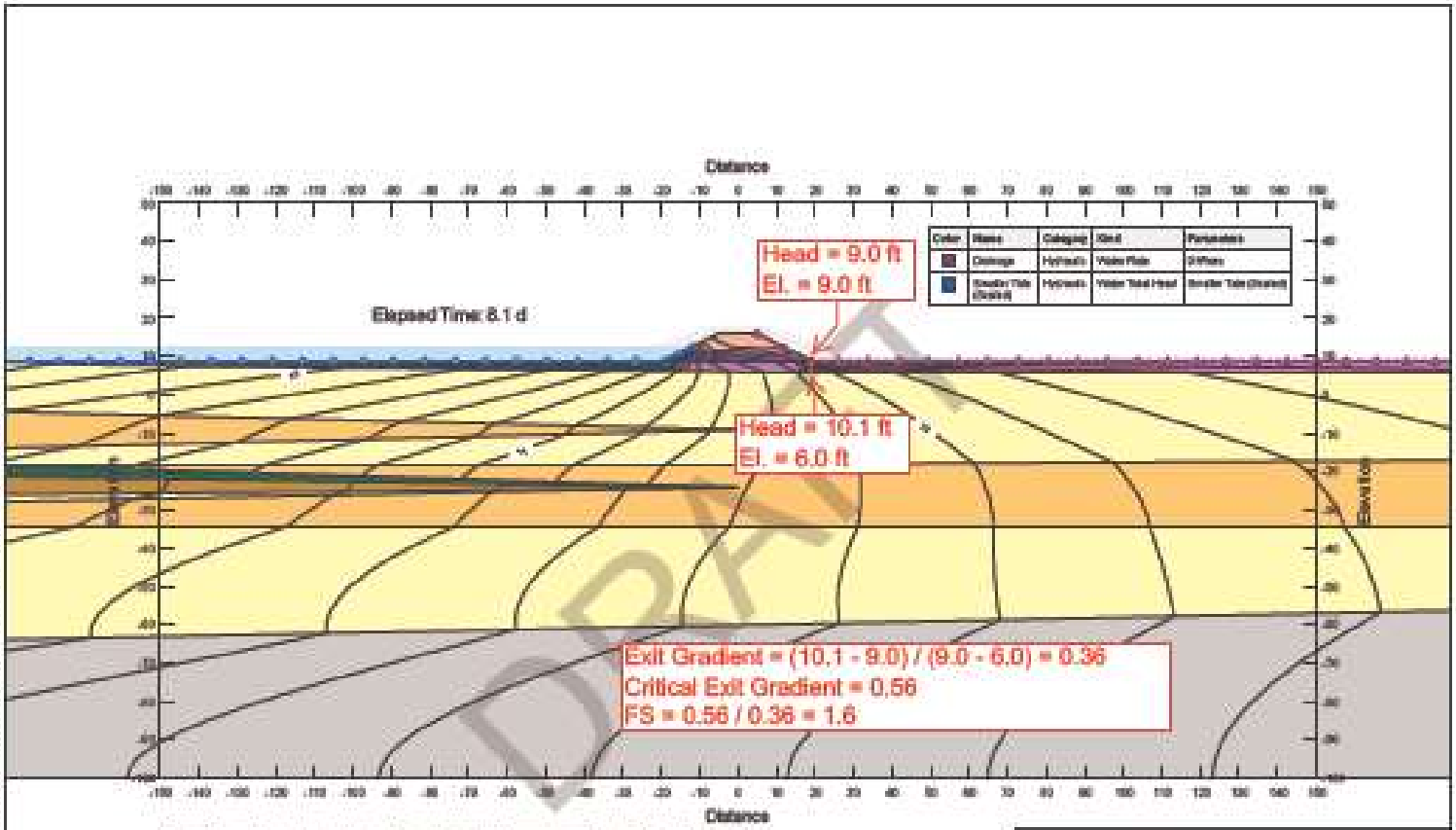
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Seepage Analysis (Sensitivity)
Transient Smaller Tides
Marsh Deposits, Ground Surface 8 ft

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FIG. D20



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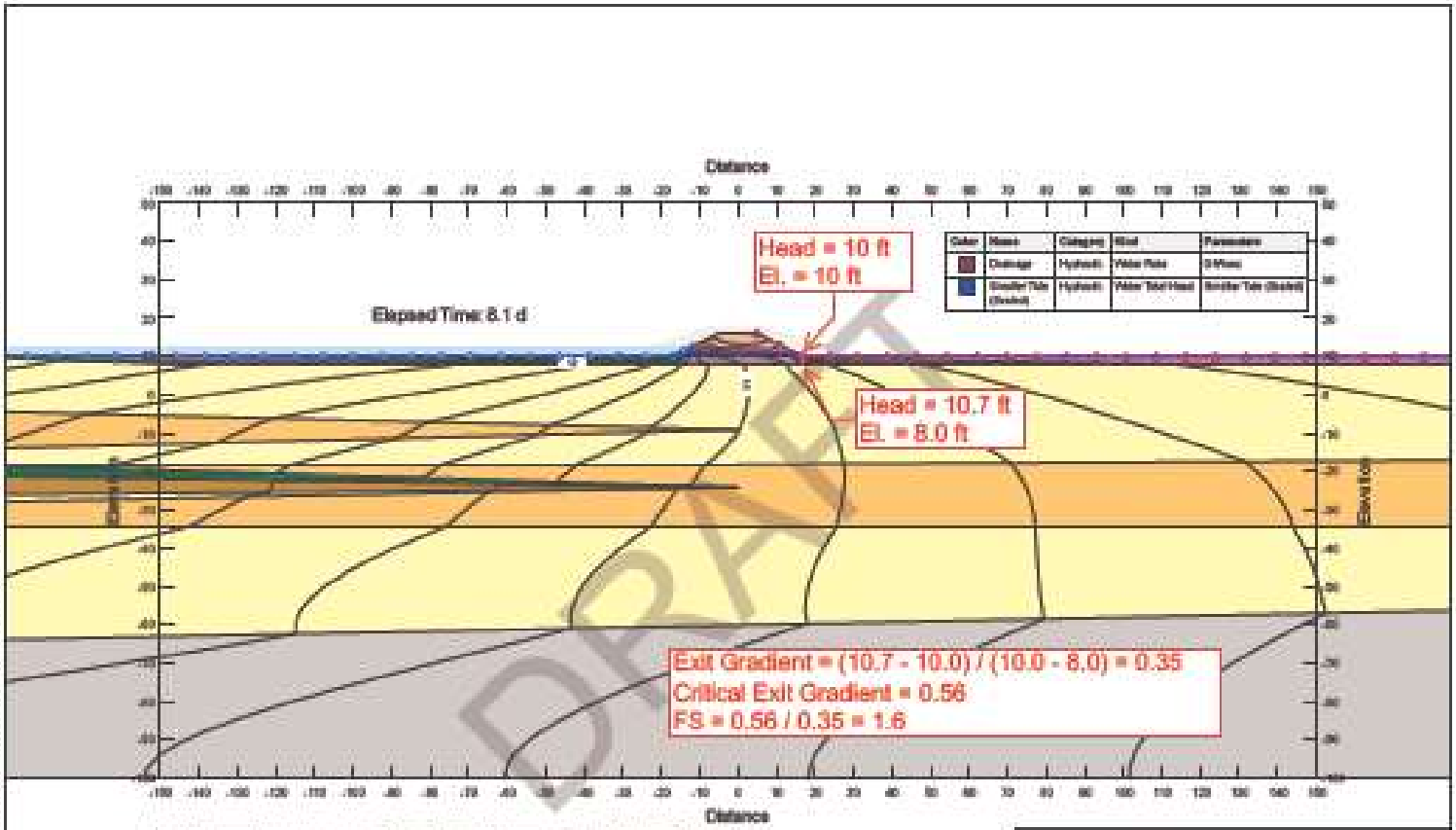
Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Seepage Analysis (Sensitivity)
Transient Smaller Tides
Marsh Deposits, Ground Surface 9 ft

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FIG. D21



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Sitka Sedge Tidal Wetland Restoration
Tillamook County, Oregon

Seepage Analysis (Sensitivity)
Transient Smaller Tides
Marsh Deposits, Ground Surface 10 ft

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SHANNON & WILSON | FIG. D22

Important Information

About Your Geotechnical/Environmental Report

IMPORTANT INFORMATION

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IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the Geoprofessional Business Association (https://www.geoprofessional.org)

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Appendix H.
Sitka Sedge Tidal Wetland Restoration Hydraulic Analysis Memorandum

Environmental Science Associates
819 SE Morrison Street
Suite 310
Portland, OR 97214
(503) 274-2010

esassoc.com

memorandum

date April 30, 2024

to Christer LeBrecque – Tillamook Estuaries Partnership

from Hunter White, P.E. and Elizabeth Schalo, P.E.

subject **Sitka Sedge Tidal Wetland – Hydraulic Analysis Memorandum**

1.0 Introduction and Overview

Oregon Parks and Recreation Department (ORPD) is considering a series of improvements and modifications to the infrastructure at Sitka Sedge State Natural Area (SSSNA) to restore tidal hydrology, aquatic organism passage, and habitat conditions in the area known as Beltz Marsh. In 2017-2019, OPRD contracted with consultants to perform a range of site assessments, develop preliminary restoration and infrastructure improvement alternatives, and conduct preliminary hydrologic analysis to select a preliminary preferred alternative including a dike breach to reconnect Beltz Marsh to Sand Lake, coupled with a setback dike near the south end of Beltz Marsh to maintain flood protection for the adjacent community of Tierra Del Mar and Sandlake Road (PGG, 2019). At the conclusion of the 2019 study, ESA and PGG recommended additional site assessment, detailed hydrodynamic modeling, and design analysis to advance the design concepts toward implementation.

In 2022, TEP, in partnership with OPRD, contracted ESA for the Sitka Sedge Tidal Wetland Restoration Alternatives Analysis and Preliminary Design project. The current project phase utilizes 2D hydrodynamic modeling to perform a more detailed hydraulic analysis of the proposed design elements, as well as further developing the details of conceptual design elements, to refine the preferred alternative and advance the preferred alternative to a preliminary 30%-level design.

The purpose of this memorandum is to summarize the hydrologic and hydraulic analyses conducted in support for the Sitka Sedge Tidal Wetland project. Primary goals of the Sitka Sedge Tidal Wetland project are to:

- Reconnect Beltz Marsh to tidal and estuarine processes in the adjacent Sand Lake Estuary,
- Improve fish passage conditions into Beltz Marsh and the three creeks (Reneke, Beltz, and No Name/East Marsh) to address limiting factors preventing recovery of ESA-listed Oregon Coast coho salmon and benefit other native fish species,

- Cause no negative impacts to neighboring properties in Tierra Del Mar (TDM) and, to the extent feasible, improve TDM's flood protection, stormwater management, and resilience to climate change and sea-level rise,
- Maintain or improve recreational access to the Sitka Sedge State Natural Area trail network.

Modeling was performed to evaluate in-depth design alternatives. The methodology for development of the hydrologic and hydraulic models is detailed within as well as analysis of model results and design alternatives.

At the outset of the project phase, ESA worked with TEP and Environmental Protection Agency (EPA) to develop a Modeling Quality Assurance Plan (QAPP). ESA conducted the hydraulic modeling and analysis in accordance with the QAPP. Minor deviations from the QAPP are described at the end of this memorandum.

2.0 Existing Conditions

Site Overview and Topography

In 2014, OPRD acquired property north of Pacific City, including and adjacent to portions of Sand Lake Estuary. The property established as Sitka Sedge State Natural Area (SSSNA) consists of approximately 87 acres of marsh, including approximately 42 acres of tidal marsh, sand, and mudflats and approximately 12 acres of estuary altered by the half-mile long Beltz Dike which runs east/west across the property.) Tierra Del Mar (TDM) is a residential community located immediately south of Beltz Marsh (**Figure 1**). Relative to surrounding areas, TDM is flat, with elevations ranging from around 18 to 23 feet NAVD88. TDM is triangular in shape, narrowing to the south. The project area is surrounded by the Pacific Ocean and associated foredune to the west, additional vegetated dunes and Beltz Marsh to the north, and forested bedrock hills to the east. A marshy swale referred to as No Name Creek or "East Marsh" occurs between TDM and the bedrock hills to the east (PGG, 2019).

SSSNA includes a parking lot and trailhead located adjacent to Sandlake Road immediately north of the Reneke Creek area. The trailhead provides recreational access to a network of hiking trails and beach access via the Beltz Dike Trail situated on top of the earthen levee that separates Beltz Marsh from Sand Lake.

Prior to diking, Beltz Marsh was formerly a shallow embayment fully connected to the Sand Lake Estuary via tidal channels and mudflats. Topographic modifications including construction of the Beltz dike, ditching in the marsh, construction of Sandlake Road, quarry activities, and development of the Tierra Del Mar community continue to affect site hydrology (PGG, 2019).

Figure 2 includes an aerial photo of the project area identifying the key site features. **Figure 3** depicts an annotated site topography map derived from OPRD DEM (various dates). Key site features include the Beltz Dike, box culvert and tide gate, remnant tidal channel and ditch network, primary beaver dam in the marsh, Sandlake Road prism, Tierra Del Mar community ditches, east marsh complex, and dunes to the west of the marsh. Areas within the marsh having elevations below 5.0 ft NAVD88 remain inundated most of the time, while areas with elevations above 8.0 ft NAVD88 are rarely inundated. The low point in Sandlake Road has an elevation of approximately 11.6 ft NAVD88. Residential areas within Tierra Del Mar (TDM) have ground elevations ranging from 14.0 to 22.0 ft, or higher. The TDM ditch and culvert network flows north and discharges into the marsh above the direct influence of the tide (PGG, 2019).

FEMA Flood Insurance Rate Maps from the Tillamook County Flood Insurance Study show the 100-year floodplain elevation in Beltz Marsh and the Sandlake Rd/East Marsh crossing of 11.8' NAVD88 (**Figure 4**). This flood elevation inundates the low point in Sandlake Rd and the low-lying portions of two private properties within TDM. **Figure 5** provides a summary reference of elevations of key site features and water levels in the project area.

Surface Water Hydrology Characterization

Tides

Tides are the primary driver of surface water level fluctuations in Beltz Marsh. Tidal fluctuations in the Pacific Ocean are transmitted to the marsh via the shallow Sand Lake Estuary and communicated through the existing box culvert and tide gate. The tide gate has deteriorated over time and appears to be completely missing in 2023, but the undersized box culvert restricts flow into the marsh and mutes the tidal signal resulting in a reduction in peak tidal water levels.

Pacific Ocean tides are influenced by both astronomical (sun & moon) and meteorological factors (storms). Storms and other large-scale atmospheric phenomenon, such as El Niño, tend to elevate the tides above predicted levels during the winter season. These same storm events are also responsible for delivering heavy rain to the area that frequently corresponds with high tide events. Measurements of ocean tides in the immediate vicinity of Sand Lake Estuary are not available. Long-term measurements at South Beach in Newport, OR, and Garibaldi, OR (within Tillamook Bay), provide representative ocean tides offshore of Sand Lake Estuary. As ocean tides enter the Sand Lake Estuary they are affected by the shallow nature of the estuary, constantly changing sand bed forms near the mouth of Sand Lake, and a complex tidal channel network, resulting in a situation where water levels in the estuary near the Beltz Marsh never fully recede to match low tides in the adjacent ocean. Daily low tides are severely muted and never fall below elevation 5.3 ft NAVD88, while ocean tides regularly fall to 0.0 ft NAVD88 or lower.

Fluctuations behind the dike in Beltz Marsh are primarily controlled by the interaction of tides in Sand Lake with the box culvert and tide gate, and secondarily by inflow from creeks and local drainages. Measurements show that under existing conditions, the missing tide gate and undersized box culvert allows a muted tidal flux to fill the marsh on rising tides. Water passes through the box culvert and water levels behind the dike rise more slowly and lag behind those in Sand Lake due to the limited flow through the structure. Tides in Sand Lake peak and begin to fall before the marsh completely fills and as water levels equilibrate on the ebb tide the flow reverses and water slowly drains back into Sand Lake until the minimum low water elevation of ~5.3 ft NAVD88 is reached. In 2017 when the tide gate was still partially intact, high tide elevations behind the dike were approximately 1.9 feet lower than Sand Lake, as shown in **Figure 6**. In 2023, since the tide gate is completely missing, high tide water levels behind the dike are approximately 1.4 feet lower than Sand Lake and approximately 0.5 feet higher than in 2017, as shown in **Figure 7**.

In the absence of significant freshwater input, low tides behind the dike tend to be similar to those in Sand Lake but are slightly elevated (a few inches) and of longer duration as the marsh slowly drains. Freshwater inputs from creeks and local drainages that arrive on a rising tide accumulate within the marsh and contribute to elevated water levels behind the dike. Following heavy rainfall events, it can take multiple tidal cycles for accumulated freshwater to be discharged through the existing box culvert. The leaky/missing tide gate is no longer functional,

though the size of the box culvert itself limits flux through the dike and thereby causes lower water levels than in Sand Lake at high tide, and higher water levels than in Sand Lake at low tide and following intense rainfall events.

Existing tidal fluctuations in the south end of the marsh are further impeded by the existing beaver dam that impounds water in the ditches and wetlands near Sandlake Road. Water levels remain relatively constant in time at about elevation 8.0 ft NAVD88 during the wet season and drop down to elevation 7.0 ft during summer low flow period. The beaver dam is approximately 3.0 ft high, based on observations by ESA staff in February 2018. **Figure 8** depicts typical water level fluctuations in the project vicinity during the summer (no rainfall), and **Figure 9** depicts typical water level conditions in the winter during and after a storm event (PGG, 2019).

Freshwater Inputs

Frequent and intense rainfall is common along the Oregon coast, and in the vicinity of the project site. Mean annual rainfall is about 80 inches per year, with a 2-year peak 24-hour rainfall intensity of about 3 inches (USGS, 2018). Other rainfall records are available at Tillamook and Cloverdale, OR. Periods of intense rainfall correspond to both elevated water levels in Beltz Marsh and photographs of flooding provided by the TDM community from 2016-2017 (pers. com). Given the size of Beltz Marsh, direct rainfall in the marsh may contribute meaningful surface water inflow during high-intensity storm events.

Beltz Creek (0.5 square mile drainage) and Reneke Creek (0.9 square mile drainage) are the primary freshwater sources that discharge into Beltz Marsh under existing conditions. Neither of the creeks has been gaged and few streamflow measurements are available to describe the response of the creek drainages to rainfall events. Therefore, the current understanding of creek hydrology relies on limited observations, comparisons with nearby gaged steams (Tucca Creek), and scaling to approximate discharge from Beltz and Reneke Creeks (PGG, 2019).

Reneke Creek flows west toward Sandlake Road, where it encounters a blocked culvert and then diverts southwest along Sandlake Road. The creek then flows into a partially blocked 38-ft long, 2-ft diameter, culvert that passes beneath Sandlake Road into Beltz Marsh. The culvert invert elevations are low enough to be subject to tidal fluctuations in the marsh during extreme high tides and ESA observed what appeared to be salmonid fish in the pool immediately downstream of the culvert. Upon entering the riparian area west of Sandlake Road, the creek channel meanders through wetlands and then bifurcates into multiple smaller braided channels that separately discharge into the marsh. Signs of erosion and road overtopping indicate that the existing culvert is undersized and does not pass all creek flows beneath the road. Historic photos and ground observations indicate that Reneke Creek was formerly routed through the blocked culvert and then to the north along what is currently a dry swale (PGG, 2019).

Beltz Creek arrives at Sandlake Road at a higher elevation than Reneke Creek and passes beneath Sandlake Road through an angled 80-ft long, approximately 3-ft diameter, culvert. The downstream end of the culvert is perched about 2.5 ft above the adjacent creek channel and well above the influence of the tide. The creek channel then flows through the former rock quarry and splits into at least two smaller channels before entering the marsh and ditch network.

The primary drainage ditch from TDM (identified as the East Ditch or Sandlake Road Ditch) flows north through a series of small culverts that parallel Sandlake Road. The final culvert beneath Roma Ave. consists of a 40-foot long 12-inch diameter corrugated smooth interior wall HDPE pipe with an outlet invert elevation of 11.8 ft

NAVD88. The ditch continues north and connects with other ditch networks upstream of the existing beaver dam within the marsh. Flows through the ditch along Sandlake Road are limited by the capacity of the culverts and condition of the ditch. A secondary drainage swale within TDM (identified as the North Ditch in **Figure 3**) runs parallel to the dune line and flows northeast through TDM, including beneath roads and structures, before turning in an east-northeast direction north of Roma Ave.

Surface water ponding is a common problem in TDM as evidenced by photographs provided by property owners to OPRD. ESA reviewed these photos and LiDAR topography and found that much of this ponding can be attributed in part to topographic depressions that lack positive drainage. Such ponding likely is exacerbated when the soil becomes saturated with groundwater following heavy rains. **Figure 10** illustrates ESA's conceptual understanding of frequently ponded areas based on a review of topographic depressions that correlate well with site photographs following intense rainfall events. Ponding depths may vary from a few inches to more than one foot, depending on local topographic conditions (PGG, 2019). During the current project phase, TDM's drainage system was analyzed in more detail to identify specific deficiencies and potential improvements, as described in Section 5.2.

Finally, drainage from the East Marsh located on the east side of Sandlake Road and across from TDM, flows into the south end of Beltz Marsh via a 2-ft diameter culvert beneath Sandlake Road. The East Marsh watershed is approximately 202 acres or 0.32 square miles. Limited measurements in this marsh by OPRD in 2018 indicate that water levels in the immediate vicinity of Sandlake Road are controlled by the backwater created by the beaver dam in Beltz Marsh. Ground levels and surface water levels in the East Marsh continue to rise to the south, likely impounded by relic beaver dams and/or emergent vegetation. Observations by ORPD in spring of 2018 indicated water ponded approximately 1 ft above the ground surface throughout this marsh, with deeper water closer to the Sandlake Road culvert (PGG, 2019).

Groundwater discharge and upwelling likely also contribute surface water inputs to the marsh, particularly during the wet season. However, PGG indicated that groundwater discharge flows are likely quite small compared to peak runoff flows during extreme storm events (PGG, 2019).

Hydrologic Characterization Summary

- The Beltz dike tide gate no longer functions properly and allows water to flow into the marsh when water levels are higher in Sand Lake. Still, water levels at high tide are lower on the inside of the dike under existing conditions due to limited flux allowed in and out of the existing box culvert.
- The existing box culvert/tide gate is undersized to efficiently drain accumulated freshwater within the marsh following intense rainfall events and thus it currently takes multiple low tide cycles for water levels in Beltz Marsh to equilibrate with Sand Lake.
- The existing beaver dam at the south end of Beltz Marsh impounds water and creates a backwater area that causes water to pond along both sides of Sandlake Road and within the East Marsh.
- Reneke Creek's northern culvert is completely blocked by gravel, and the southern culvert is undersized for passing peak stream flows. This constriction moderates Reneke Creek flows through the culvert at moderate flows and causes overtopping of the roadway during peak streamflow events.

- The Beltz Creek culvert is perched above the downstream channel and is well above the influence of the tide.
- The upland drainage network in TDM has limited capability to drain TDM following intense rainfall events due to a combination of local topographic depressions with a lack of positive drainage, absence of a drainage network along residential roads, and limited capacity of the culverts along Sandlake Road to transmit flows.
- The East Marsh is affected by the backwater of the beaver dam in Beltz Marsh and is subject to tidal fluctuations during extreme high tides and storm events. Ponded water has been observed in the East Marsh well above the influence of the backwater from Beltz Marsh.
- **Figure 11** shows and describes the major hydraulic control features affecting surface water movement into, out of, and within Beltz Marsh, including the Beltz Dike, culvert and tide gate, and the main beaver dam in the marsh's interior.
- **Figure 5** summarizes elevations of key site features and water levels
- Groundwater hydrology characterization and alternatives modeling are covered in detail in the ***Sitka Sedge State Natural Area Hydrologic Analysis Final Report*** (PGG, 2019) Groundwater modeling of a dike breach scenario without a setback dike predicted insignificant changes in shallow groundwater elevations beneath TDM during average conditions and slightly reduced groundwater elevations during large storm events.

3.0 Hydrologic and Hydraulic Analysis

To evaluate the site's existing condition and inform and evaluate alternative design concepts, ESA developed and utilized a two-dimensional (2D) HEC-RAS hydraulic model. 2D hydraulic models use a depth-averaged approach to represent flow patterns in the horizontal plane, producing maps of depth and velocity across the simulated landscape. HEC-RAS 2D was used for this project to represent the complex, multi-directional flows within Sand Lake, Beltz Marsh, flood-prone areas, and local infrastructure.

A series of hydrologic scenarios were developed for the site to calibrate the model and analyze design alternatives. Flooding is a concern for TDM and as such, project performance under flood conditions was a priority to analyze. Streamflow inputs for the TDM drainage areas were developed utilizing the Santa Barbara Urban Hydrograph Method. Streamflow inputs for creek watersheds were developed utilizing USGS StreamStats, verified by a watershed scaling of the nearest gaged stream comparable to the project creek. The 2D hydraulic model requires topography, hydrology, and roughness as inputs for the model, which are represented within the model mesh. Alternative design concepts were assessed with model depth, water level, and velocity outputs.

Hydrologic Inputs for 2D Model

There are five hydrologic inputs into the Sitka Sedge marsh:

1. Reneke Creek Watershed

2. Beltz Creek Watershed
3. East Marsh Watershed
4. Tierra Del Mar stormwater subbasin 1 (Sandlake Rd Ditch Subbasin)
5. Tierra Del Mar stormwater subbasin 2 (TDM Northwest Ditch Subbasin)

Reneke Creek, Beltz Creek, and East Marsh watersheds do not have flow gages. A nearby gaged watershed, Tucca Creek watershed, was used to develop inflow hydrographs for these sources. The hydrographs for each watershed were scaled from the Tucca Creek hydrograph using watershed drainage area. (USGS, 2023)

Tierra Del Mar is a residential community south of the marsh. Its landcover is developed and consists mainly of single-family homes, commercial businesses, and roads. A total inflow hydrograph from this area was developed based on direct rainfall and a total surface area of 190,000 square meters. Rainfall data was filtered with backward-facing 2-day average to simulate the delay between instantaneous and eventual rainfall. The drainage area was then delineated into two subbasins, each draining into the marsh through a small culvert. An inflow hydrograph was scaled from the total hydrograph for each subbasin based on subbasin area. Inflow hydrographs for the February 2017 winter storm used in calibration are shown in **Figure 12**.

Two large statistical storm events were also developed as hydrologic scenarios: the 100-year and 50-year events. The 100-year and 50-year inflow hydrographs for Reneke, Beltz, and East Marsh watersheds were scaled from 100-year and 50-year hydrographs of Tucca Creek. The 100-year and 50-year hydrographs of Tucca Creek were developed by using a flood frequency analysis to find peak flows, following the procedures of Bulletin 17B. (Interagency Advisory Committee on Water Data 1982) The February 2017 Tucca Creek hydrograph was scaled to peak at the 100-year and 50-year peak flows, and the Reneke, Beltz, and East Marsh watershed hydrographs were scaled from that based on drainage area.

The 100-year and 50-year inflow hydrographs for the Tierra Del Mar subbasins were developed using the Santa Barbra Urban Hydrograph method and splicing in its 24-hour hydrograph into the February 2017 hydrograph. Inflow hydrographs for the 100-year and 50-year events are shown in **Figures 13 and 14**, respectively.

2D Hydraulic Model Development

Topography and Model Terrain Development

Topography for the hydraulic model was generated from the following sources in order of priority:

1. SWLS 1 foot. This surface was developed following a survey conducted on January 27, 2023, to support the project. It includes small areas around culvert structures. This survey also surveyed the culvert materials, shapes, and invert elevations. (SWLS, 2023)
2. PhoDAR. This DEM is a combination of a photogrammetrically derived model and LiDAR data with manual adjustments. The aerial photography used in the photogrammetrically derived model was taken on September 18, 2015. The LiDAR is dated to 2009. Manual adjustments were made in some areas of

dense vegetation. However, the PhoDAR includes some artificially elevated artifacts outside of the vegetation zone adjustments. (OPRD, 2017).

Additional terrain edits were performed in HEC-RAS to enforce low ground in channels as well as high ground elevations along roadways, the Beltz Dike, and alternative setback dikes.

Model Boundary Conditions

HEC-RAS requires flow as input at the upstream model boundaries and river stage at the downstream boundary. Streamflow estimates were developed as detailed in section 3.1. The downstream boundary of the model is located in Sand Lake approximately 2,000 feet downstream of Beltz Dike. This downstream boundary condition is tidal. The tides data used was water level data measured outside of the Beltz Dike February 1-13, 2017. This data is appropriate for use for all three hydrologic scenarios (Calibration, 100-year, and 50-year) because of the high tides (11.9 ft NAVD and 11.3 ft NAVD) included in the dataset that are concurrent with the hydrograph peak. Tidal water level data used is shown in **Figure 15**.

Model Layout

The full model extent, along with hydrologic inflow and boundary conditions locations, are shown in **Figure 16**. For 2D hydraulic models, the model domain is divided into a mesh composed of cells. In HEC-RAS, the water surface elevation is calculated at each model cell, and velocity is calculated at the faces between cells. As such, cell size and cell face orientation are important to consider to accurately represent flow patterns within channels and the floodplain. Break lines were added to the mesh to ensure that roads, berms, Beltz Dike, channels, and swales were properly captured within the model mesh. Additionally, the mesh resolution was assigned to 20 ft inside the Beltz Dike to appropriately capture flows within channels and across the floodplain. Existing culvert structures within the model domain were included using culvert materials, shapes, and invert elevations as surveyed by Statewide Land Surveying, Inc. (SWLS, 2023)

Roughness

HEC-RAS requires an estimation of the roughness of the landscape as characterized by the parameter Manning's *n*. For the 2D hydraulic model, the National Land Cover Database (NLCD) 2019 Land Cover was used (**Table 1**). (MRLC, 2021)

Table 1. Roughness Categories and Final Manning's *n* Values

Type	Manning's *n*
Open Water	0.035
Barren Land Rock-Sand-Clay	0.03
Evergreen Forest	0.15
Developed, Open Space	0.035
Woody Wetlands	0.07
Developed, Low Intensity	0.08
Grassland-Herbaceous	0.04
Developed, Medium Intensity	0.12
Shrub-Scrub	0.05

Mixed Forest	0.12
Deciduous Forest	0.1
Emergent Herbaceous Wetlands	0.045
Developed, High Intensity	0.15
Pasture-Hay	0.045

Model Calibration

Model calibration requires on-site data to be compared against modeled outputs. Water levels are not regularly monitored in Sitka Sedge State Natural Area. Therefore, water level loggers were installed to record site data inside and outside of the Beltz Dike at the existing culvert. Water level was recorded from November 2016 to June 2017 and again from January 2023 to March 2023. During these periods, a representative winter storm that caused flooding occurred between February 1-13, 2017. This storm event was used to calibrate the existing conditions 2D model.

4.0 Alternatives Analysis

Alternatives Development

ESA developed a series of design alternatives for restoring tidal connectivity and fish passage between Sand Lake and Beltz Marsh, while maintaining or improving drainage and flood protection for Tierra Del Mar and Sandlake Road and maintaining recreational access to the trail network at SSSNA.

Hydraulic analysis of the design alternatives and variations focused on the following key project elements and considerations:

Dike Breach -

- the size, location, and number of dike breaches through Beltz Dike
- how different dike breach sizing or full dike removal would affect water levels, velocities, shear stresses, and circulation patterns in Beltz Marsh and Tierra Del Mar

Setback Dike -

- the optimum location of a setback dike for flood protection and storm drainage function balancing storage capacity with wetland impacts
- potential stormwater drainage improvements in TDM that would reduce flooding experienced by residents or improve the functionality of a setback dike

TDM Stormwater Drainage System Assessment

In order to understand how the SSTW project will affect the function of TDM's stormwater drainage system, specifically how a setback dike in a given location would perform, or how high tide levels could affect TDM's drainage, it is important to characterize the stormwater drainage network and facilities within TDM and the East

Marsh watershed. Most of the land in TDM sits above maximum tidal or flood levels of Sand Lake, but the flat terrain and depressional areas within TDM hold water during storm events due to a lack of positive drainage pathways for water to escape. While the ditch along Sandlake Road and an existing swale at the northwest end of TDM have some ability to convey water downstream to Beltz Marsh, many areas, particularly in the western half of TDM, have no drainage connection to these conveyance features. Furthermore, culverts and ditches along Sandlake Road have limited conveyance capacity and regularly clog with sediment and debris. These conditions lead to real drainage problems and exacerbate residents' perception of flooding in TDM and their concern that reconnection of Beltz Marsh could make these conditions worse. While reconnection of Beltz Marsh could influence drainage and flooding conditions in the lower portion of the TDM drainage system, much of the flooding experienced in TDM further away from the marsh is unrelated to conditions in the marsh and would not be affected by the proposed restoration actions.

This assessment of TDM's drainage system is aimed at:

- defining drainage patterns within TDM and estimating stormwater runoff rates and volumes that contribute to the East Marsh and Beltz Marsh for incorporation into the 2D HEC-RAS modeling,
- identifying the deficiencies within the system that cause or contribute to flooding and ponding of water within TDM,
- differentiating between flooding caused by stormwater drainage issues and flooding that could be influenced by restoration actions at SSSNA, and
- identifying potential drainage improvements in TDM that could alleviate flooding conditions experienced by residents or improve the functionality of the proposed setback dike system.

ESA used a combination of LiDAR and survey data, site reconnaissance and interviews with TDM residents, and winter-time storm observations and water level data to define drainage patterns/flow routing, sub-catchment areas, stormwater discharge points/connections to SSTW, and system deficiencies that could be improved. **Figures 17 and 18** show an overview of the existing TDM Drainage System with aerial photo and LiDAR backgrounds, respectively.

ESA's full assessment of TDM's drainage system, identified deficiencies, and potential improvements are described in detail in a separate memorandum, **Tierra Del Mar Stormwater Drainage System Assessment by ESA, May 2023**.

TDM Drainage System Interaction with Setback Dike

During large storm events, runoff from TDM and the East Marsh watershed would drain into the storage basin behind the setback dike. During high tides when the water level in Sand Lake and the reconnected portion of Beltz Marsh is elevated, the setback dike's tide gate would be closed, and runoff would accumulate to fill the storage basin and cause levels behind the dike would rise. When the tide recedes, or the inboard water level exceeds the outboard water level, the tide gate would open and allow the storage basin to drain out.

For analysis of the setback dike alternatives described in Section 5.4, it is important to understand the runoff and streamflow contributions that enter the south end of Beltz Marsh in the area that would be upstream of each

potential setback dike alternative alignment. Depending on the chosen location for the setback dike and potential re-routing of stormwater, some of the runoff could be routed to flow outside of the setback dike and therefore not occupy storage volume behind the setback dike.

ESA identified three separate drainage basins within TDM including:

- East Marsh/No Name Creek watershed – approximately 202 acres (0.315 square miles) including portions of TDM that lie east of Sandlake Road and the forested hillslopes east of TDM
- Sandlake Road Ditch subbasin – approximately 30 acres including most of the residential area of TDM that lies west of Sandlake Road. These areas generally drain toward the Sandlake Road ditch where water is conveyed through a series of culverts at each side street intersection.
- TDM Northwest Ditch subbasin – approximately 12 acres including a smaller portion of the TDM residential area situated between the Pacific Ocean foredune and a topographic rise that separates this area from the Sandlake Road Ditch subbasin to the east. This area drains northward through swales that have been partially blocked by infill development, becoming a channel that flows through culverts at Jasmine Ave, Pollock Ave, and Roma Ave and toward Beltz Marsh.

A setback dike located near the existing beaver dam would be downstream of all TDM and East Marsh runoff contribution points, and all runoff and streamflow from these areas would drain into the marsh upstream of the dike. Therefore, a setback dike in this location would need to include sufficient storage volume to contain all runoff from TDM and the East Marsh subbasin.

It could be possible to orient a new setback dike alignment such that the TDM Northwest Ditch would drain directly into Beltz Marsh downstream or outside of the setback dike. Furthermore, it could be possible to intercept flows from the TDM Northwest Ditch subbasin and reroute these flows to a new outfall on the beach. These actions could reduce the amount of runoff draining into the storage basin upstream of the proposed setback dike and could reduce the amount of storage capacity needed. These modifications were considered and analyzed as described in Section 5.4.3. However, the TDM Northwest Ditch subbasin contributes three times less runoff to Beltz Marsh than the Sandlake Road Ditch subbasin, and the East Marsh watershed is five times larger than both subbasins combined. Hydraulic modeling determined that re-routing flows from the Northwest Ditch subbasin to the outside of the setback dike would not substantially influence the performance of a setback dike or required storage volume. Capturing runoff within the TDM Northwest Ditch Subbasin and the west end of Austin Ave, and rerouting flows to a new outfall on the beach, could improve localized flooding in the subbasin by providing a more efficient way to drain water out of these flat depressional areas.

Dike Breach

Size, Location, and Number of Dike Breaches

During the 2019 hydrologic study, ESA estimated the minimum dike breach dimensions that would allow for full equilibration of tidal water levels between Sand Lake and Beltz Marsh based on a combination of Tidal Channel Design Guidelines (PWA, 1995), hydraulic geometry (Williams, 2002), and analysis of nearby reference tidal channels. Using the parameters of contributing marsh surface area and tidal prism volume (the potential volume

of water contained within the marsh between mean higher high water and mean lower low water datum elevations), ESA estimated the dike breach tidal channel should have a minimum bottom width of 37 feet, with a top width (at the top of the dike breach) of approximately 80 feet. However, a larger dike breach opening may be beneficial for:

- restoring more natural circulation patterns and sediment transport/accretion processes,
- reducing water velocities through the dike breach to reduce erosion and facilitate fish passage, and
- reducing shear stress on the banks of the dike breach channel to minimize potential for erosion at potential pedestrian bridge abutments.

Full removal of the earthen levee known as Beltz Dike would restore tidal connectivity closest to the conditions that would have existed prior to the construction of Beltz Dike allowing for unconstrained flow of tidal water across the marsh surface. However, if the dike were fully removed, the Beltz Dike Trail which provides access from the trailhead to the rest of the SSSNA trail network would be severed. An alternate trail connection could be provided by constructing a boardwalk across the marsh or by establishing a new trail around the perimeter of Beltz Marsh and across the south end of the marsh. These options for maintaining recreational access are discussed herein. TDM residents have also raised concerns about removing the full dike.

Number of Dike Breaches

ESA considered whether it would be beneficial to construct multiple dike breaches, or if it would be better to focus on a single dike breach. Multiple breaches could provide additional fish passage pathways between Sand Lake and Beltz Marsh. However, including multiple breaches would have several disadvantages including:

- requiring additional pedestrian bridges or culverts to maintain trail access,
- splitting tidal flow and tidal prism volume between multiple breaches/inlets, rather than focusing tidal prism volume through a single breach to help maintain the channel width and depth in the future,
- reducing predictability due to splitting of flow through multiple breaches that could evolve/enlarge over time and cause damage to bridges or culverts. For example, if one large breach and one small breach were constructed, it is possible that the larger breach could be blocked by sediment or other means, causing the smaller breach to enlarge and undermine bridge abutments. All breaches and associated access infrastructure would need to be sized to handle all flow in case it became the preferential flow path.

Additionally, natural tidal channel networks in similar settings along the Oregon Coast typically exhibit a fractal dendritic planform resembling a tree truck that branches into many smaller channels. **Figure 19** shows an example of a dendritic tidal channel network where multiple small first-order channels combine to create a 2nd order channel, 2nd order channels combine to create a 3rd order channel, and 3rd order channels combine to create a 4th order channel, all funneling tidal flows to a single inlet/outlet point (Hughes, 2012). This dendritic channel network planform can be observed in the northeastern portion of the Sand Lake Estuary (**Figure 20**), and in many other locations along the Oregon Coast and low-gradient marshes across the world.

For these reasons, we recommend focusing on constructing a single dike breach that is generously sized large enough for flood conveyance capacity, fish passage, and to accommodate some channel adjustment in the future.

Location of Dike Breach

To determine the ideal location for a dike breach, ESA analyzed historical aerial photos along with current-day aerial photos and onsite observations of existing flow paths and relict channel features. In the oldest available historical aerial photo from 1939 shown in **Figure 21** (University of Oregon Libraries, 2020), two large channels appear near the midpoint of Beltz Dike, and it appears that the drainage structure was also near this location at the time. In the historical aerial, as well as recent aerial photos (**Figure 22**), it appears that these large central channels upstream of the dike generally align with the largest channel features downstream of the dike in Sand Lake. This central area appears to be the most logical location to construct a new dike breach channel to restore historical flow patterns and connect the prominent channel features observed upstream and downstream of the dike.

Two other potential locations were considered for a dike breach. A breach could be excavated near the current location of the existing concrete box culvert and tide gate, near the eastern end of the dike. However, while the existing culvert drains out into Sand Lake and into downstream channel forms, the upstream channel network within Beltz Marsh does not align well with this location. The existing box culvert has large scour holes upstream and downstream of the culvert. The upstream scour may be partially caused by the location of the culvert relative to the upstream channel, which causes water from Beltz Marsh to funnel into this area and form a large eddy. Allowing for more tidal exchange in this location could continue causing erosion of the adjacent banks and dike embankment. At the west end of Beltz Dike, a dike breach could be excavated to connect the northwestern lobe of Beltz Marsh to a small tidal subbasin observed outside of the dike. The 1939 aerial does not show any evidence of a historical channel connection in this location, and the small tidal subbasin observed today appears to have formed within the borrow ditch excavated outside of the dike.

These four potential dike breach locations considered are identified in **Figure 23**.

We recommend locating a new single dike breach near the midpoint of Beltz Dike to restore connectivity between the large channel features visible in historical and current aerial photos inboard and outboard of Beltz Dike.

Sizing a Dike Breach

ESA utilized the 2D model to analyze dike breach and removal alternatives including:

- 100-Foot Breach - A single dike breach with a 40-foot bottom width and 100-foot top width
- 200-Foot Breach – A single dike breach with a 200-foot top width allowing for a wider marshplain bench through the opening
- Full Dike Removal – Full removal of the entire Beltz Dike lowered to match the existing adjacent marsh plain elevations with a breach channel

Factors for evaluating the performance, benefits, and disadvantages of each dike breach or removal option include:

- Changes or differences in water levels in Beltz Marsh or behind a setback dike
- Circulation patterns of water moving between Sand Lake and Beltz Marsh, across the marshplain, and within channels
- Velocities and shear stresses through the dike breach openings, across the marsh, and within channels to inform fish passage, erosion potential, and sedimentation/accretion responses

Water Levels

Because the 100-foot breach is large enough to allow for full equilibration of water levels inside the marsh, the model showed no difference in water levels generated inside the marsh under the dike breach scenarios compared to the full dike removal alternative. The model also showed negligible differences in water levels behind the setback dike between the dike breach or full dike removal alternatives. **Figure 24** shows a comparison of modeled water levels behind a setback dike under a 100-foot dike breach or full dike removal compared to the existing dike configuration and water levels in Sand Lake.

Circulation Patterns

While there was no discernable difference in water levels modeled under the dike breach and full dike removal scenarios, there are differences in the circulation patterns of how water moves in, out, and across the marsh surface.

Under the dike breach scenarios, water moving between Sand Lake and Beltz Marsh is funneled to and concentrated through the dike breach opening. Low velocities are present on the marshplain, with relatively higher velocities in the concentrated flows within the channels. With full dike removal, tidal flux is distributed more evenly across the marsh and interface between Sand Lake and the marsh, particularly at higher water levels. During lower water levels under the full dike removal alternative, water is still generally funneled into the channel and the proposed dike breach location, concentrating flows with higher velocities observed in the channels.

100-foot dike breach compared to full dike removal – tidal circulation during high tide drain out: **Figure 25** shows a comparison of water circulation patterns and flow directions under the 100-foot dike breach and full dike removal alternatives during drain out while water levels are still high (above 9' NAVD88). **Figure 26** shows the same comparison after water levels have fallen below 9' NAVD88 and flows are concentrated in channels under both conditions.

Under the 100-foot dike breach condition, an eddy is visible in **Figure 25** just inside of the marsh near the dike breach. This eddy was not observed in modeling under the 200-foot dike breach condition. An eddy has potential to cause erosion on the banks of the dike breach if shear stresses are high enough to erode the soils and could require scour countermeasures such as armoring with rip rap or other means.

Velocity/Shear Stress

The 100-foot breach results in water velocities through the breach that are less than 2 feet per second (ft/sec) for most of the time (ODFW's threshold for fish passage peak velocities in fishway transport channels). Short-

duration peak velocities of approximately 3 ft/sec occur during peak drain out during the 100-year storm scenario. **Figure 27** shows modeled water velocities through the 100-foot breach throughout the 100-year storm hydrologic scenario compared to the 2 ft/sec fish passage velocity threshold. During the moments of peak velocity, the peak shear stress through the dike breach reaches approximately 0.19 pounds per square foot (lb/sf), which has the potential to erode medium to coarse gravels and may require light armoring to prevent erosion in the vicinity of potential bridge abutments. **Figure 28** shows a 2D model output of peak shear stress through the 100-foot dike breach compared to a table for limiting shear stress for uniform noncohesive sediments (Fischenich, 2011).

The 200-foot dike breach would result in even lower velocities through the dike breach opening with peak velocities across the breach at approximately 1 ft/sec. Slightly higher velocities occur in tidal channels near the breach. **Figure 29** shows a comparison of peak velocity through 100-foot and 200-foot dike breaches during the 100-year storm hydrologic scenario. The lower peak velocities and shear stresses through a 200-foot dike breach would likely not require any armoring to protect potential bridge abutments. For comparison, the nearby Whalen Island road bridge has an upstream contributing marsh area of 180 acres, more than twice that of the Beltz Marsh. The Whalen Island Bridge has an 80-foot span and channel opening, but the velocities and shear stresses through that bridge section require extensive rip rap armoring and deep sheet pile walls to protect the bridge abutments as seen in **Figure 30**.

It is unclear how these differences in circulation patterns and velocity distributions would influence sedimentation or accretion within Beltz Marsh. Once the marsh is fully reconnected to tidal flows, accretion rates may be more dependent on sediment supply within the water than peak tide circulation patterns. The remaining portions of the dike after breaching would act as berms which can slow water on the marshplain and increase sedimentation by reducing wind/wave erosion. The remnant dike may act as a 'wave break berm' and serve this function. However, areas where water is funneled back into channels and through the dike breach would likely experience more erosion than deposition.

Setback Dike

Following the 2019 hydrologic study, stakeholder coordination, and subsequent technical advisory committee meetings, OPRD selected a preferred alternative concept that included breaching the outer Beltz Dike while constructing a new setback dike closer to TDM to maintain or improve flood protection for the adjacent community and Sandlake Road. This alternative provided the compromise that best met all the diverse goals of the project (OPRD, 2020).

At this time, a setback dike was conceived to be located approximately at or near the location of the existing primary beaver dam, approximately 700 feet north of the TDM/SSSNA boundary. In 2019, ESA performed a preliminary feasibility assessment to determine if a setback dike located near the beaver dam could have sufficient storage volume upstream of the setback dike to contain stormwater runoff from a 100-year storm event without resulting in higher peak water levels than under existing conditions (**Figure 31**).

The current project phase aims to refine the location of a potential setback dike to provide sufficient stormwater runoff storage capacity while minimizing wetland impacts and maximizing the area of marsh that could be reconnected to the Sand Lake estuary.

Optimizing Location/Alignment of Setback Dike

Constructing a new setback dike across the south end of Beltz Marsh will result in wetland impacts including wetland fill within the footprint of the setback dike as well as limiting the area of marsh that can be reconnected to full tidal interaction with Sand Lake. The further south that a setback dike can be located, the smaller the dike's footprint would be and the smaller the portion of wetland remaining disconnected from Sand Lake would be. However, shifting the setback dike southward would also reduce the potential storage volume available to contain stormwater runoff from TDM during high tides when a new tide gate would be closed. **Figure 32** shows a conceptual cross section of a setback dike that would be applied at Sitka Sedge, with a minimum, long-term, top elevation of 15' NAVD88 to maintain approximately 3 feet of freeboard over the maximum observed water levels at the site. **Figure 33** shows a conceptual plan of potential setback dike locations relative to TDM, Sandlake Road, and the channels/ditches that would contribute runoff to the area behind a setback dike.

In an effort to minimize wetland impacts and maximize estuary restoration, while protecting upstream infrastructure and private property, ESA developed a series of four setback dike alignment alternatives for detailed hydraulic modeling and analysis including:

- **Alternative 1** – located at or near the existing primary beaver dam and approximately 700 feet north of TDM (similar to the preliminary setback dike location evaluated in 2019). This alternative would provide the greatest potential storage volume to contain stormwater runoff from TDM. If this storage volume was determined to be insufficient, additional setback dike alternative locations further north would be considered.

- **Alternative 2** – located approximately 300 feet south of the existing primary beaver dam and 300 feet north of TDM. This alternative would reduce runoff storage volume and wetland impacts.

- **Alternative 3** – located at the southern park boundary and configured to allow the TDM Northwest Ditch to flow directly to the marsh outside of the setback dike thereby not contributing runoff to the setback dike's upstream storage basin. This alternative would significantly reduce wetland impacts and runoff storage volume but would route a portion of TDM's runoff outside of the setback dike, possibly reducing the necessary storage capacity.

- **Alternative 4a** – located along Sandlake Road and Roma Avenue, raising these roadways to a minimum elevation of 15' NAVD88 to function as the setback dike with a new culvert and tide gate at the Sandlake Rd/East Marsh crossing. Under this scenario, the NW Ditch and Sandlake Rd Ditch would flow directly to the marsh, while the East Marsh and adjacent TDM properties would be protected from the highest tides, and the East Marsh would function as the runoff storage basin. This option would nearly eliminate wetland impacts associated with the project.

- **Alternative 4b** – similar to 4a but without a new tide gate at the Sandlake Rd/East Marsh crossing. This alternative would allow full peak tide levels at the south end of Beltz Marsh and in the East Marsh but not retain any stormwater runoff upstream of the elevated embankment. This alternative would essentially be a non-setback dike alternative while raising Sandlake Road and Roma Ave above current and expected future peak water levels. This alternative could maintain roadway access during the highest expected water levels but would expose portions of the lowest-lying private properties in TDM to increased frequency of tidal inundation.

All setback dike alternatives assume the setback dike would have a 12-foot span box culvert fitted with a tide gate controlled by a muted tidal regulator (MTR) system, and the MTR would be set to stay open until upstream water levels exceeded 8.0' NAVD88. All alternatives also assume that the culvert at the Sandlake Rd/East Marsh crossing is replaced with a 12-foot span box culvert so that the water levels behind the setback dike freely propagate upstream into the east marsh.

Figure 34 shows the approximate locations of Setback Dike Alternatives 1 through 4 as described above.

Hydraulic Performance of Setback Dike Alternatives

Using the 2D HEC-RAS Model setup discussed in Section 4, ESA evaluated the performance of each setback dike alternative under a range of hydrologic conditions. For the purposes of understanding how each setback dike alternative would affect water levels at the south end of the marsh/north end of Tierra Del Mar, this analysis focused on modeling combinations of high tides and large storm events, including:

- High tides ranging from average high tide levels of 7-8' NAVD88, to frequently occurring peak high tides of 10' NAVD88, to maximum observed high tides or King Tides reaching approximately 12' NAVD88.
- Stream flows ranging from wintertime base flow conditions to peak stream flows and stormwater runoff contributions under frequently occurring 2-year storm events and peak stream flows and stormwater runoff during an extreme 100-year storm event.

This full range of potential wintertime conditions were combined into a single 14-day hydrologic scenario that includes:

- 3 days of typical wintertime conditions with base flow from the East Marsh and no runoff from the TDM subbasins combined with average tide conditions,
- 5 days of a 'first storm' that includes 2-year peak stream flows and stormwater runoff rates, combined with high tides reaching 10' NAVD88, and
- 6 days of a 'second storm' that includes 100-year peak stream flows and stormwater runoff rates, combined with King Tides reaching 12' NAVD88.

This longer-duration hydrologic scenario is intended to allow for a single alternative model run to capture a wide range of anticipated wintertime conditions and provide a realistic but conservative starting point for the extreme tide/storm event (with water already in the system). Water level plots shown in the following sections may include the full 14-day hydrologic scenario or focus in on one of the three periods described above (typical wintertime conditions, first storm/2-year storm, and 100-year storm).

Modeled Water Levels Inside Beltz Dike:

As a baseline for understanding the water levels that would occur outside of the setback dike under a dike breach scenario, ESA used the 2D model to compare water levels inside Beltz Dike (north end of Beltz Marsh) after dike breaching to existing conditions. Confirming conclusions made during the 2019 hydrologic study, the 2D model

results show that after dike breaching, water levels in Beltz Marsh would match the water levels occurring in Sand Lake. During typical wintertime conditions, dike breaching would allow for higher, short-duration peak water levels that are higher than water levels under existing conditions. However, under existing conditions, the existing undersized box culvert prevents efficient drain out of water and leads to higher water levels that take several tide cycles to drain out. During multi-day storm events, stormwater accumulates behind Beltz Dike creating higher water levels with each successive high tide event. During the simulated extreme storm event of 100-year stream flows, 100-year stormwater runoff, and king tides, the existing condition results in the highest peak water level, and it takes more than five days for water levels in Beltz Marsh to drain out to pre-storm levels. These modeled water level conditions inside Beltz Marsh under the 100-year storm scenario are shown in **Figure 35**. The 2019 hydrologic study concluded that these prolonged elevated water levels would contribute more to elevated groundwater levels than the short-duration peak tide levels that would occur under a dike breach alternative (PGG, 2019).

If no setback dike were constructed, these modeled peak tides under the dike breach scenario would occur at the south end of the marsh/north end of TDM. However, due to the presence of the existing beaver dam, water levels at the south end of the marsh would never fall below approximately 8' NAVD88.

Modeled Water Levels at South End of Marsh Under Setback Dike Alternatives:

ESA utilized the 2D model to simulate water levels at the south end of Beltz Marsh (at the north end of TDM) under each setback dike alternative compared to existing conditions. **Figure 36** shows predicted water levels at the south end of the marsh during the 100-year storm hydrologic scenario. **Figure 37** zooms in on results from the 'first storm' (2-year flows with 10' tides), and **Figure 38** zooms in on the 100-year storm event (100-year flows with King Tides). As previously described, the existing conditions result in the highest peak water level and slow drain out of water leading to prolonged elevated water levels.

Setback Dike Alternative 1 has the largest upstream storage capacity of all setback dike alternatives and results in lower water levels than existing conditions during almost all points in the 14-day simulation period. Alternative 1 results in higher peak water levels than existing conditions during the first two high tides of the 2-year storm event, but these peaks are 8.5' and 8.9' NAVD88 respectively which would not be problematic. In addition to lower peak water levels, Alternative 1 also provides rapid drain out after the peak of high tide.

Setback Dike Alternative 2 similarly provides rapid drain out after the peak of high tides. However, the reduced storage capacity compared to Alternative 1 results in higher peak water levels than existing conditions during all high tides within the 2-year storm and only marginally reduces the peak water levels during the 100-year storm event.

Because Alternative 2 would generally be an improvement over existing conditions during large storm events but would not be as effective as Alternative 1, ESA tested a variation of rerouting flows from the TDM Northwest Ditch to a new outfall on the beach. This modification would reduce the amount of stormwater runoff entering the storage basin behind the setback dike and may result in lower water levels behind Setback Dike Alternative 2. However, model results show that this modification would have a negligible effect on water levels behind the setback dike because the relative proportion of runoff from the TDM Northwest Ditch subbasin is very small compared to the Sandlake Road Ditch subbasin (both of which are dwarfed by the streamflow contribution of No Name Creek/East Marsh watershed). These model results are shown in **Figure 39**. While this modification would

not improve the performance of a setback dike, rerouting flows from the Northwest Ditch subbasin would still be beneficial for relieving localized ponding in depressional areas within the subbasin such as at Austin Ave, Pier Ave, and Jasmine Ave.

Setback Dike Alternative 3 also provides rapid drain out, but because of the minimal storage capacity behind the setback dike, peak water levels are only marginally lower than unrestrained water levels in Sand Lake.

Setback Dike Alternative 4, which raises Sandlake Road and Roma Ave to function as the setback dike for areas upstream, would have no influence on water levels at the south end of Beltz Marsh. Under this alternative, peak water levels at the south end of the marsh would match water levels in Sand Lake. Because this alternative is coupled with dike breaching, this configuration would also provide rapid drain out after the peak of high tide and would also be an improvement compared to existing conditions during a 100-year storm event with King Tides.

Modeled Water Levels in the East Marsh/Upstream of Sandlake Road:

ESA also evaluated modeled water levels upstream of Sandlake Road in the East Marsh under all setback dike alternatives. Because Tillamook County Public Works and other stakeholders are planning to replace the existing culvert in this location, all modeled alternatives assume this culvert is upsized to a 12-foot span box culvert. This culvert upsizing generally results in similar water levels upstream of Sandlake Rd as are predicted downstream of Sandlake Rd for all setback dike alternatives. **Figure 40** shows modeled water levels in the East Marsh for all setback dike alternatives compared to existing conditions during the 2-year storm with 10' tides. **Figure 41** shows modeled water levels in the East Marsh during the 100-year storm with King Tides.

Setback Dike Alternatives 4a and 4b (with and without a tide gate, respectively) result in similar water levels within the East Marsh due to the minimal storage capacity available within the East Marsh.

5.0 Deviations from QAPP

At the outset of the project phase, ESA worked with TEP and Environmental Protection Agency (EPA) to develop a Modeling Quality Assurance Plan (QAPP). ESA conducted the hydraulic modeling and analysis in accordance with the QAPP. Minor deviations from the QAPP are described here. These deviations have not affected the selection of the preferred alternative described in the 30% Basis of Design Report.

In the next project phase, ESA and subconsultant team members will advance and refine the design of the preferred alternative elements based on an upcoming geotechnical investigation. During the final design phase, ESA will conduct additional hydraulic modeling of the final configuration to document anticipated hydraulic performance for stakeholder communication and permitting. Deviations from the QAPP during the alternatives analysis phase will be reconsidered for implementation in the next stage of hydraulic modeling.

5.1 – Field Data Collection

Water Level Monitoring

As planned, TEP and ESA conducted additional water level monitoring within the project area in 2022/2023. However, 2D modeling largely relied on hydrologic data from 2017-2019 for several reasons. Calibration of existing conditions 2D HEC-RAS model was conducted before new water level monitoring data was available.

Additionally, there were no significant rainfall storm events that coincided with King Tide events during the 2022-2023 monitoring period, so utilizing the newer water level data would not have provided any better storm/tide events to evaluate storm and extreme tide conditions.

2022/2023 water level monitoring data was used to evaluate how further deterioration of the existing tide gate has affected water levels in Beltz Marsh. In 2017 the tide gate was missing one or more boards, allowing water to flow into the marsh during high tide resulting in peak water levels approximately 1.4' higher than the outboard peak water level in Sand Lake, as shown in **Figure 7**. In 2022/2023, the tide gate boards are completely missing allowing more water in and resulting in inboard peak water levels approximately 1.9' higher than outboard water levels. Therefore, the further deterioration of the tide gate has resulted in peak tidal water levels in Beltz Marsh approximately 0.5' higher in 2023 than in 2017.

2022/2023 water level monitoring also included a water level logger located closer to the mouth of Sand Lake. During low tides, water levels at this 'estuary channel' location fell to approximately 3.0' NAVD88 (more than 2 feet lower than water levels immediately outside of Beltz Dike but still several feet higher than low tide water levels in the Ocean. This data confirmed our understanding that the bed features in Sand Lake retain water and prevent full tidal drain out of Sand Lake but provided evidence that this effect decreases closer to the mouth at the Pacific Ocean.

Topographic and Bathymetric Data

As planned, ESA generated a modeling terrain surface incorporating the PhoDAR surface and the SWLS 2023 survey data. Drone survey data was taken and developed in 2022, as detailed in the QAPP. However, the model calibration process demonstrated the PhoDAR data better represented the site conditions, with the exception of the Beltz Dike. ESA modified the PhoDAR-based model terrain to enforce the top of the Beltz Dike, as confirmed in the drone survey.

Although identified in the QAPP, ESA did not collect a significant amount of additional ground survey measurements for the purposes of incorporating vegetation-bias adjustments implemented in OPRD's PhoDAR dataset or bathymetry data. In the model calibration process, ESA effectively modified the Sand Lake and Beltz Marsh channel bathymetry in the vicinity of the dike, yielding water levels inside the dike within 0.5 feet of monitored levels, by burning in channels with a bottom elevation of 3 feet (NAVD88). Bathymetric adjustments in the southern portion of Beltz Marsh were not necessary because the existing beaver dam consistently maintains a ponded water level of 8 feet. Vegetation adjustment was not necessary because great care had already been taken during the previous hydrologic study to adjust the PhoDAR terrain model for vegetation conditions.

Calibration yields water levels within 0.5' of monitored water levels at peak flood levels, demonstrating representation of marsh water levels sufficient for 30% design effort. Additionally, modeled water levels are the same or higher than monitored water levels, showing slightly higher peaks (up to 0.5') and slightly slower drainage following flood peaks, as shown in **Figure 42**. This shows the modeling is slightly conservative under flood conditions.

5.2 – Hydrologic Scenarios

Streamflow hydrographs for each of the contributing streams/watersheds (Reneke Creek, Beltz Creek, and East Marsh watershed) were developed using a watershed scaling approach based on a nearby gaged stream (Tucca Creek). Using an actual streamflow hydrograph from Tucca Creek corresponding to the January 2017 storm event used for existing conditions model calibration, the proportional ratio of watershed area (contributing watershed/Tucca Creek watershed) was applied to generate streamflow hydrographs for each of the contributing watersheds. To develop streamflow hydrographs for the extreme storm/flood model scenarios, these hydrographs were again scaled up by a factor that would result in Tucca Creek's peak streamflow reaching the level expected for a 2-year, 50-year, and 100-year recurrence interval event. The resulting peak stream flows for Reneke Creek and Beltz Creeks were compared to USGS Streamstats peak flow estimates for these creeks. During model setup and upon a detailed review of previous streamflow hydrograph development methodology, streamflow hydrographs based on a watershed scaling approach were deemed sufficient for calibrating the existing conditions 2D model and for comparing the hydraulic performance of the proposed design alternatives. Additionally, the Tucca Creek watershed has a higher 24-hour rainfall depth than the project site according to available isophurvial maps (NOAA, 2001). Therefore, the scaled hydrographs from Tucca Creek are likely conservative. HEC-HMS was not used to develop streamflow hydrographs as described in the QAPP.

The QAPP also noted that ESA may use the 'rain-on-grid' feature in HEC-RAS to apply precipitation data directly onto the model terrain mesh to generate stormwater runoff. However, this method was not utilized due to increased model run times and instability caused by attempting this method. As planned and described in the QAPP, storm event stormwater runoff rates from the Tierra Del Mar drainage basins were developed using Santa Barbara Unit Hydrograph (SBUH) method.

Additional hydrologic analysis and hydraulic modeling will be conducted in subsequent design stages to further refine the design of the setback dike and its drainage structures. We will consider developing additional hydrologic input data using HEC-HMS or rain-on-grid method at that time.

The QAPP identified additional hydrologic scenarios including summertime periods for water quality considerations and future climate-change scenarios. This phase of project work focused on identifying a preferred project alternative. Typical winter tides, high tides and extreme flood events were the basis for this effort. Further design stages may include additional hydrologic analysis and modeling to consider water quality and climate change.

5.3 – Develop and Calibrate 2D Existing Conditions Model

As detailed in the QAPP, water levels were monitored in 2022-2023. The monitoring time did not include any extreme high tides or flood events. Therefore, the 2022-2023 data was deemed not suitable for calibration of the model's extreme flood scenarios. The model was calibrated to previously collected water level data and time-stamped site photos of problematic flooding within the project area.

5.4 – Proposed Conditions Modeling of Alternatives

ESA modeled the proposed alternatives features including the dike breach size, number, and location; and potential setback dike locations. Proposed restoration of historic tidal channels, filling linear drainage, and potential improvements to TDM's stormwater system were evaluated qualitatively (i.e., they were not incorporated in the Alternatives model geometries). These features are still being considered for inclusion in the design. While it is believed that adding sinuosity and complexity to the site's channel network, completing this earthwork would cause impacts to existing high-quality vegetation and disturbance to the marsh soils. Additional site reconnaissance and coordination will be needed to determine if the key stakeholders believe the impacts would be worth the improvements. If proposed for inclusion in the final design, the updated 2D hydraulic model will be modified to include these channel modifications to determine how these changes would affect site hydraulics and drainage/flood control performance in areas south of the setback dike.

6.0 References

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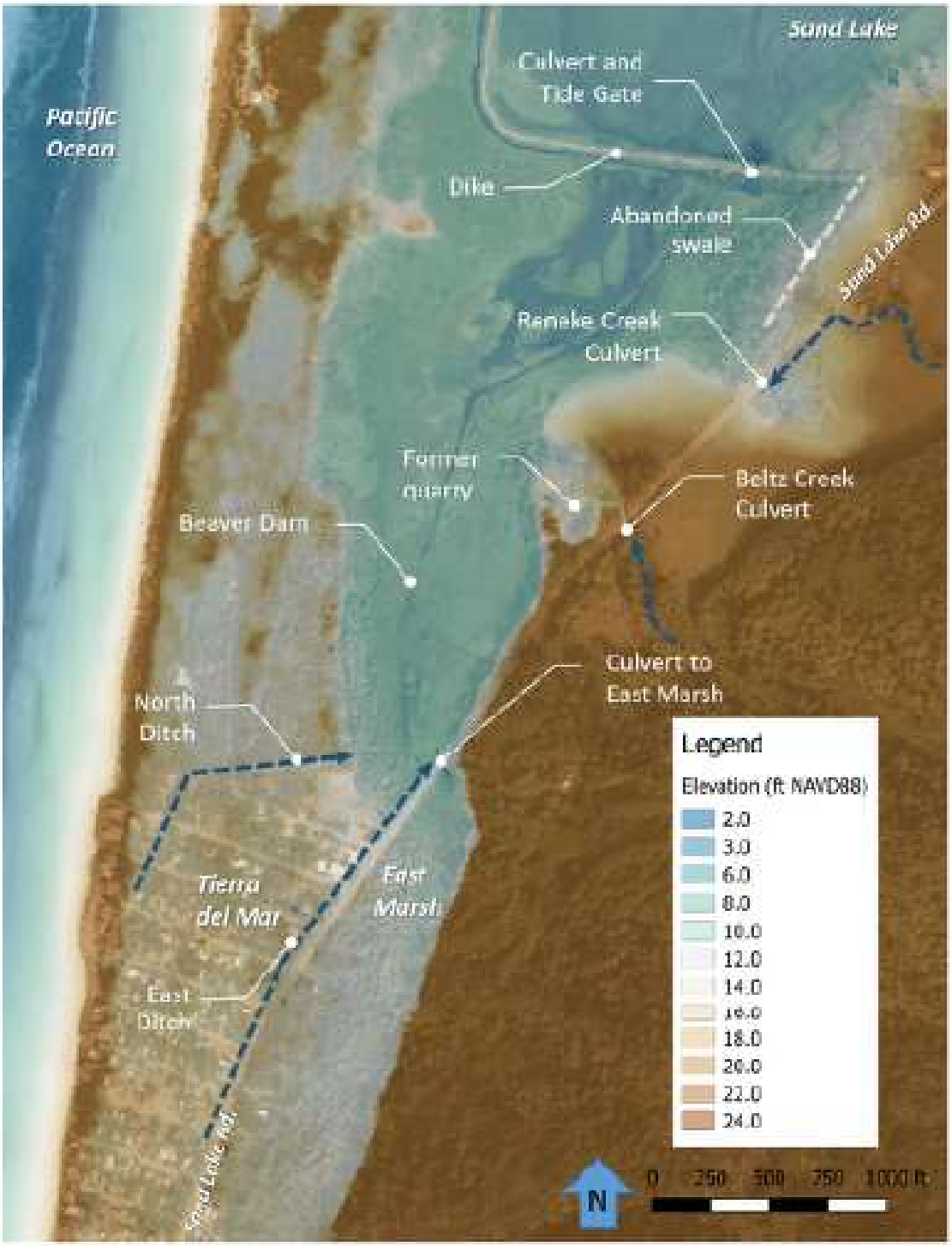
Figure 1
Project Location Map

ESA



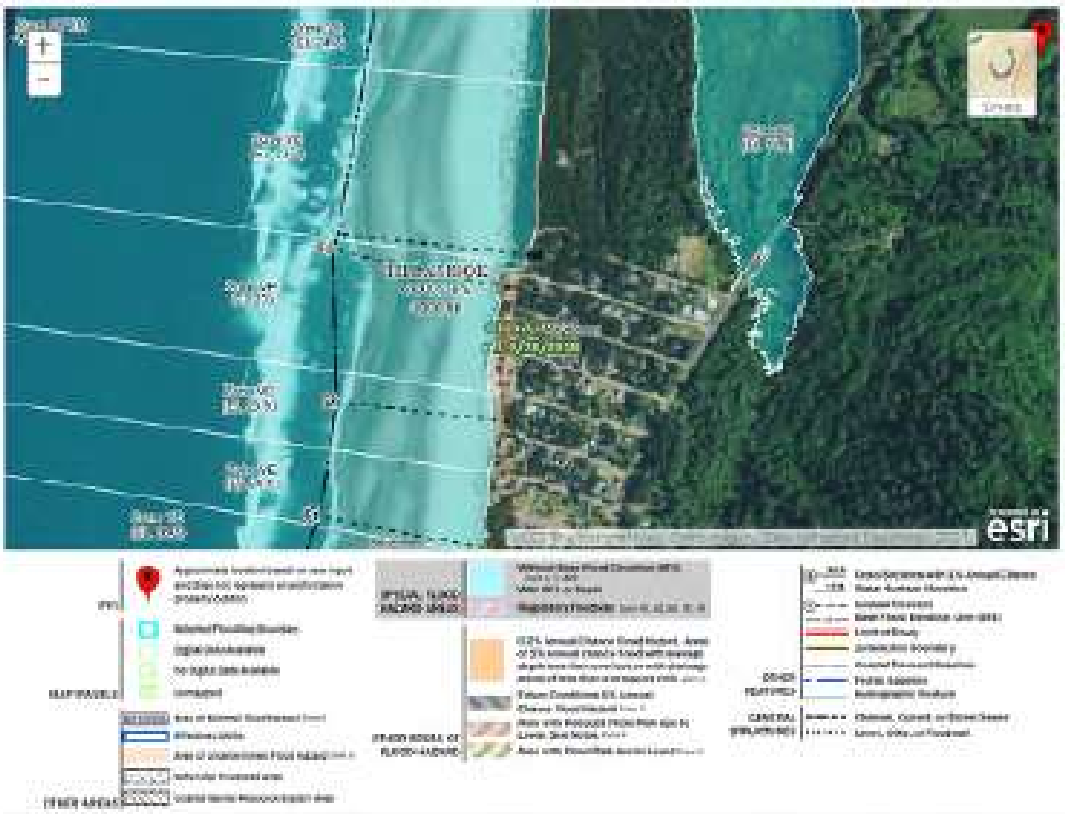
Note: This figure is an aerial photo identifying key site features of the project area.

Figure 2
Annotated Site Map and Aerial Photo



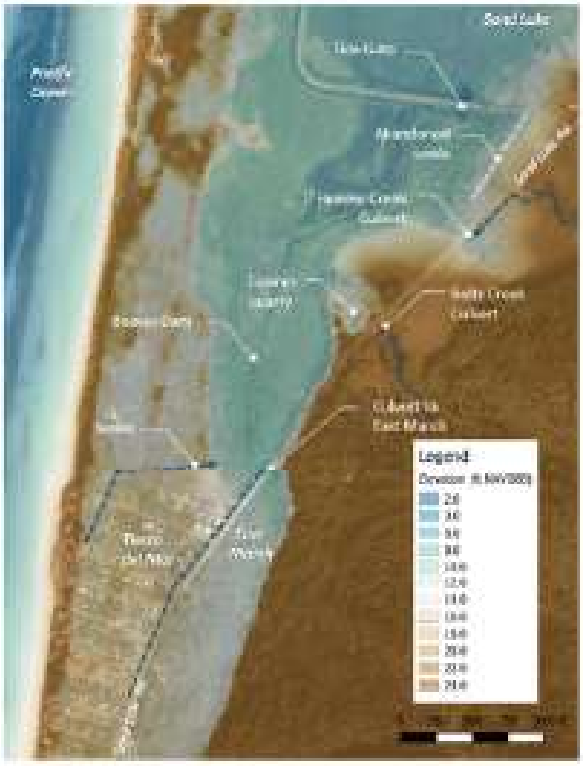
Note: This figure is an aerial photo with a LiDAR elevation data overlay. Key site features are labeled for reference.

Figure 3
Annotated Site Topographic and Hydrologic Map



Note: *This figure shows* the FEMA Flood Insurance Rate Map of the project area, including the mapped 100-year floodplain boundary which inundates the lowest lying portions of Tierra Del Mar and overtops the low point in Sandlake Road at the East Marsh crossing. The road low point elevation is 11.6' NAVD88 compared to the 100-year floodplain elevation of 11.8'.

Figure 4
FEMA Flood Insurance Rate Map Excerpt from North End of TDM



FEMA Base Flood EL. +11.8 ft

Low top of Dike EL. +12.8 ft
Bottom of Tide Gate EL. +1.3 ft
Scour hole EL. -4.5 ft

Low tide in Sand Lake EL. +5.5 ft
MSL in Ocean EL. +3.8 ft

Reneke Ck. Culvert Invert. EL. +6.6 ft

Beltz Ck. Culvert Invert. EL. +20.7 ft

Top of Beaver Dam EL. ~6.0 ft

Culvert to East Marsh Invert EL. 5.15 ft
Low Point in Sand Lk. Rd EL. ~11.6 ft

Culvert Invert at Roma Ave = +11.7 ft
Land in Tierra del Mar EL. +14 to 22 ft

Note: This figure summarizes the elevations of key features within the project area. All elevations are referenced to North American Vertical Datum 1988 (NAVD88).

Figure 5
Elevations of Key Site Features and Water Levels (NAVD88)

ESA

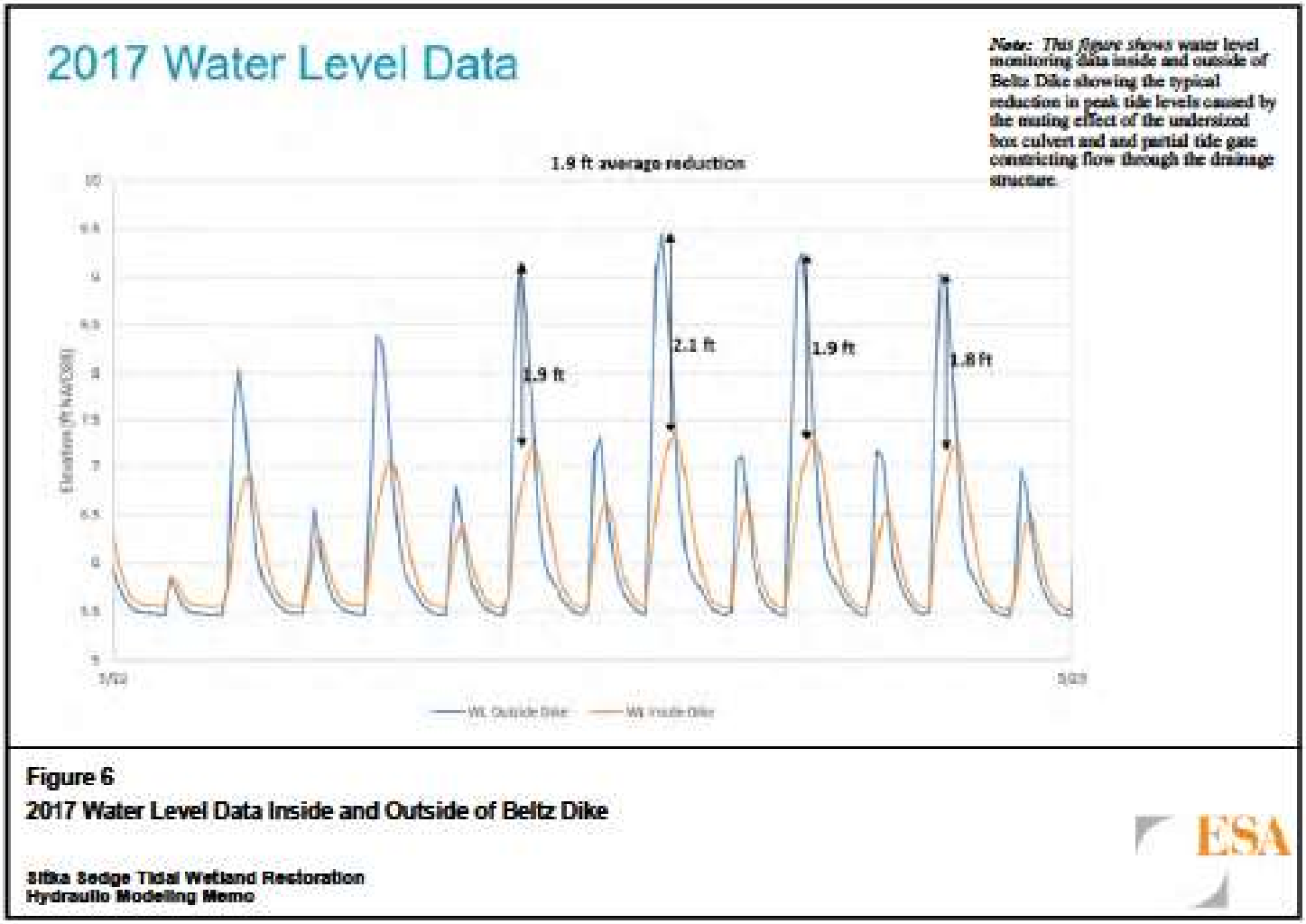


Figure 6
2017 Water Level Data Inside and Outside of Beltz Dike

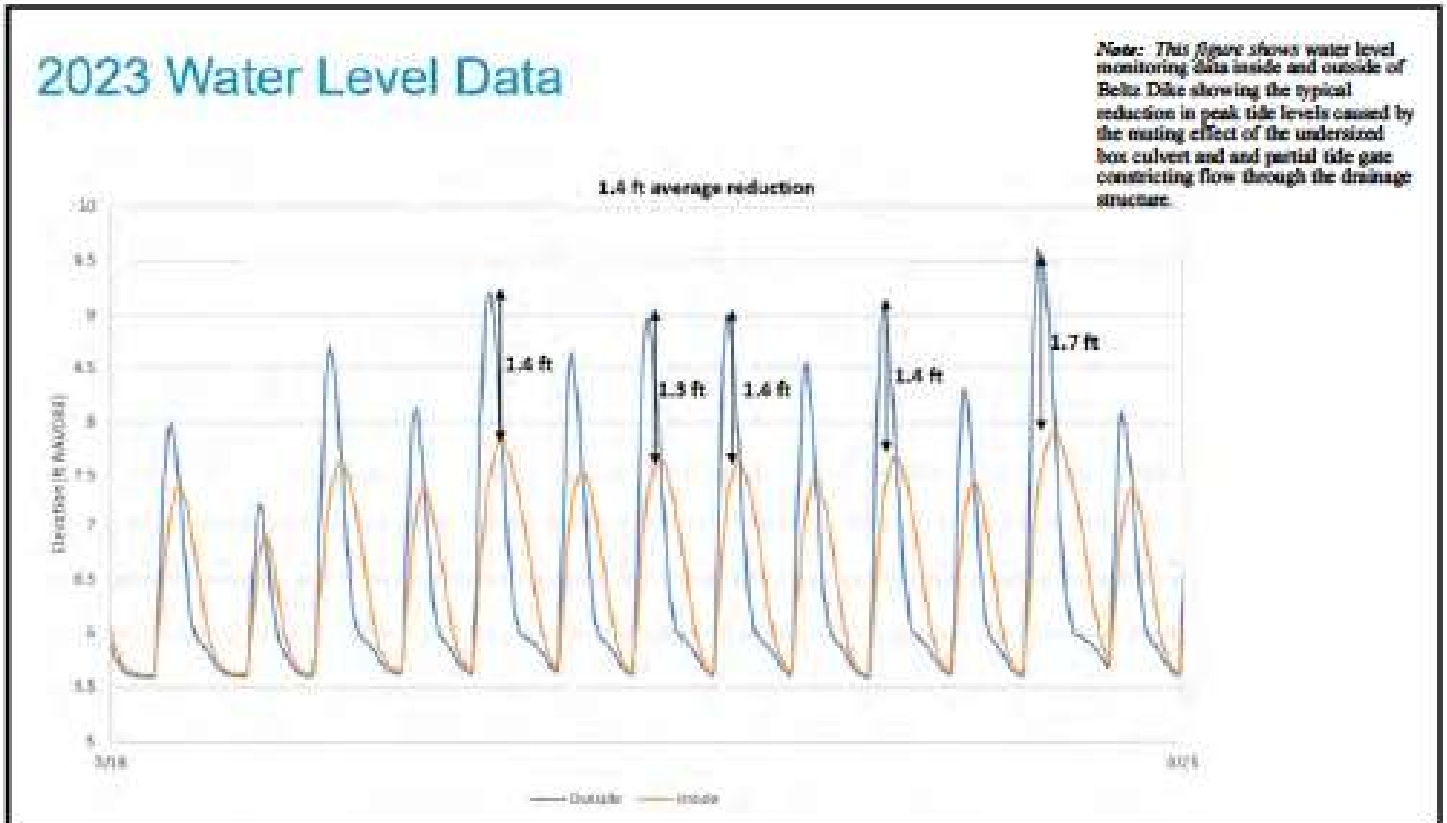
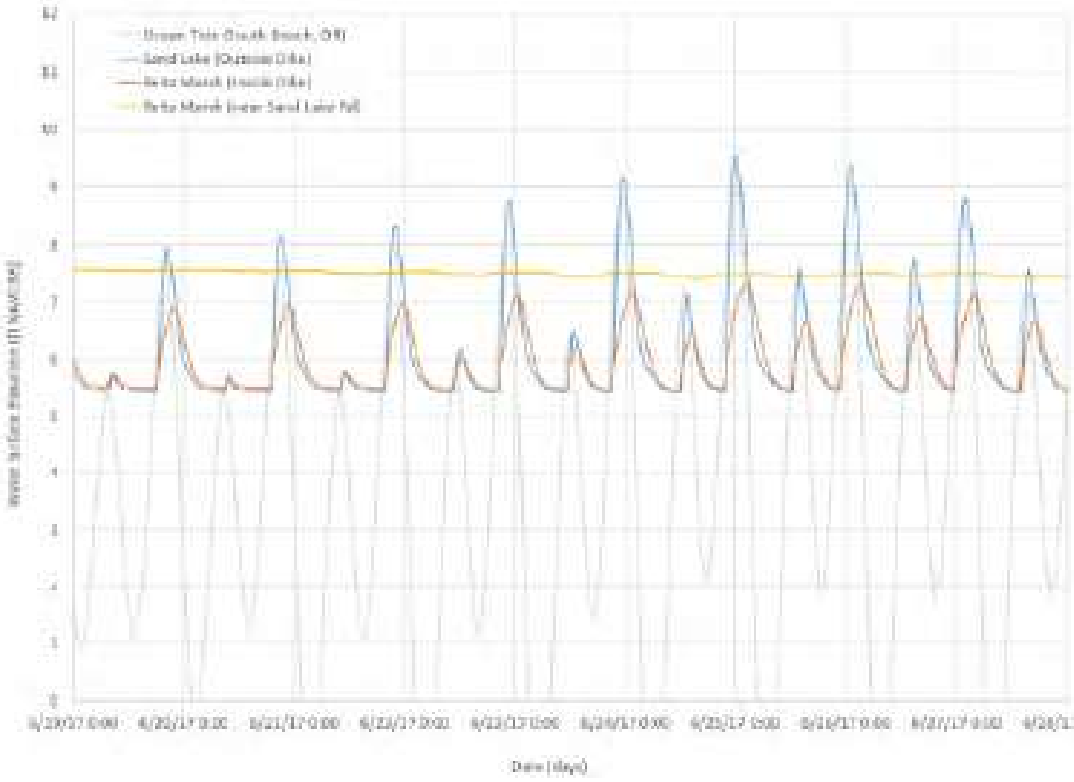


Figure 7
2023 Water Level Data Inside and Outside of Beltz Dike



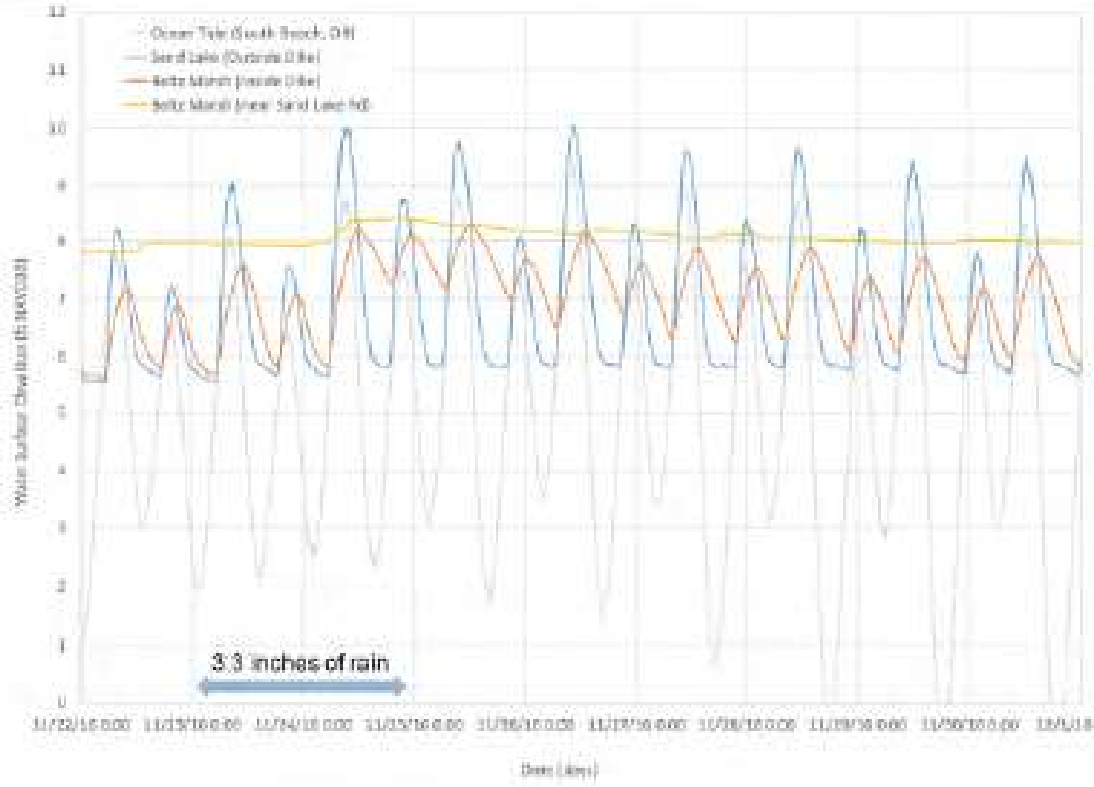
Note: *This figure shows water level data collected during a typical period during the dry season.*

Ocean tides fluctuate between 0' and 9.5'. Sand Lake water levels are typically slightly higher than the ocean at high tide. Low tides in Sand Lake never fall below 5.5' due to controlling bed elevations.

Water levels inside the dike/ Marsh show a muted tidal connection. The tide gate prevents high tides from entering the marsh. Water drains out of the marsh as tides recede, slightly delayed due to limited capacity of the culvert.

Water levels at Sand Lake Road upstream of the beaver dam are relatively constant at 7.5'.

Figure 8
Typical Water Level Fluctuations in Beltz Marsh During the Dry Season



Note: *This figure shows water level data collected during a typical period during the wet season.*

Ocean tides fluctuate between 0' and 9.5'. Sand Lake water levels are typically slightly higher than the ocean at high tide. Low tides in Sand Lake never fall below 5.5' due to controlling bed elevations.

Water levels inside the dike/ Marsh show a muted tidal connection and low capacity for drainage. The tide gate prevents high tides from entering the marsh. After a rainfall event, water accumulates within the marsh, and the undersized culvert prevents efficient drainage. Accumulated water takes several days, and several low tides, to drain.

Water levels at Sand Lake Road upstream of the beaver dam are relatively constant at 8', with a slight increase following a rainfall event.

Figure 9
Typical Water Level Fluctuations in Beltz Marsh During the Wet Season



Recent Events	48-Hr Rainfall (inches)
Dec. 18, 2015	4.1
Nov. 24, 2016	3.3
Dec. 20, 2016	2.3
Feb. 9, 2017	3.1
Feb. 16-19, 2017	2.4 + 1.6
Mar. 7, 2017	1.3
Mar. 15, 2017	2.9

Note: This figure shows areas of surface-water ponding observed following rainfall events.

These ponded areas generally correspond to depression areas without positive drainage.

Figure 10
Areas of Surface-water Ponding in Tierra Del Mar





- Beltz Dike and tide gate
  - Limited tidal flux in/out of marsh
  - High tides behind dike are 1-2' lower than Sand Lake
  - Water still flowing into marsh when tides begin to fall
  - Culvert limits draining of marsh
  - Heavy rainfall events can take several tide cycles for freshwater to drain out of marsh
- Beaver Dam within marsh
  - About 3 feet tall
  - Limits tidal fluctuation at south end of marsh
  - Steady water levels 7 to 8' NAVD88
  - Backwater at culvert and east marsh
- Multiple Culverts (not shown)

Note: This figure provides summary information on the primary features that control surface-water movement into, out of, and within Beltz Marsh.

Figure 11
Hydraulic Control Features Affecting Surface-water Movement

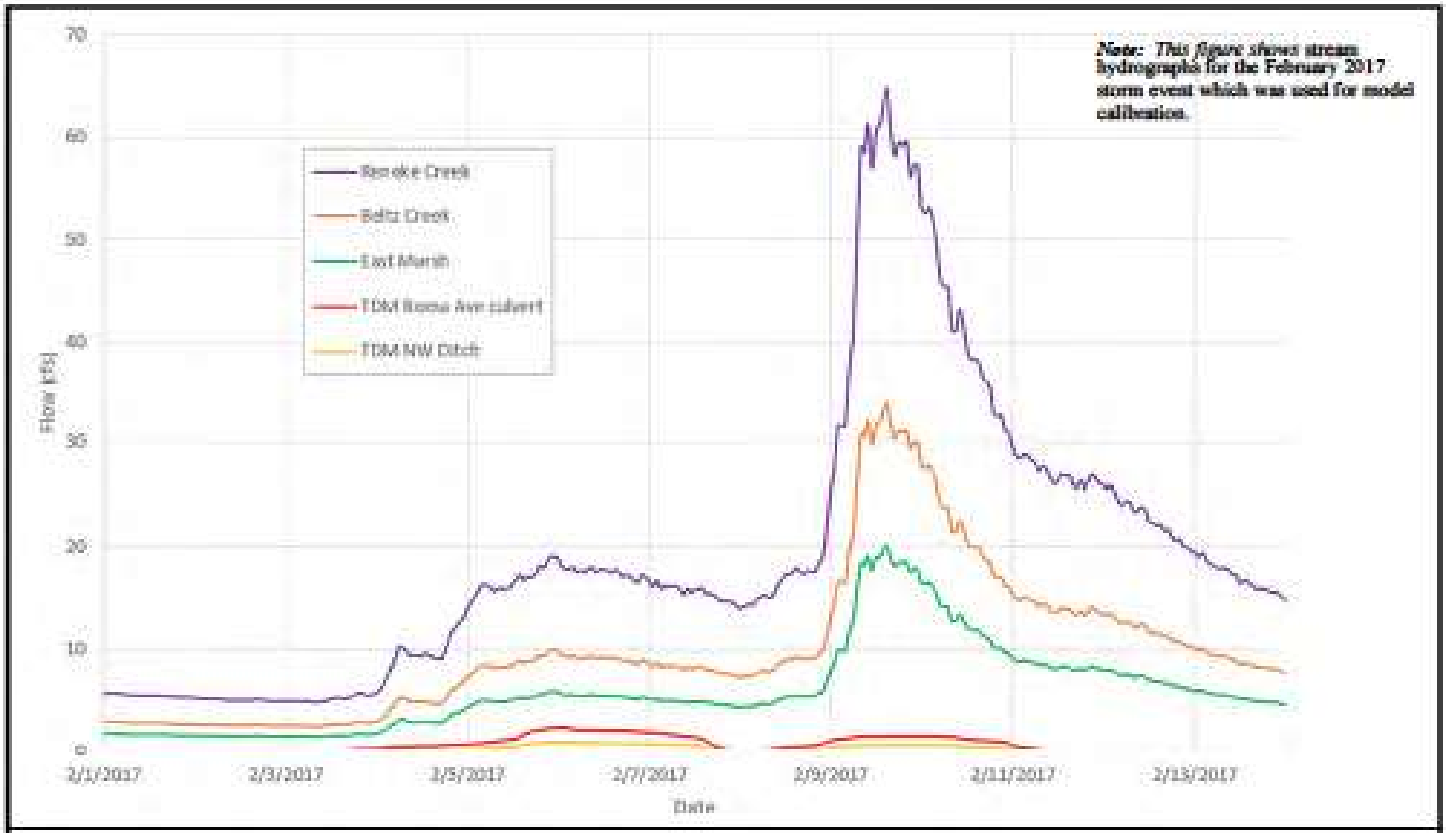


Figure 12
2D Model Inflow Hydrographs, February 2017 Calibration Event

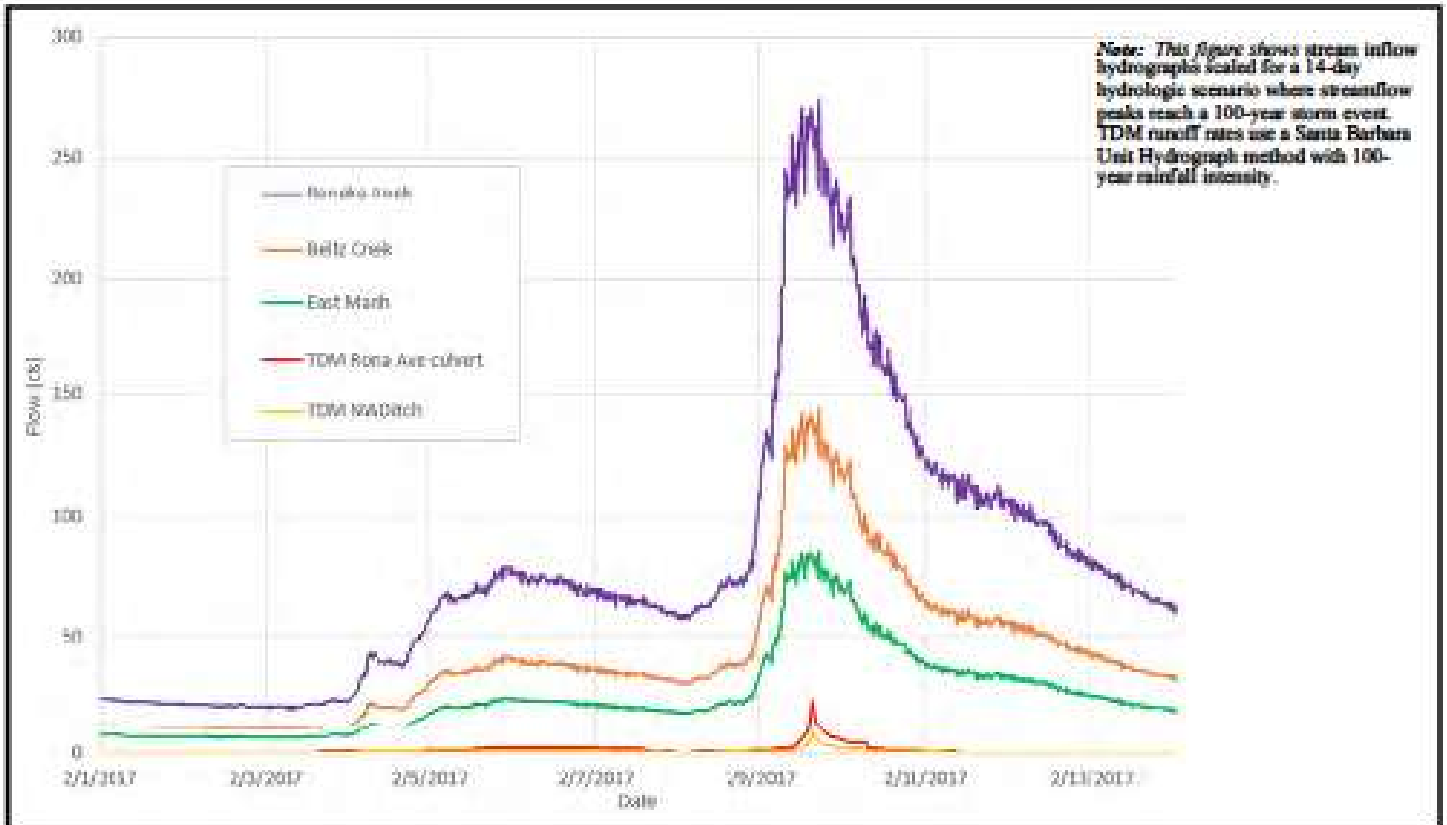


Figure 13
2D Model Inflow Hydrographs for the 100-year storm even

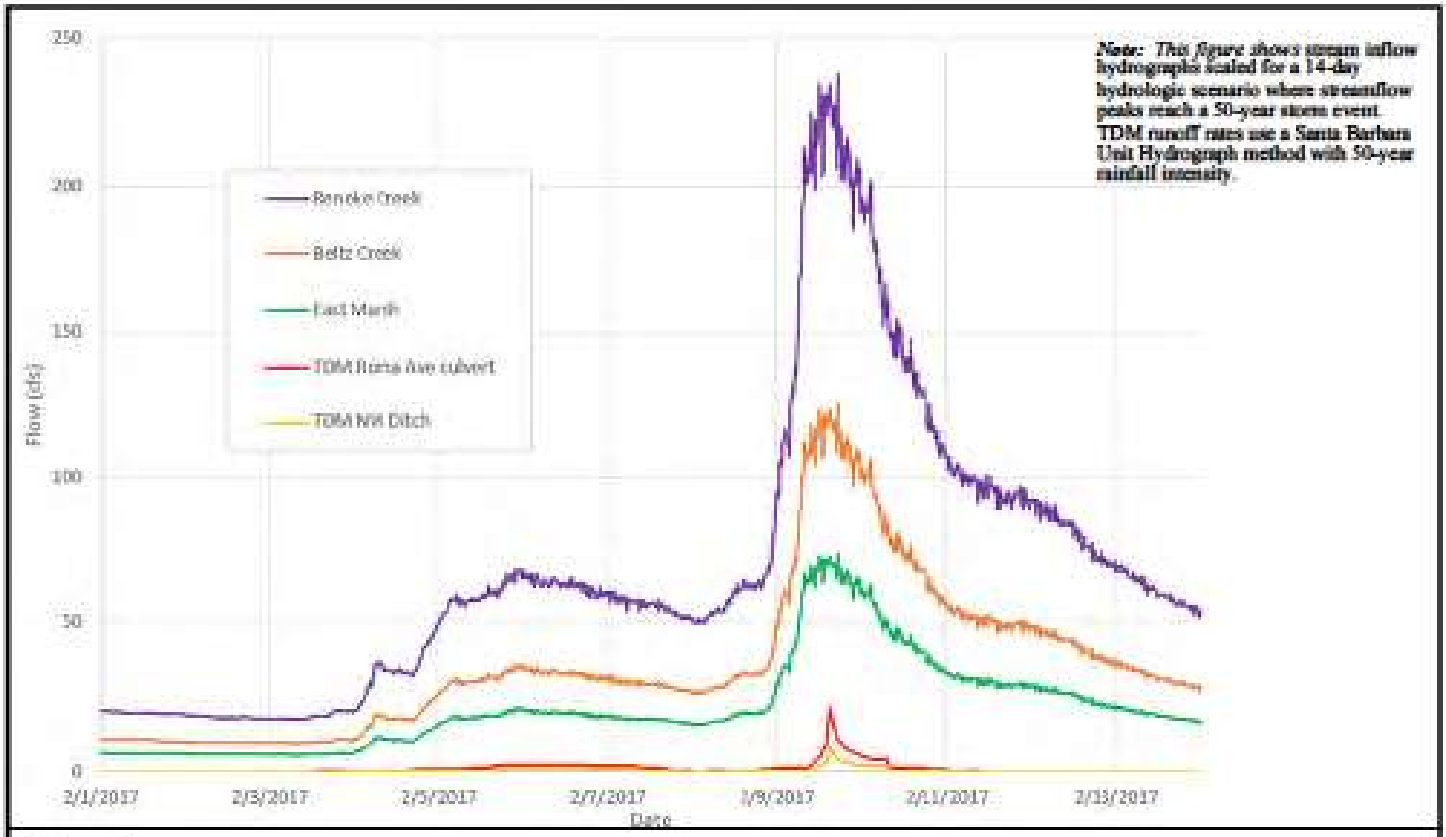


Figure 14
2D Model Inflow Hydrographs for the 50-year storm event

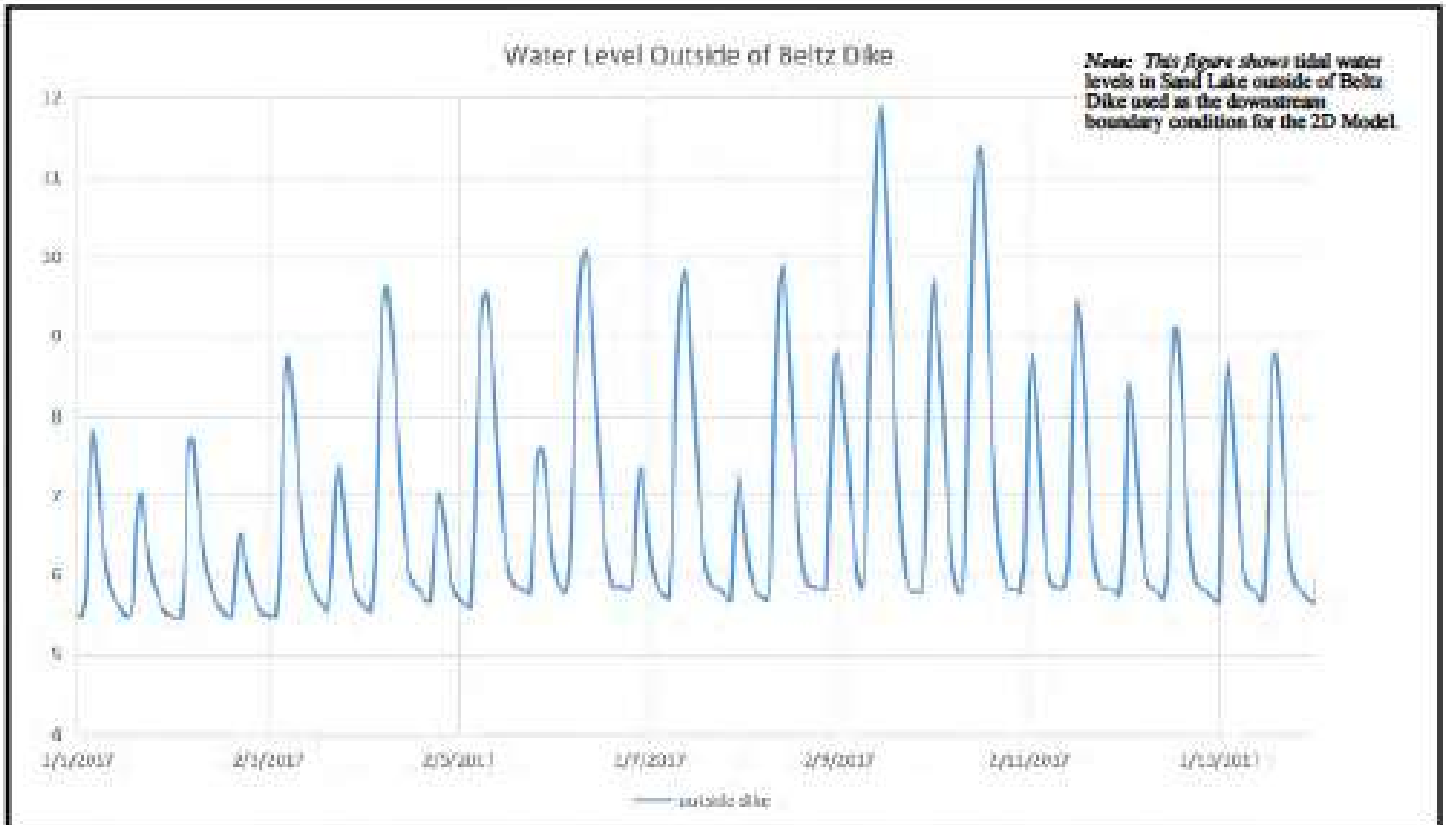


Figure 15
2D Model Downstream Boundary Condition



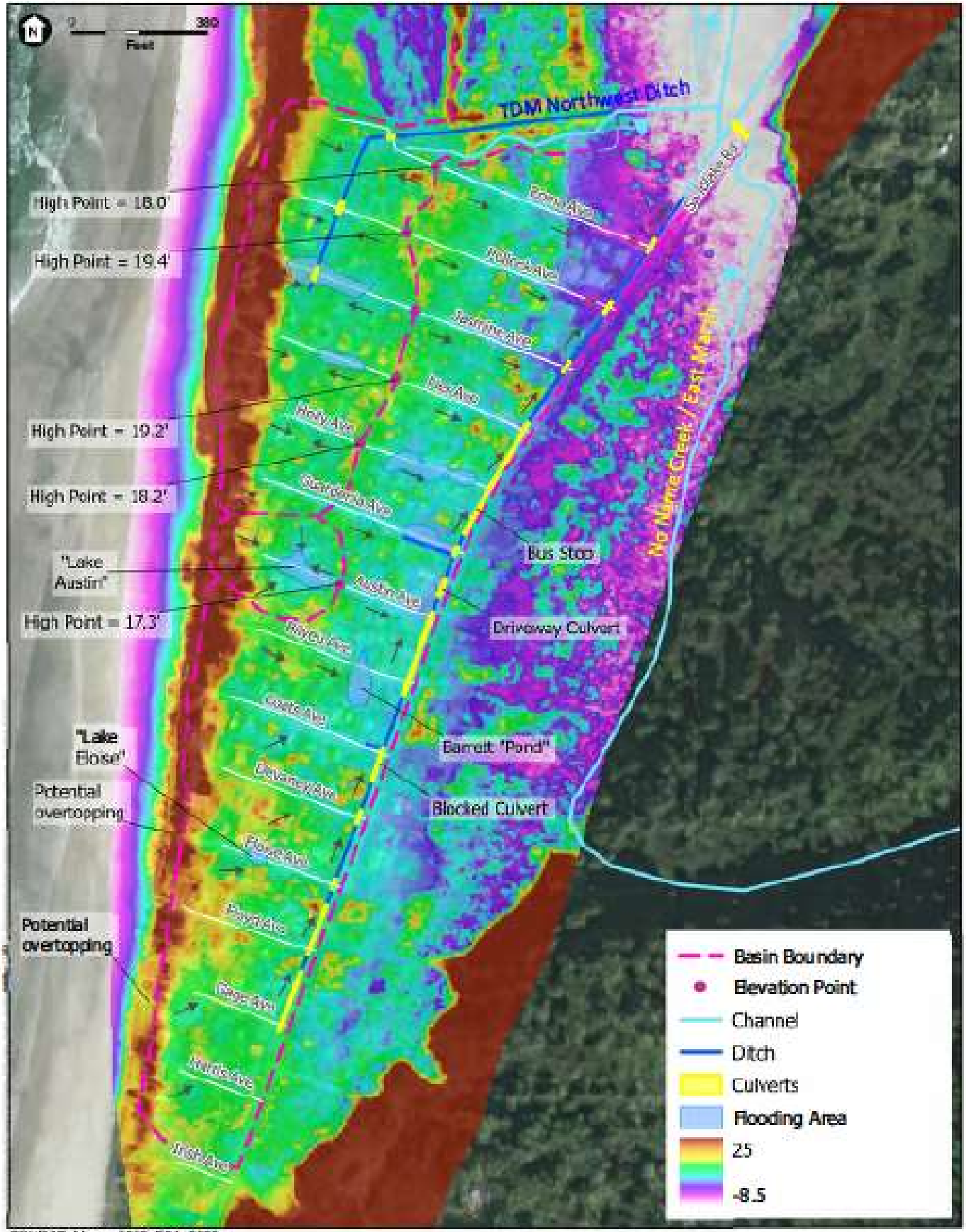
Note: This figure shows the extent of the 2D HEC-RAS Model, hydrologic inputs, and downstream boundary.

Figure 16
2D Model Extents, Downstream Boundary, and Hydrologic Inputs



SOURCE: Maxar, 2022; ESA, 2023

Figure 17
Tierra Del Mar Drainage System Analysis - Aerial Photo



SOURCE: Maxar, 2022; ESA, 2023

Figure 18

Tierra Del Mar Drainage System Analysis - LiDAR



Note: This figure shows an example of a fractal, dendritic tidal channel network, a planform that commonly occurs in low-gradient tidal marshes where many small tidal channels drain individual subbasins and funnel water into progressively larger channels that all drain to a single outlet.

This typical tidal channel planform supports a justification to focus dike breaching on a single dike breach channel rather than multiple dike breaches.

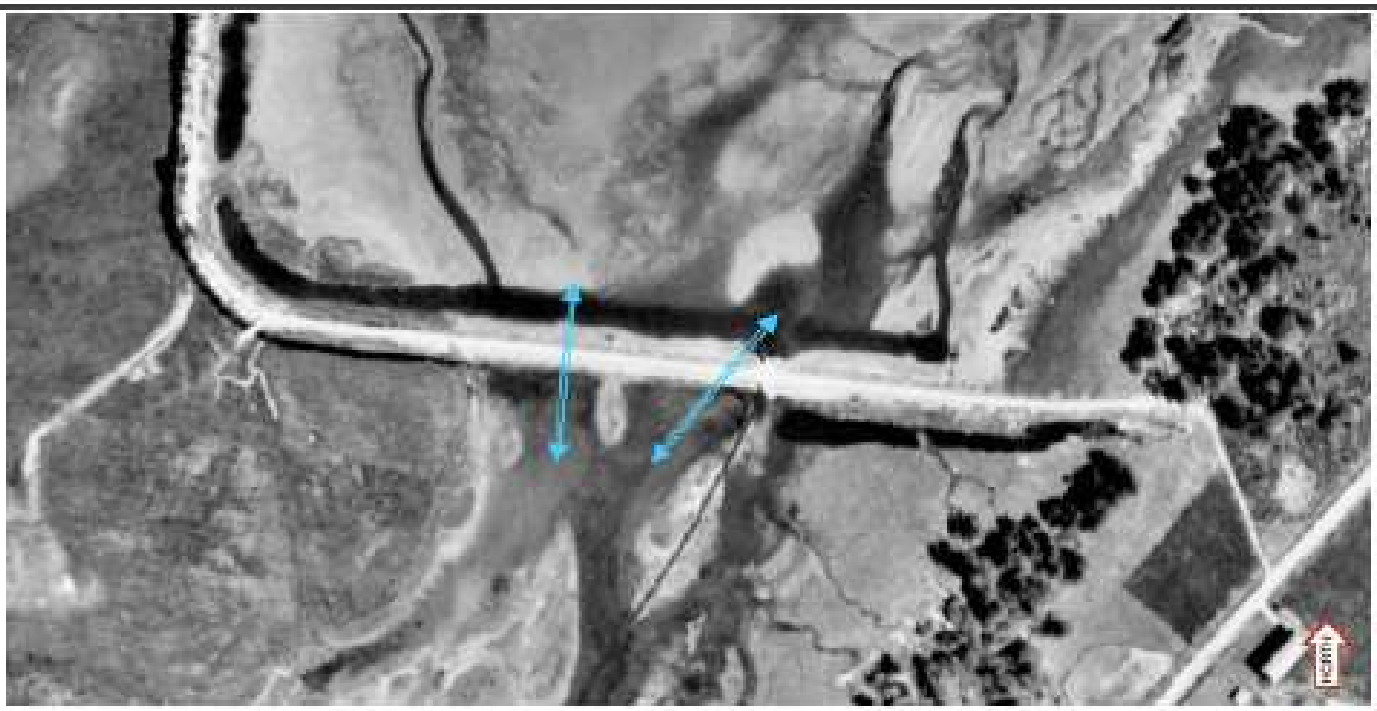
Figure 19
Example of Dendritic Tidal Channel Network Draining to Single Outlet



Note: This figure shows an example of a dendritic tidal channel network that exists nearby in the northeastern portion of the Sand Lake Estuary, a planform that commonly occurs in low-gradient tidal marshes where many small tidal channels drain individual subbasins and funnel water into progressively larger channels that all drain to a single outlet.

This typical tidal channel planform supports a justification to focus dike breaching on a single dike breach channel rather than multiple dike breaches.

Figure 20
Example of Dendritic Tidal Channel Network in Sand Lake Estuary



Note: This figure shows a historical aerial photo from 1939 (UofD Libraries, 2020), highlighting the apparent location where two large tidal channel forms cross Beltz Dike. This analysis is used to identify the appropriate location for a dike breach to restore tidal connectivity between Sand Lake and Beltz Marsh.

Figure 21
1939 Aerial Photo of Beltz Dike and Large Tidal Channel Planforms



Note: This figure shows a current aerial photo (NS Consulting, 2022), showing the central historical flow path connecting channels inboard and outboard of Beltz Dike, as well as the existing box culvert/tide gate location which is not aligned with historical flow paths.

Figure 22
2022 Aerial Photo of Beltz Dike, Existing Culvert, and Historical Flow Path



Note: This figure shows four potential dike breach locations under consideration.

Figure 23
Four Potential Dike Breach Locations Considered

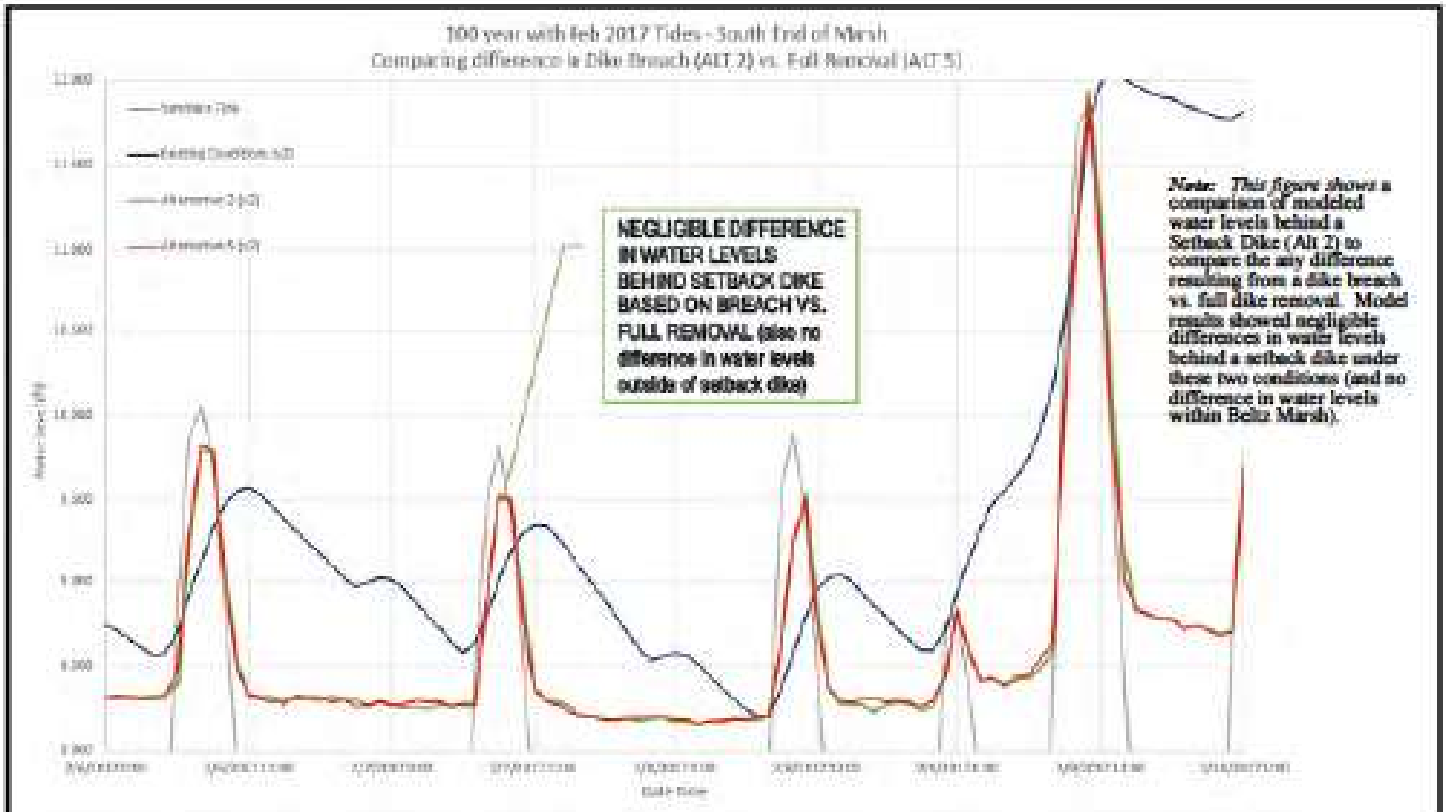
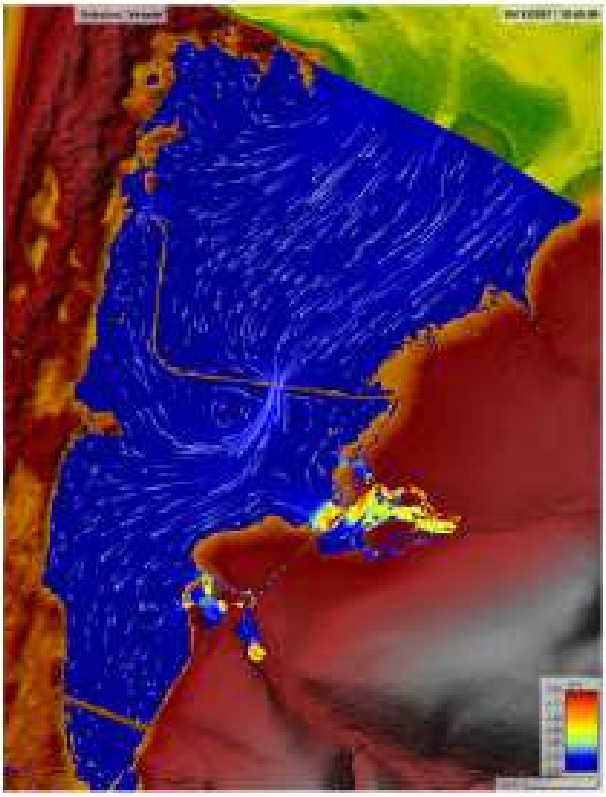
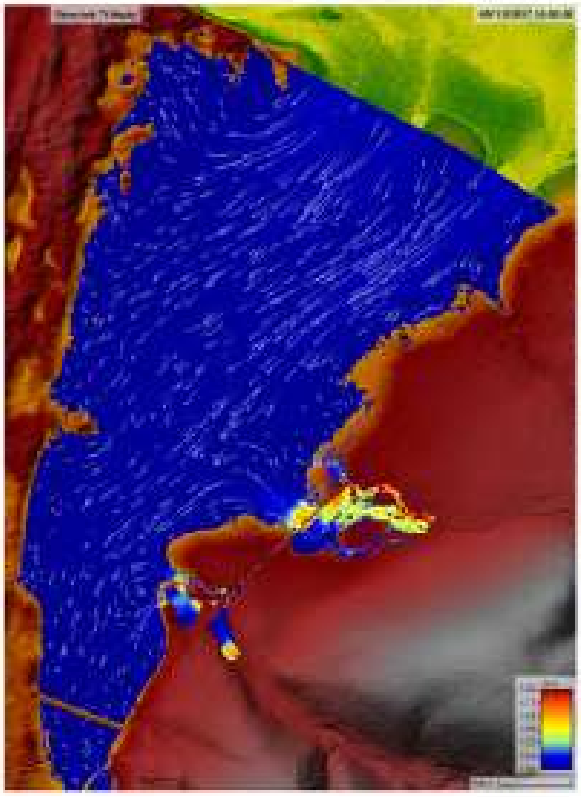


Figure 24
Comparison of Modeled Water Behind a Setback Dike Under Dike Breach and Full Dike Removal

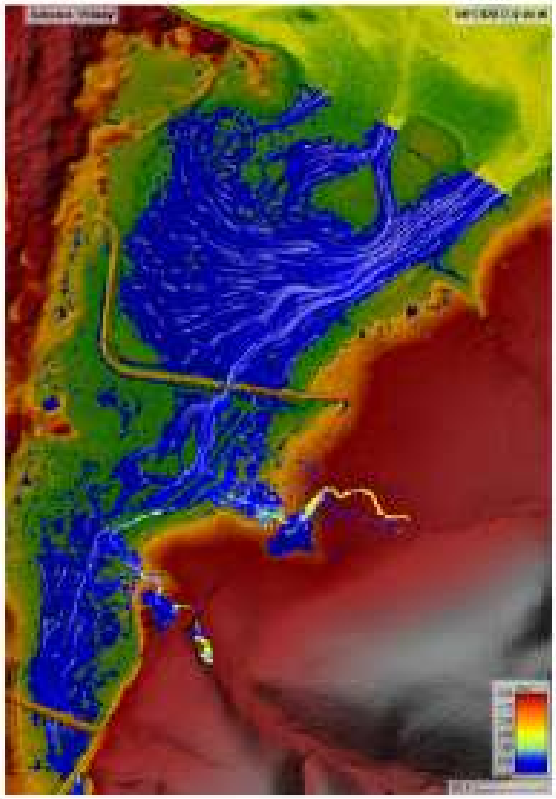


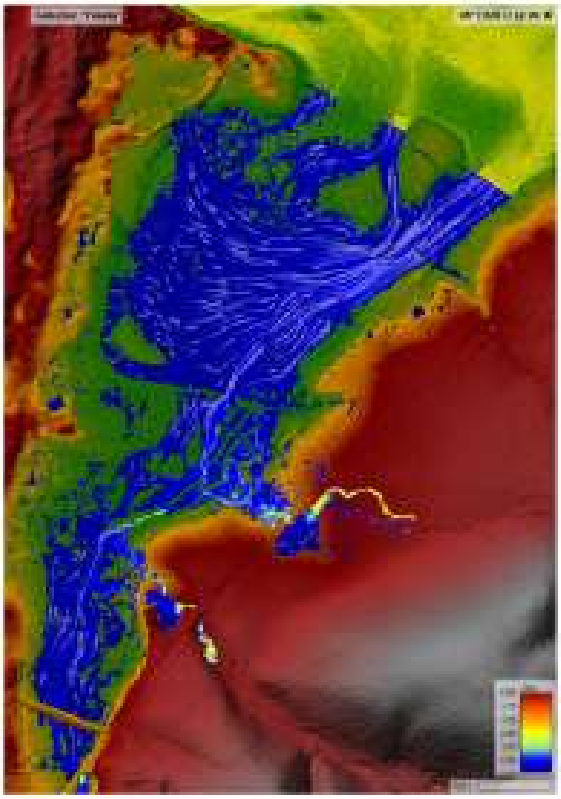


Note: *This figure shows a* comparison of water circulation patterns and flow directions under a dike breach vs. full dike removal during drain out while water levels are still high (above 9' NAVD88).

When water level are high, water in Belle Marsh is funneled to the center of the marsh towards the dike breach where velocities are higher and a small eddy is visible upstream of the dike breach. Under full dike removal, water flux is evenly distributed across the marsh.

Figure 25
Comparison of Circulation Patterns Under Dike Breach Vs. Full Dike Removal (High Water)





Note: This figure shows a comparison of water circulation patterns and flow directions under a dike breach vs. full dike removal during drain out after water levels have fallen below 9' NAVD88.

When water level are low, water in Belle Marsh is funneled to the center of the marsh under both dike breach and full dike removal conditions, where water is more confined to the channels.

Figure 26
Comparison of Circulation Patterns Under Dike Breach Vs. Full Dike Removal (Low Water)

PEAK VELOCITY DURING 100-YR STORM RUN (MAX TIDE SWING)

Peak velocity ~ 3ft/sec (short duration peak, majority of time is less than 2 ft/sec: fish passage velocity threshold)

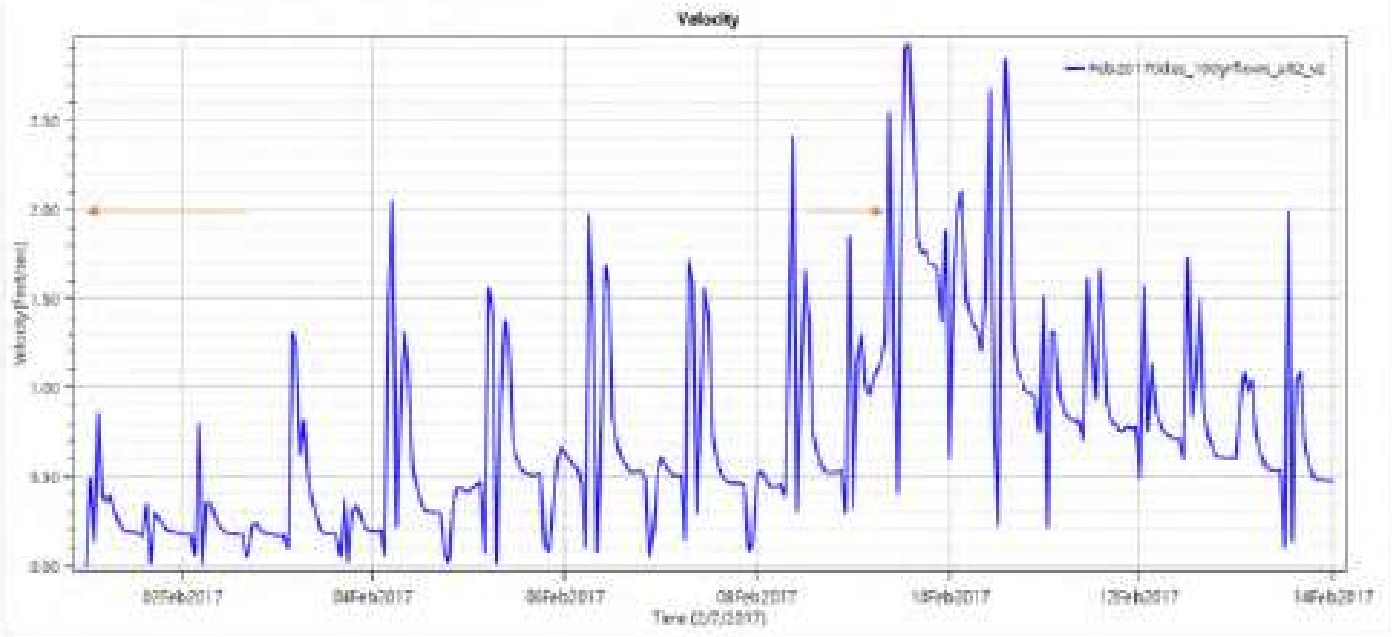


Figure 27
Water Velocity Through a 100-Foot Dike Breach During the 100-Year Hydrologic Scenario

100-Foot Dike Breach – Peak Shear Stress (lb/sf):

Alt 2 – 100-year run at the time step with the highest shear stress and velocity at the Beltz dike breach

Peak Shear Stress = 0.19 lb/sf (able to erode medium to coarse gravels. May need light armoring)

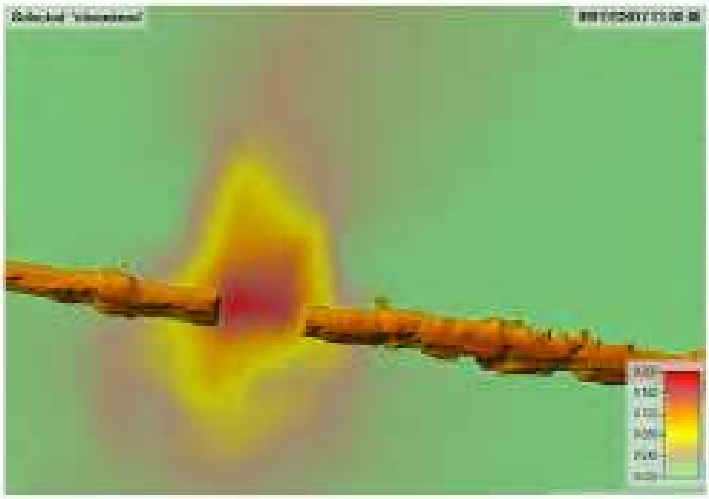


Table 1. Limiting Shear Stress and Velocity for Uniform Noncohesive Sediments

Class name	d_i (in)	ϕ (psf)	τ_{ci}	τ_{ci} (lb/sf)	V_{ci} (ft/s)
Boulder					
Very large	>80	12	0.084	37.4	4.96
Large	>40	12	0.084	18.7	3.68
Medium	>20	12	0.084	9.3	2.70
Small	>10	12	0.084	4.7	1.94
Cobble					
Large	>5	12	0.084	2.3	1.38
Small	>2.5	11	0.082	1.1	0.76
Gravel					
Very coarse	>1.3	10	0.058	0.54	0.52
Coarse	>0.6	10	0.047	0.25	0.36
Medium	>0.3	10	0.044	0.12	0.24
Fine	>0.15	10	0.042	0.06	0.17
Very fine	>0.08	10	0.039	0.03	0.13
Sands					
Very coarse	>0.04	12	0.029	0.01	0.070
Coarse	>0.02	11	0.033	0.006	0.055
Medium	>0.01	10	0.048	0.004	0.045
Fine	>0.006	10	0.072	0.003	0.040
Very fine	>0.003	10	0.109	0.002	0.035
Silts					
Coarse	>0.002	10	0.165	0.001	0.030
Medium	>0.001	10	0.25	0.001	0.025

Stability Thresholds for Stream Restoration Materials

by Craig Fischenich[1]

May 2001

Figure 28

Peak Shear Stress Through 100-Foot Dike Breach Compared to Stability Thresholds of Sediments

PEAK VELOCITY DURING 100-YR STORM RUN (MAX TIDE SWING)

100-Foot Breach : Peak velocity ~ 3ft/sec (short duration peak, majority of time is less than 2 ft/sec: fish passage velocity threshold)

200-Foot Breach: Peak velocity ~ 1ft/sec (slightly higher velocities in site's internal tidal channels)

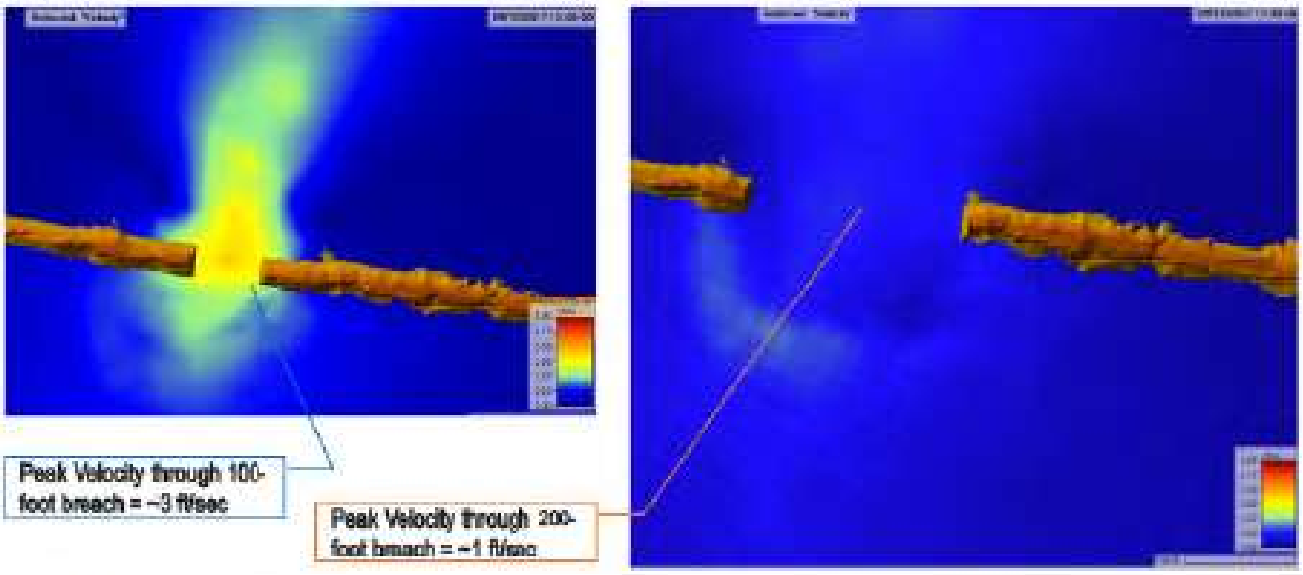


Figure 29
Comparison of Peak Velocity Through 100-Foot and 200-Foot Dike Breaches

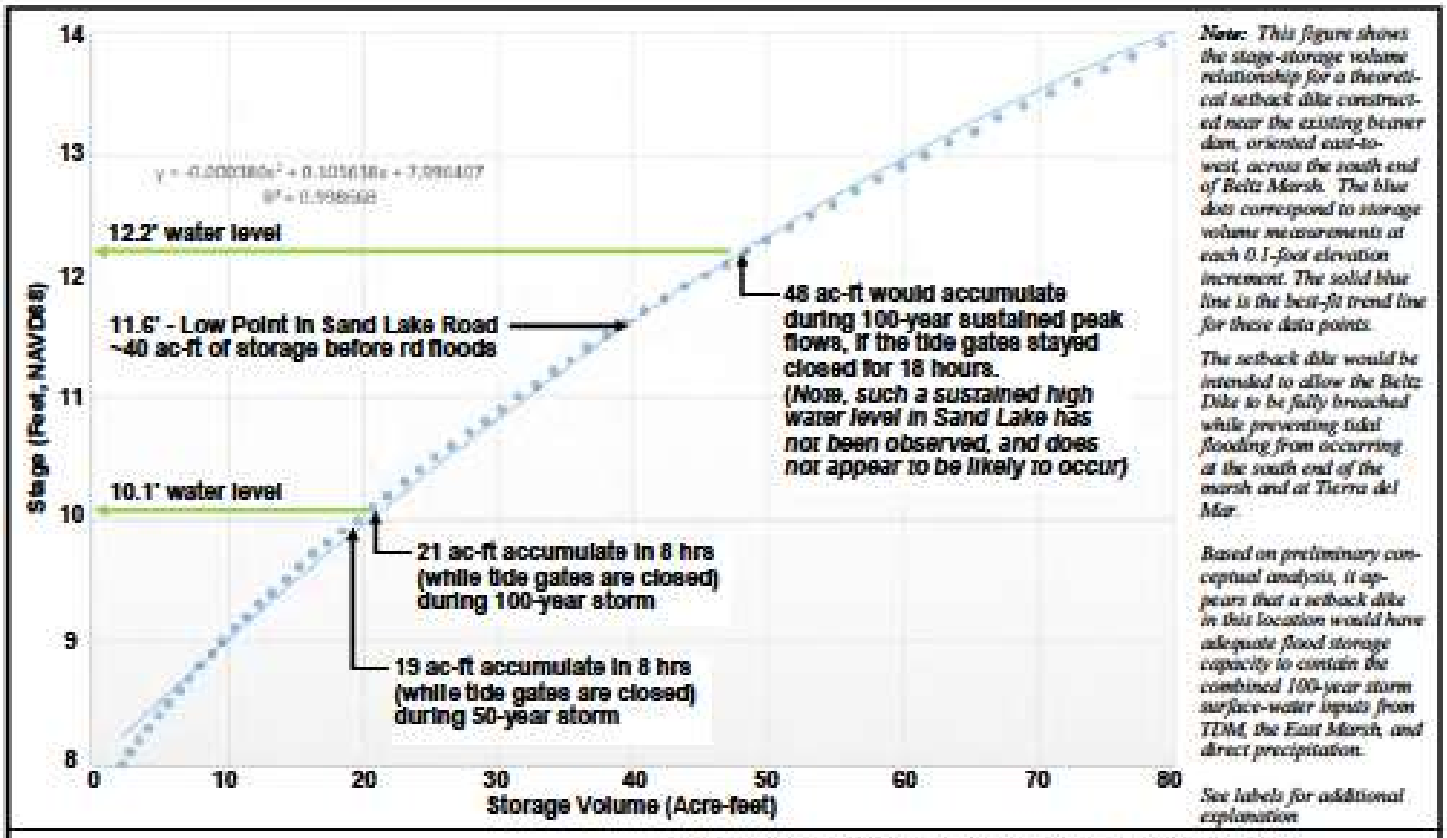
Whalen Island Rd Bridge

- 180-acre contributing marsh area (80 acres at Sitka Sedge)
- 80-foot span
- Heavily armored opening/banks with riprap



Note: This figure shows the nearby Whalen Island Road bridge as a point of comparison of span length compared to upstream contributing marsh area and required armoring due to constriction and high velocities/shear stresses.

Figure 30
Whalen Island Bridge



Note: This figure shows the stage-storage volume relationship for a theoretical setback dike constructed near the existing berner dam, oriented east-to-west, across the south end of Beltz Marsh. The blue dots correspond to storage volume measurements at each 0.1-foot elevation increment. The solid blue line is the best-fit trend line for these data points.

The setback dike would be intended to allow the Beltz Dike to be fully breached while preventing tidal flooding from occurring at the south end of the marsh and at Tierra del Mar.

Based on preliminary conceptual analysis, it appears that a setback dike in this location would have adequate flood storage capacity to contain the combined 100-year storm surface-water inputs from TDM, the East Marsh, and direct precipitation.

See labels for additional explanation

SOURCE: ESA derived stage-storage relationship based on PhotoSAR terrain surface provided by DPRD.

Figure 31
Preliminary Feasibility Analysis of a Setback Dike from 2019 Study

Note: This figure shows a conceptual cross section of a setback dike that would likely be appropriate for the Sitka Sedge project, including a 12-foot minimum top width, 3:1 and 5:1 side slopes, resulting in a bottom width/ footprint of 80 to 100 feet.

Setback dike dimensions, build height, and subgrade preparation/stabilization measures will be determined in subsequent design stages informed by a subsurface investigation and geotechnical engineering recommendations.

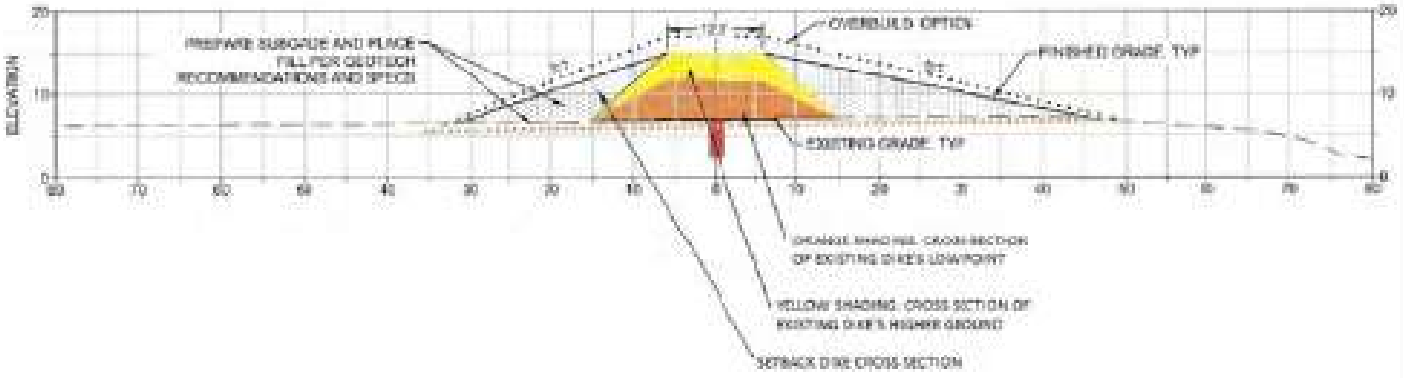


Figure 32
Conceptual Cross Section of a Setback Dike at Sitka Sedge

Note: This figure shows a a concept plan for optimizing the setback dike location showing potential setback dike alternative alignments and flow contributions to the area behind a setback dike. (North is down in this image)

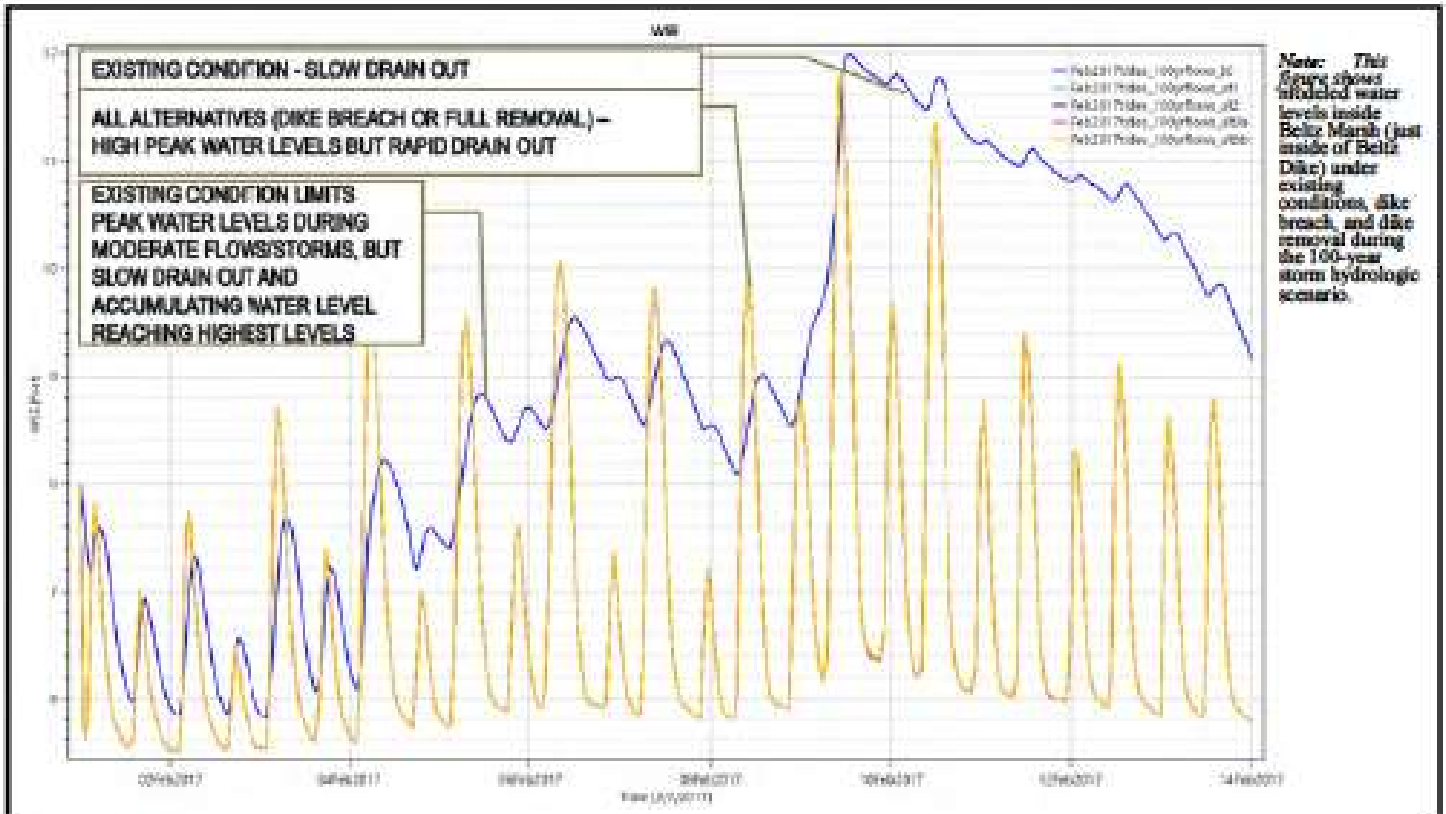


Figure 33
Concept Plan for Optimizing Location of Setback Dike



Note: This figure shows four alternative setback dike alignments.

Figure 34
Setback Dike Alternative Approximate Locations



Note: *This figure shows* modeled water levels inside Beltz Marsh (just inside of Beltz Dike) under existing conditions, dike breach, and dike removal during the 100-year storm hydrologic scenario.

Figure 35
Modeled Water Levels Inside Beltz Marsh During 100-Year Storm Scenario under Dike Breach and Dike Removal vs. Existing Conditions

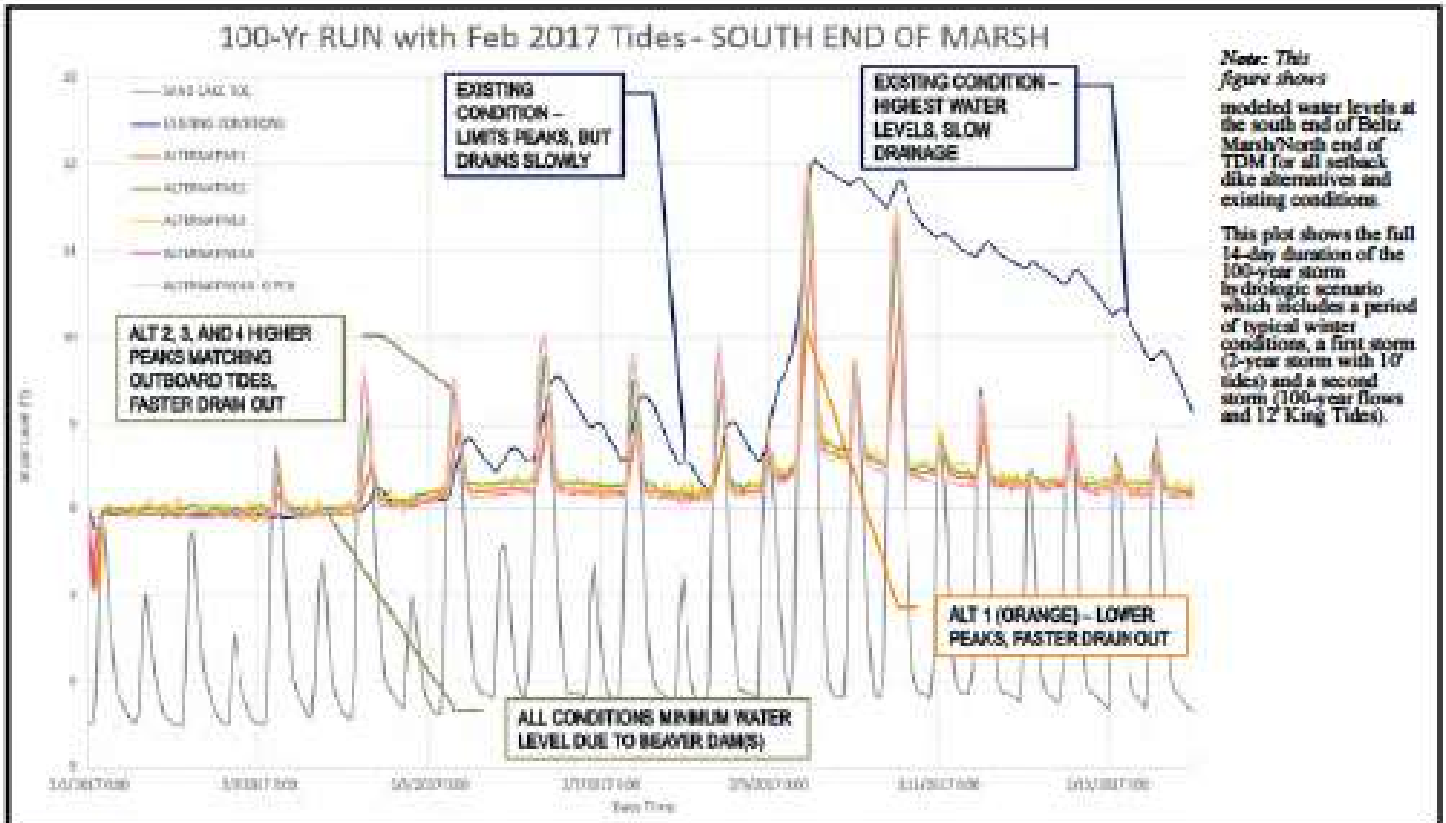
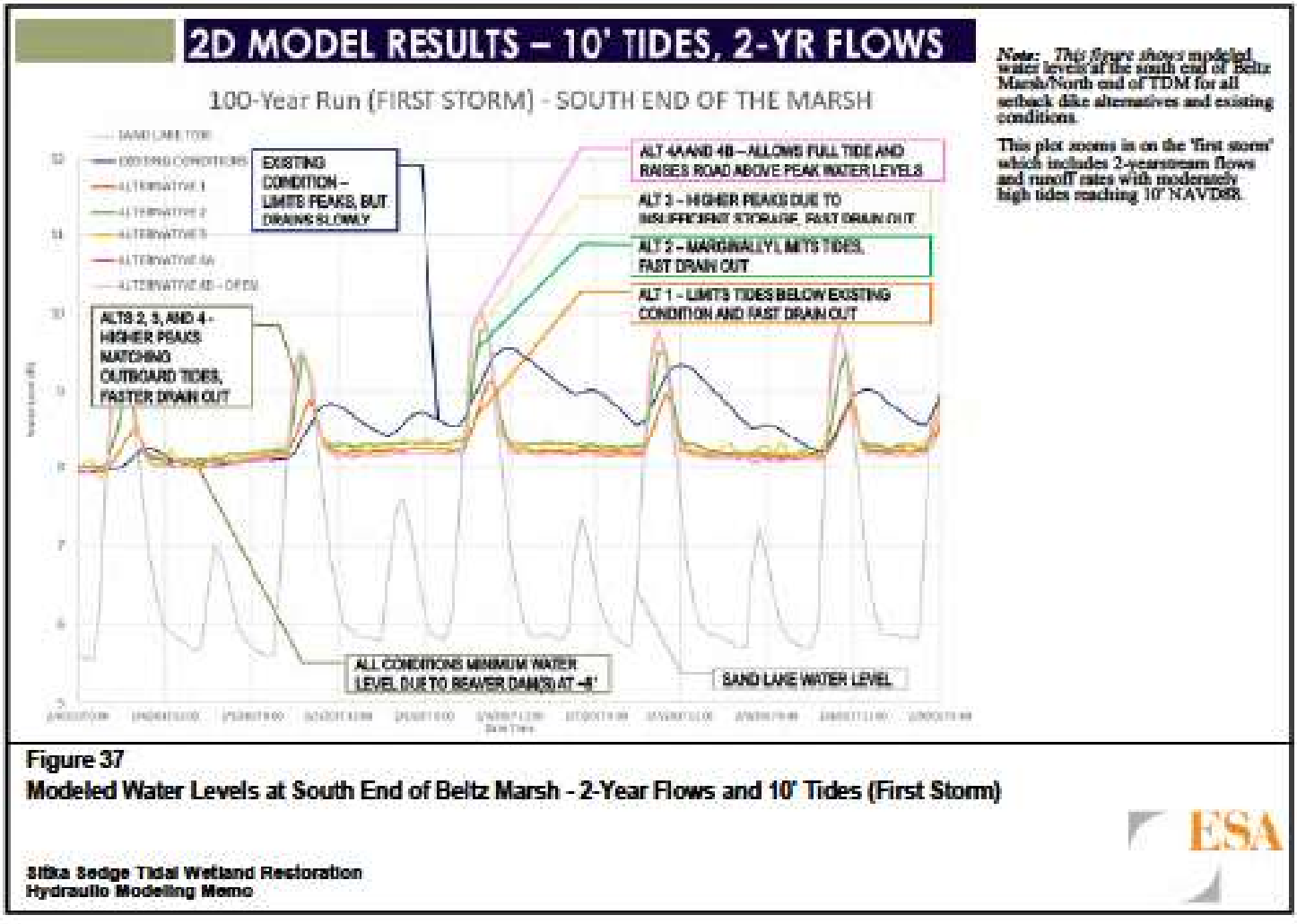


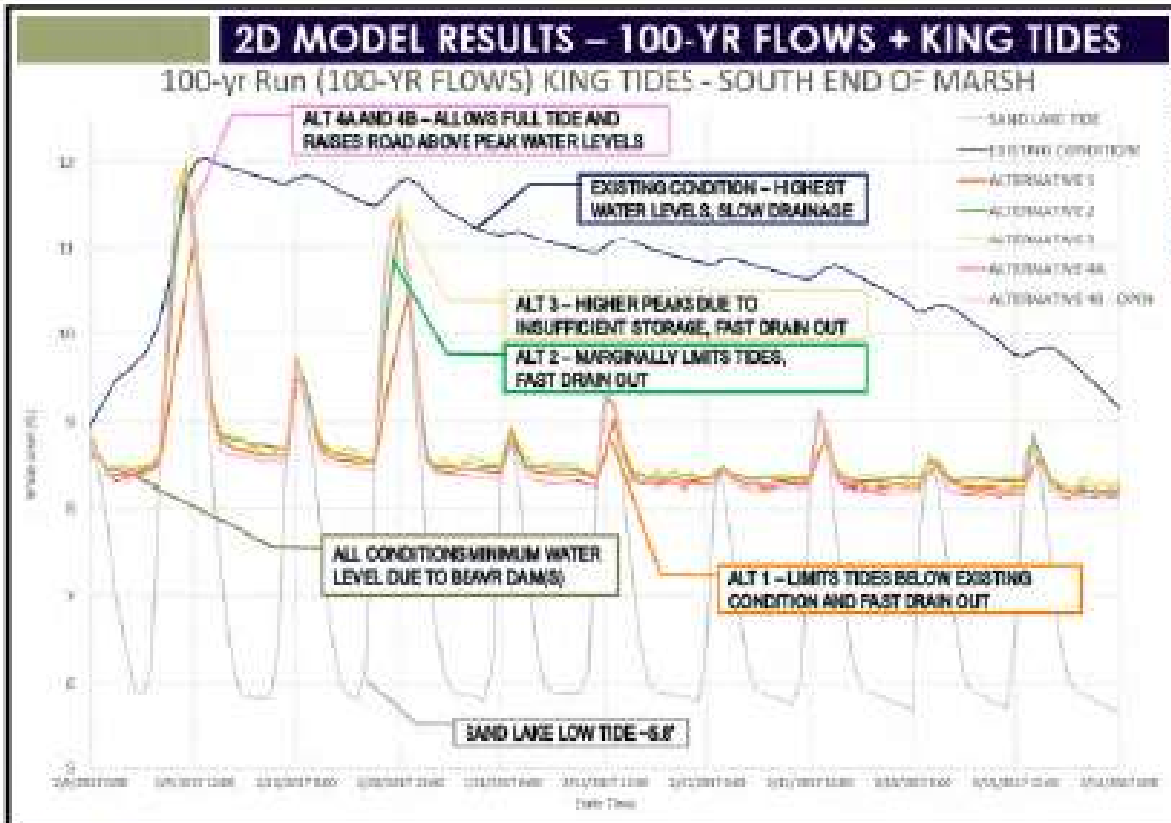
Figure 36
Modeled Water Levels at South End of Beltz Marsh During 100-Year Storm Scenario (Full Duration)



Note: *This figure shows modeled water levels at the south end of Beltz Marsh/North end of TDM for all setback dike alternatives and existing conditions.*

This plot zooms in on the 'first storm' which includes 2-year stream flows and runoff rates with moderately high tides reaching 10' NAVD88.

Figure 37
Modeled Water Levels at South End of Beltz Marsh - 2-Year Flows and 10' Tides (First Storm)



Note: This figure shows modeled water levels at the south end of Beltz Marsh/ North end of TDM for all setback dike alternatives and existing conditions.

This plot zooms in on the 'second storm' which includes 100-year stream flows and runoff rates with King Tides reaching 12' NAVD88.

Figure 38
Modeled Water Levels at South End of Beltz Marsh - 100-Year Flows and King Tides

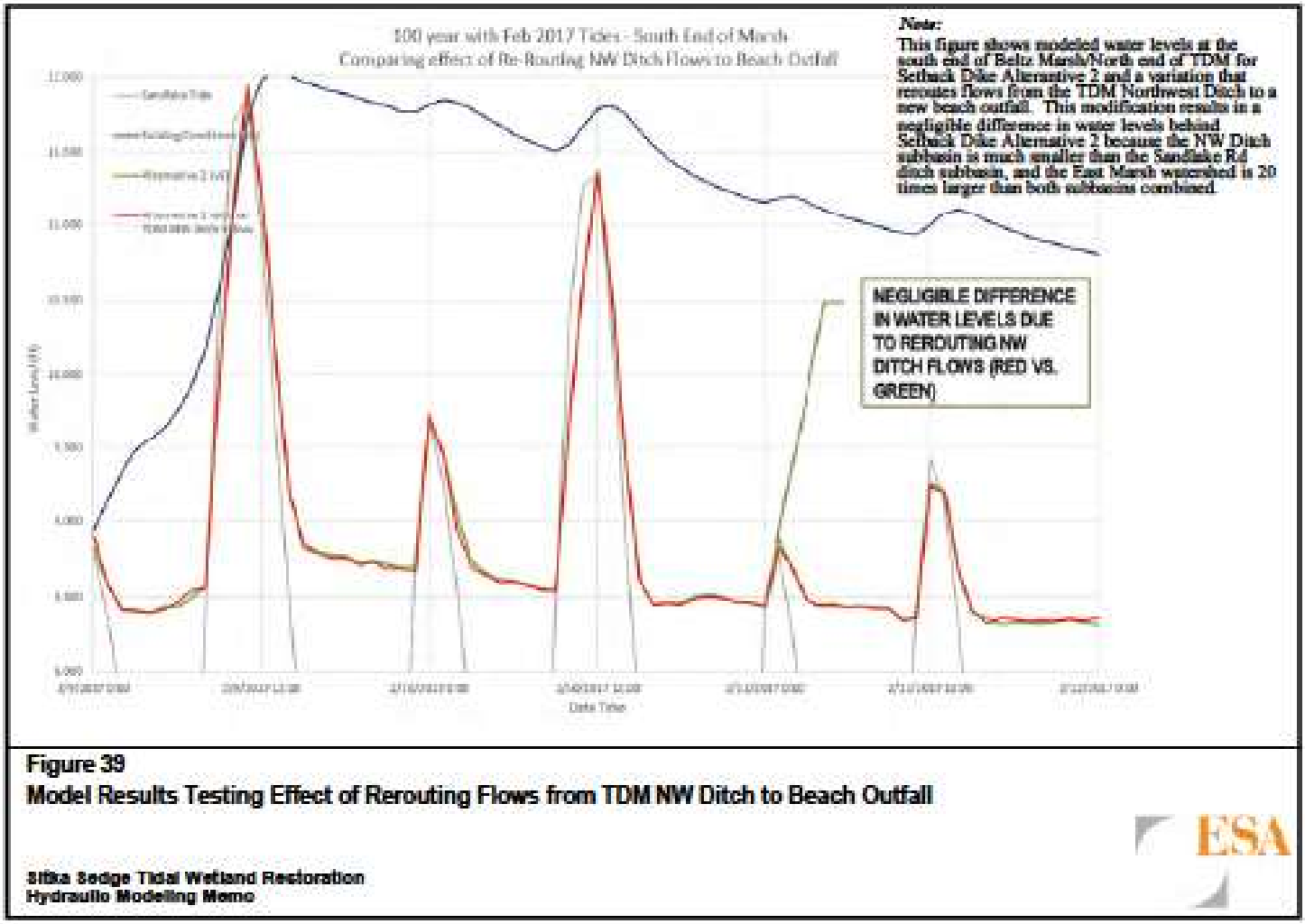


Figure 39
Model Results Testing Effect of Rerouting Flows from TDM NW Ditch to Beach Outfall

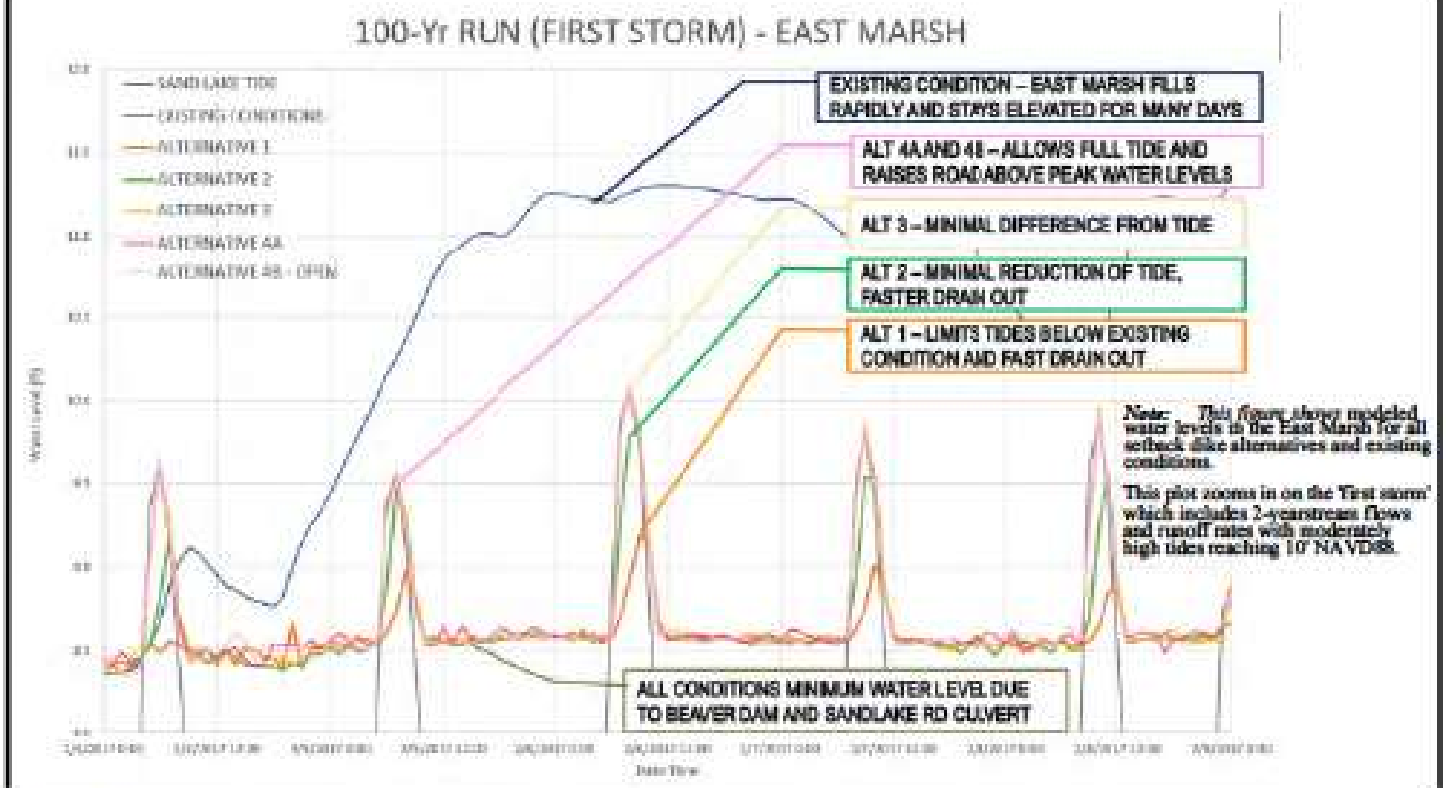


Figure 40
Modeled Water Levels in East Marsh - 2-Year Flows and 10' Tides (First Storm)

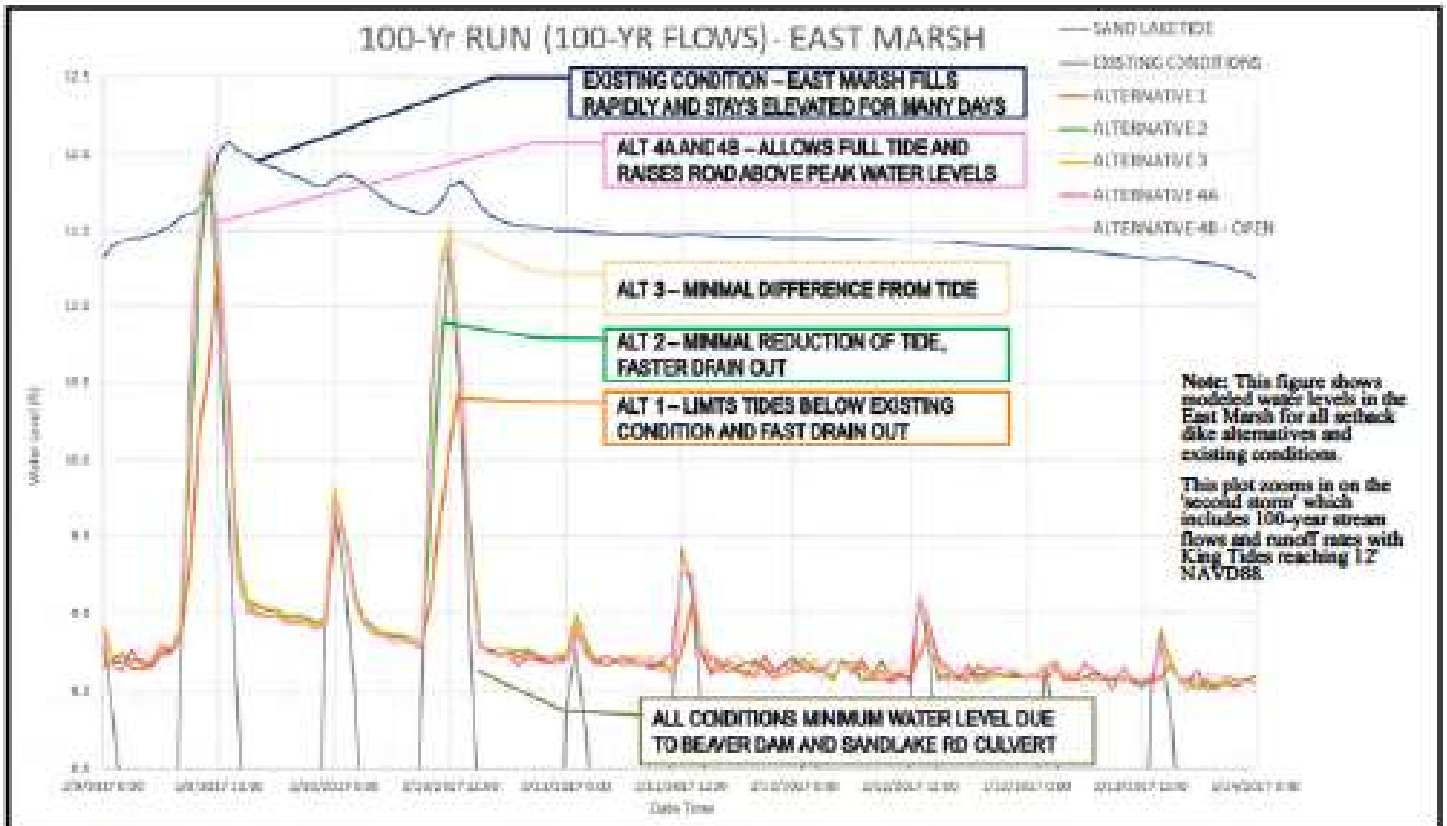
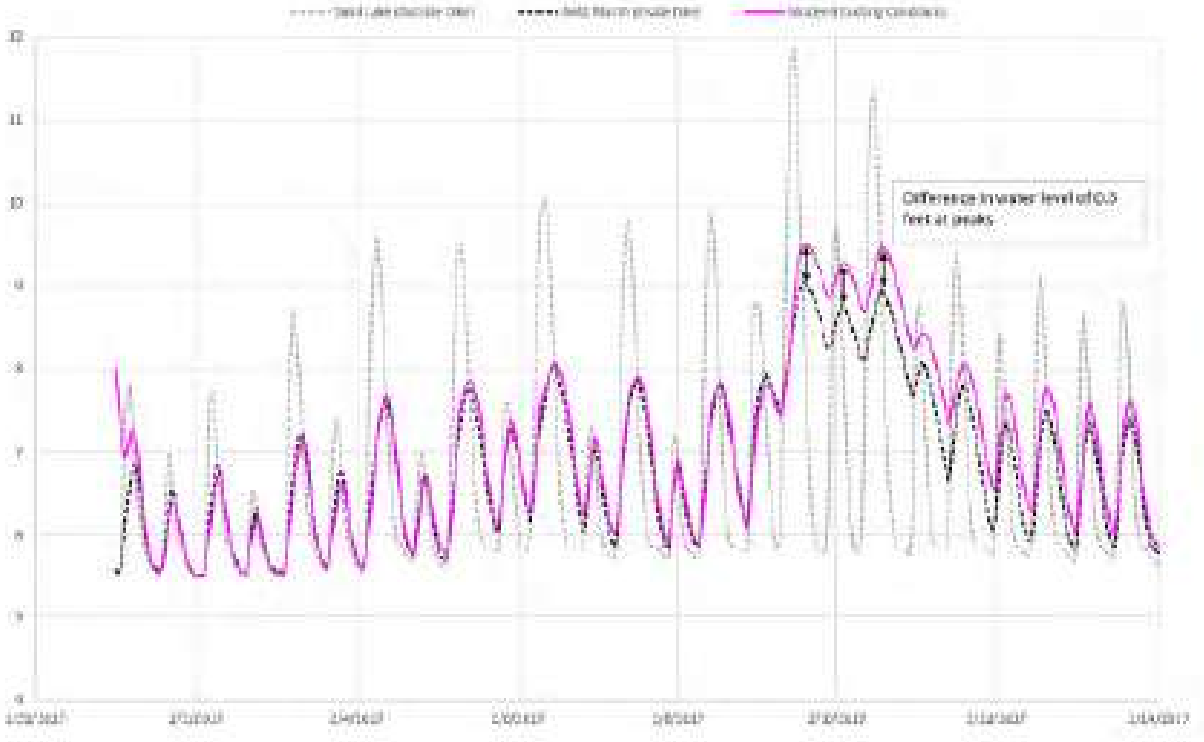


Figure 41
Modeled Water Levels in East Marsh - 100-Year Flows and King Tides



Note: This figure shows a comparison of modeled water levels to monitoring data during the 2017 storm event used for calibration. Modeled water levels generally match monitoring data very closely, except where the model predicts water levels 0.5' higher than the monitoring data, indicating the model is sufficiently calibrated and slightly conservative.

Figure 42
Model Calibration Comparison

Appendix I.
Reneke Creek Culvert Replacement – Hydraulic and Geomorphic Analysis Memorandum

Environmental Science Associates
819 SE Morrison Street
Suite 310
Portland, OR 97214
(503) 274-2010

esassoc.com

memorandum

date January 31, 2024

to Chris Laity, P.E. - Tillamook County Public Works (TCPW)

from Hunter White, P.E., Alaina Floor, P.E., and Elizabeth Schalo, P.E.

subject **DRAFT Reneke Creek Culvert Replacement – Hydraulic and Geomorphic Analysis Memorandum**

1.0 Introduction and Overview

The purpose of this memorandum is to summarize the hydrologic, hydraulic, and geomorphic analyses conducted in support for the Reneke Creek Culvert Replacement project. Analysis and modeling were performed to inform the appropriate Type, Size, and Location (TS&L) for the culvert replacement where Reneke Creek crosses Sandlake Road. This memo describes the methodology for development of the hydrologic and hydraulic models, results of the hydraulic and geomorphic analysis, and recommendations for the culvert replacement and associated channel restoration upstream of Sandlake Road.

This project is located within the Sitka Sedge State Natural Area (SSNA) at the crossing of Reneke Creek and Sandlake Road in Tillamook County. Reneke Creek has become a high priority culvert replacement for TCPW due to frequent overtopping that occurs at the crossing. The 1-square mile watershed includes forested hills in its upper watershed and a low gradient alluvial fan with a narrow riparian corridor in its lower watershed where it reaches Sandlake Road. Downstream of Sandlake Road, Reneke Creek flows through a dense and complex riparian zone before connecting with Beltz Marsh behind the Beltz Dike. Along with other contributing drainage to Beltz Marsh, including Beltz Creek, No Name Creek/East Marsh, and runoff from Tierra Del Mar, water drains through the Beltz Dike culvert to Sand Lake and ultimately to the Pacific Ocean.

This Hydraulic and Geomorphic Analysis Memo provides a basis for selecting the proposed stream/road crossing location for improvement, including:

- analysis of Reneke Creek's current and historical alignments
- manipulations to the creek channel and surrounding areas
- current geomorphic and habitat conditions in the Reneke Creek area upstream and downstream of Sandlake Road,

- hydraulic analysis and modeling of existing and proposed alternative conditions in Reneke Creek to inform fish passage design, stream channel dimensions and streambed characteristics, and safe passage of peak flows through a new crossing structure, and
- coordination with Oregon Department of Fish and Wildlife (ODFW) to gain consensus on the proposed stream/road crossing location and associated channel improvements.

Based on the analysis described herein, we recommend locating the new Reneke Creek crossing structure at the existing low point in Sandlake Road, approximately 800 feet south of the Sitka Sedge State Natural Area trailhead driveway location. This recommendation, along with preliminary channel dimensions and modeled 100-year water surface elevations expected at the crossing were conveyed to structural and roadway engineers at David Evans Associates (DEA), informing their roadway and structure alternatives analysis described in **Type, Size, and Location Memorandum – Reneke Creek Bridge at Sandlake Road (DEA, 2024).**

Additional coordination and stream restoration design will be required to refine the associated channel modifications including realigning Reneke Creek upstream of the roadway. Analysis of the potential channel alignments, coordination with ODFW, and proposed stream improvements are described in this memorandum for further coordination and design.

2.0 Reneke Creek Historical Alignment Analysis and Changes Over Time

The current alignment of Reneke Creek is shown in **Figure 1**. Once Reneke Creek emerges from the forested hillside, the creek generally flows from east to west through a narrow riparian corridor towards Sandlake Road. At Sandlake Road, the creek makes a sharp 90 degree turn to the south where it flows through an enlarged roadside drainage ditch towards the low point on Sandlake Road. At this low point, the creek again makes a 90 degree turn back to the west to flow through a 24-inch diameter culvert beneath the road and winds its way towards the Beltz Marsh in a complex system of multithreaded channels and wetlands. This forested wetland area in the lower reach of Reneke Creek contains notable beaver activity and juvenile fish use.

The alignment of Reneke Creek from its headwaters to Sand Lake has changed over time, likely resulting from geomorphic processes associated with alluvial fans, as well as anthropogenic manipulations made in conjunction with agricultural land development and drainage modifications around Sandlake Road and Beltz Dike. Alluvial fans typically form where flow emerges from a confined channel and is free to spread out across a broad area. As with Reneke Creek, this commonly occur where a stream emerges from a steep mountainous area carrying a high bedload of sediment and enters a low gradient area, such as a valley floor, where the stream loses energy and deposits sediment causing the channel to migrate across a broad area emanating from point of transition in a fan pattern. Over time, the channels within the alluvial fan may be filled in by deposited sediment, causing the stream flows to occupy new channel locations, which can result in a complex system of side channels and distributary flow paths. Based on analysis of LiDAR elevation data, it appears that Reneke Creek may have historically flowed through many locations across the broad alluvial fan towards Sand Lake. However, over the last century, Reneke Creek has progressively been channelized and constrained to the location where its main channel is located today. **Figure 2** includes a schematic representation of a typical alluvial fan (**Figure 2a**), LiDAR elevation data revealing an alluvial fan formation in the Reneke Creek area (**Figure 2b**), and existing and potential historical alignments of Reneke Creek across the alluvial fan (**Figure 2c**).

Figure 3 shows a comparison of historical aerial photos of the Reneke Creek area, highlighting the progressive modification and simplification of Reneke Creek and its riparian zone.

Figure 3a shows the stream conditions in 1939 included dense forest vegetation upstream of Sandlake Road, dense vegetation immediately downstream of Sandlake Road, and multiple sinuous channels flowing out to connect with Beltz Marsh (yellow oval). These small channels appear to be tidal channels but likely also carried Reneke Creek flows to the marsh. In this less altered condition, the forested areas upstream and downstream of the roadway likely included complex stream habitat and multithreaded channels that commonly develop in alluvial fans through a repeated process of gravel deposits filling channels and causing the creek to realign and carve new channels. This complex stream system was likely eliminated and simplified through land clearing and grading to develop the open pastures observed in the subsequent aerial photos shown in **Figures 3b and 3c**.

By 1953 (**Figure 3b**), the small sinuous channels in the lower Reneke Creek area had been filled, apparently for pasture production. Reneke Creek was rerouted around the east side of Beltz Marsh in a linear manmade channel to flow through a culvert at the eastern end of Beltz Dike. The manmade diversion channel of Reneke Creek is traced in a blue dashed line. In prior discussions with stakeholders, some have suggested that this diversion channel was Reneke Creek's original channel and should be restored. However, the aerial photo comparison does not support this assertion, and others report that this manmade channel required significant maintenance over time and ultimately failed due to blockages and gravel deposition. This channel/swale form is still present today, along with the culvert that conveyed Reneke Creek through the Beltz Dike into another linear channel towards Sand Lake. Upstream of Sandlake Road, the forested area along and south of Reneke Creek was still intact in 1953. It is unclear from the aerial photos but based on topography and common patterns observed in alluvial fans, it is likely that this forested area included multiple active and relict side channels that would have conveyed Reneke Creek typical and flood flows towards the topographic low point where Reneke Creek currently crosses Sandlake Road.

By 1977 (**Figure 3c**), the forested vegetation upstream of Sandlake Road had been extensively cleared for agricultural use, except for a narrow riparian corridor along Reneke Creek and a small triangle-shaped forested wetland in the low-lying area on the southeast side of Sandlake Road. It is likely that the land clearing south of Reneke Creek also included grading and re-shaping of the land surface for pasture development and may have eliminated any of the side channels that would have carried high flows down to the low point. Recent reconnaissance within the remnant forested wetland area revealed at least four distinct flow paths (channels/swales) that historically and currently convey some Reneke Creek flows towards the culvert crossing at the Sandlake Road low point.

By 1977, Reneke Creek had been constricted into a single-thread channel upstream of Sandlake Road, crossing the road at the location of the 'blocked culvert' identified in **Figure 1**. This culvert remains in place today. However, the culvert has been completely filled with deposited gravels, rendering it nonfunctional and directing all of Reneke Creek's flow towards the active culvert at the Sandlake Road low point. It is unclear when the blocked culvert was buried in gravels. However, a review of Google Streetview imagery from 2013 shows that at this time, the 'blocked culvert' may still have been active, but a portion of Reneke Creek flows bypassed the culvert inlet and flowed southward in the roadside ditch that carries the full flow of Reneke Creek today. These findings support the theory that some portion of Reneke Creek flows have always flowed down to the topographic low point where Reneke Creek crosses Sandlake Road today. A 2-foot diameter concrete culvert pipe situated at the low point in Sandlake Rd conveys flows beneath the roadway and out towards Beltz Marsh (SWLS, 2023). In

recent years, this undersized culvert has experienced blockages and roadway overtopping that has damaged the roadway embankment requiring repairs and frequent maintenance by Tillamook County Public Works crews.

3.0 Reneke Creek Current Conditions

Reneke Creek's current conditions (**Figure 1**) within the project area have been shaped by geomorphic processes associated with alluvial fans, historical channelization, land development for pasture production, and construction of Sandlake Road including the culverts and drainage ditches. Conditions upstream of Sandlake Road are dramatically different than the conditions observed downstream of the roadway.

Upstream of Sandlake Road, Reneke Creek emerges from the forested hillslope area and winds through a narrow riparian corridor until it reaches the blocked culvert. This portion of the channel is a gravel bed system that begins with a slope of up to 15% in the forested hillslope, transitions to approximately 3.7% through the valley, and gently flattens to approximately 2.3% when reaching the inactive culvert buried in gravels. Reneke Creek has an active channel width ranging from 12 to 18 feet. Though the creek appears to be channelized with some berms along the channel, there are a number of locations where high flows overtop the banks and can flow southward across the pasture area towards the remnant forested wetland and to the low point of Sandlake Road. These southward flow paths that activate during high flows are described in more detail in Section 6.2 (**Figure 11**).

At the blocked culvert, Reneke Creek redirects southwest, following the roadside ditch until it reaches a natural low point where it crosses Sandlake Road through a 24-inch diameter concrete culvert. The road-adjacent portion of Reneke Creek consists of an enlarged drainage ditch with a slope ranging from 1.3 to 3%. The ditch is lined with streambed cobbles and gravels carried in by Reneke Creek, some pool-riffle structure has started to develop within the straight channel. The roadside embankment has been eroded and repaired by County crews, along with some ditch enlargement to provide additional capacity and reduce damage to the embankment. There is minimal vegetation able to provide shade or cover and the water is exposed to direct sunlight adjacent to the roadway. The remnant forested wetland is located directly east of the roadside ditch, but the ditch is incised such that once water is in the ditch it rarely overtops the banks to access the adjacent floodplain/wetland.

Downstream of Sandlake Road, Reneke Creek enters high-quality wetland habitat with dense tree and shrub canopy providing full shade over the stream. In this area, Reneke Creek has several side channels and braided flow paths influenced by gravel and sediment deposition, flat gradient, and hydraulic roughness due to the dense vegetation. Beaver activity, including many small dams and channel blockages cause further braiding of the channel that appears to join Beltz Marsh in multiple locations. A large amount of juvenile salmonids have been observed in these channels during multiple site visits.

The remnant forested wetland upstream of Sandlake Road exhibits many of the same characteristics of this high-quality wetland habitat downstream of the roadway, including multiple flow paths, dense trees, shrubs, and herbaceous native vegetation. Modeling shows that high flows likely still pass through this remnant forested wetland area, but this has not been observed to date by the design team. However, even during summer time observations, the relict flow paths have saturated soils and a small amount of water in multiple locations observed slowly flowing towards the low point.

Downstream of the blocked culvert, some channel/swale forms are present, including the manmade channel that once routed flows around the east end of Beltz Dike. However, these channels are shallow and overgrown with vegetation, and appear to have been filled with sediment likely when the culvert was blocked.

4.0 Reneke Creek Hydrologic and Hydraulic Analysis

To inform fish passage design, stream channel dimensions and streambed characteristics, and safe passage of peak flows through a new crossing structure, a two-dimensional (2D) HEC-RAS hydraulic model was utilized. 2D hydraulic models use a depth-averaged approach to represent flow patterns in the horizontal plane, producing maps of depth and velocity across the simulated landscape. HEC-RAS 2D was used for this project to represent the complex, multi-directional flows between the Reneke Creek, its floodplain, and local infrastructure. ESA is currently working on the Sitka Sedge Tidal Wetland (SSTW) restoration project under separate contract with Tillamook Estuaries Partnership, which is focused on modifications to the Beltz Dike to restore fish passage and connectivity to Beltz Marsh. ESA utilized its working 2D HEC-RAS model of the SSTW project, adding detail as needed to evaluate the culvert replacement at Reneke Creek. This memorandum focuses on the portion of the model relevant to Reneke Creek.

The 2D hydraulic model requires topography, hydrology, and roughness as inputs for the model, that are represented within the model mesh. Streamflow inputs for the HEC-RAS 2D model were developed utilizing USGS StreamStats, verified by a watershed scaling of the nearest gaged stream comparable to the project creek. ESA evaluated the existing conditions model to estimate water depths and velocities, and identify overbank flow patterns under a range of flow conditions. Proposed conditions modeling was used to estimate flow depths, velocities, shear stresses under channel realignment scenarios, and analyze conditions and a new crossing structure located at the low point in Sandlake Road.

4.1 Hydrologic Analysis

The 100-year storm event was used to evaluate safe passage of peak flows through a new crossing structure. The Reneke Creek watershed does not have flow gages. A nearby gaged watershed, Tucca Creek watershed, was used to develop the inflow hydrograph. The 100-year inflow hydrograph for the Reneke watershed was scaled from the 100-yr hydrograph of Tucca Creek. The 100-year hydrograph of Tucca Creek was developed by using a flood frequency analysis to find peak flows, following the procedures of Bulletin 17B (Interagency Advisory Committee on Water Data 1982). The February 2017 Tucca Creek hydrograph was scaled to peak at the 100-year flow, and the Reneke watershed hydrograph was generated by scaling the Tuck Creek hydrograph proportionally by drainage area (USGS 2023). **Figure 4** shows the 100-year inflow hydrograph for Reneke Creek and has a peak flow of approximately 275 cfs.

4.2 Hydraulic Modeling

4.2.1 Topography

Topography for the hydraulic model was generated from the following sources in order of priority:

Type, Size, and Location Memorandum

RENEKE CREEK BRIDGE AT SANDLAKE ROAD TILLAMOOK COUNTY, OREGON DEA CONTRACT #ESCI-0004





Prepared By:
David Evans and Associates
5121 Skyline Village Loop S
Suite 200
Salem, Oregon 97306
Josh Goodall – Project Manager

Prepared For:
Environmental Science Associates
819 SE Morrison Street
Suite 310
Portland, Oregon 97202
Hunter White - Project Manager

January 2024

DAVID EVANS AND ASSOCIATES INC.

TYPE, SIZE, AND LOCATION MEMORANDUM

RENEKE CREEK BRIDGE AT
SANDLAKE ROAD
TILLAMOOK COUNTY, OREGON
DEA CONTRACT #ESCI-0004

TS&L memorandum has been prepared by:





Josh Goodall, PE
Project Manager

Date: January 29, 2024

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INTRODUCTION

Location

The project is located on Sandlake Road, mile post 8.9, in Cloverdale, Tillamook County, Oregon.

Project Purpose and Need

Sandlake Road serves as the primary north-south road west of US 101. Located just north of Tierra Del Mar, this portion of Sandlake Road is in a sag vertical curve and is regularly overtopped during heavy storms. Tillamook County intends to raise the roadway elevation of Sandlake Road to prevent overtopping and preserve access for coastal communities and visitors. To achieve this, it is necessary to increase the hydraulic capacity of the Reneke Creek crossing to carry this additional flow under the roadway instead of over it. This will be achieved by the installation of a new bridge over Reneke Creek and the raising of Sandlake Road as necessary to limit future flood events. The new structure will be designed to provide a hydraulic opening capable of accommodating high flow events.

Existing Conditions

The existing roadway is a narrow, two-lane road with an approximate -2.5% longitudinal slope (south to north) into the sag curve. The roadway consists of two 11' travel lanes with no shoulders. Utilities near the site include overhead power and telecommunication lines as well as buried telecommunication lines. Both the overhead and buried utilities are on the east side of Sandlake Road. The existing culvert is effectively flat, with a small scour pool forming at the outflow. Existing vegetation within the project limits consists of grass, small trees (Conifers, Alders, Maples), blackberry bushes, and other native plants. There are driveways to the north of the creek crossing that provide access to the Sitka Sedge Natural Area to the west and farmland to the east. There is also a private driveway just south of Beltz Creek that is expected to be outside the project limits.

ADT (2020): 795
ADT (2040): 1273
% Trucks (2020): 15%

PROJECT DESIGN ELEMENTS

Design Standards

The bridge and road will be designed to the following codes and standards:

- AASHTO LRFD Bridge Design Specifications (9th Ed, 2020)
- AASHTO Guide Specifications for LRFD Seismic Bridge Design (3rd Ed, 2023)
- A Policy on Geometric Design of Highways and Streets, AASHTO (Green Book) (6th Ed, 2011)
- ODOT Highway Design Manual (HDM) (2023)
- ODOT Contract Plans Development Guide (2016)

Construction Staging

Three options were considered for staging on Sandlake Rd. Option 1 is a full closure with detour. Option 2 is a temporary detour bridge. Option 3 is staged construction of the new bridge.

Option 1 - Full Closure with Detour:

Closing the road completely would allow a contractor to stage materials and equipment on the roadway. This option provides maximum flexibility for the contractor to complete their work and will provide correspondingly low costs. Unfortunately, there is no nearby detour for this section of Sandlake Road. The only available detour route is along US 101, a detour of approximately 20 miles. A detour of this length will result in substantial impacts on nearby communities, and could result in major delays or even safety risks if US 101 were to become blocked during construction of this project. Due primarily to substantial detour length and the role Sandlake Road plays as a main north-south connector, this Option is not recommended.

Advantages	Disadvantages
Low cost	Lengthy detour
Simplifies construction	No viable route if US 101 is blocked

Option 2 - Temporary Detour Bridge:

A temporary detour bridge would prevent a long detour and allow the contractor to stage more materials and equipment on the roadway, with the added complication of needing to provide the detour road. This option would provide greater flexibility to the contractor and, depending on the width of the detour structure (one or two lanes) limited impacts to traffic. However, with the marsh to the west and Reneke ditch to the east of Sandlake Road, this option would be difficult to build without substantial environmental impacts. The soft soils on the marsh side of the road also present challenges for construction and both sides of the road have many trees near the bridge location adding further complications. Because the road is being raised substantially, this detour would also likely have to be very long (up to 1000 feet), to avoid interfering with construction. For these reasons this option is also likely to be more costly and is not recommended.

Advantages	Disadvantages
Limits disruption to public	High cost
Simplifies construction of new bridge	Difficult to construct detour

Option 3 - Staged Construction:

The expected bridge superstructure width and type are adaptable to staged construction. The final width of 28' could provide a stage one structure width of 16'. This option removes the need for a detour route, saving travel time, or a detour bridge, likely lowering construction cost. Staged construction brings other challenges not present in the other options. The contractor will be more limited in their staging of materials and equipment and traffic will be reduced to a single lane through the construction

site. The increase in roadway height could make staging difficult as the roadway height on the first half of the staged construction will be about two feet higher than the existing roadway. It might be necessary to do some temporary widening and paving to provide adequate room to accommodate construction and traffic simultaneously. These impacts can be mitigated somewhat by constructing the new piles and cap under short-duration temporary closures before the superstructure is installed. Despite these challenges, staged construction is the recommended option as it is likely to cost less than a detour road and does not have the substantial environmental impacts of filling in the tidal marsh or creek.

Advantages	Disadvantages
No full closure	Longer project time
Lower cost than a detour road	No contractor staging on site
Fewer environmental impacts	Single lane of traffic through the site
	Possible need for temporary widening

Bridge Alternatives Assessment

Based on previous discussions with the County, the preferred bridge type has been established as a precast concrete slab bridge on pile supported foundations. Steel girders, while comparable in cost, are disadvantageous in this location due to proximity to the ocean and the salt air. Long-term maintenance costs would be higher than for a precast concrete structure. The need for a cast in place deck on top of steel girders would also increase construction time. Pile abutments are a desirable choice for this location given the soft, sandy soils that are not conducive to spread footings. Piles can be driven down into firmer substrates and can also potentially be driven behind the existing culvert, reducing the duration of construction stages. For the bridge length there are multiple options. Option one would provide the minimum opening required to comply with permitting requirements (1.5 times the active channel width of 28.5 feet plus the abutment thickness for a span length of 46 feet). This minimum length would produce the lowest cost for the bridge slabs but will require longer wingwalls to conform to existing grade. A notably longer span length (such as 56 feet) would provide additional hydraulic capacity and could be constructed essentially at grade. However, due to the relatively flat roadway through the area, and the already substantial increase in span length to provide the adequate opening for the active channel, the savings due to the reduction in wingwall length does not outweigh the increase in superstructure cost for such a substantial span length increase. A more modest span length of 49 feet will provide a balance between the superstructure cost of a longer and the length of wingwalls needed to conform to existing grade.

A prefabricated prestressed concrete slab bridge is an excellent option for this location due to the decreased maintenance cost, faster construction, and construction staging options. While this location does not yet have a dedicated geotechnical report, a boring was done as part of a potential 2015 culvert replacement project at the Reneke Creek crossing. This boring indicated that the soil conditions are very soft for at least the first 13 feet below grade. This information combined with the need to raise the existing roadway profile means that a spread footing option would need to be very deep to have

adequate bearing. This could result in the need to have cofferdams and pumping to be able to excavate to the required depth due to the water table.

The proposed length of the bridge would reduce the needed length of the wingwalls as the difference between finish grade at the face of the abutment and the shoulder slopes is minimal. It is likely that the wingwalls can be kept to approximately five to seven feet in length. This length should be sufficient to protect the bridge abutments from erosion due to potential high flows in the ditches. Wingwalls will also be placed parallel to the road to minimize both the project footprint and the amount of work that needs to be done in the ditches.

Bridge Recommendations

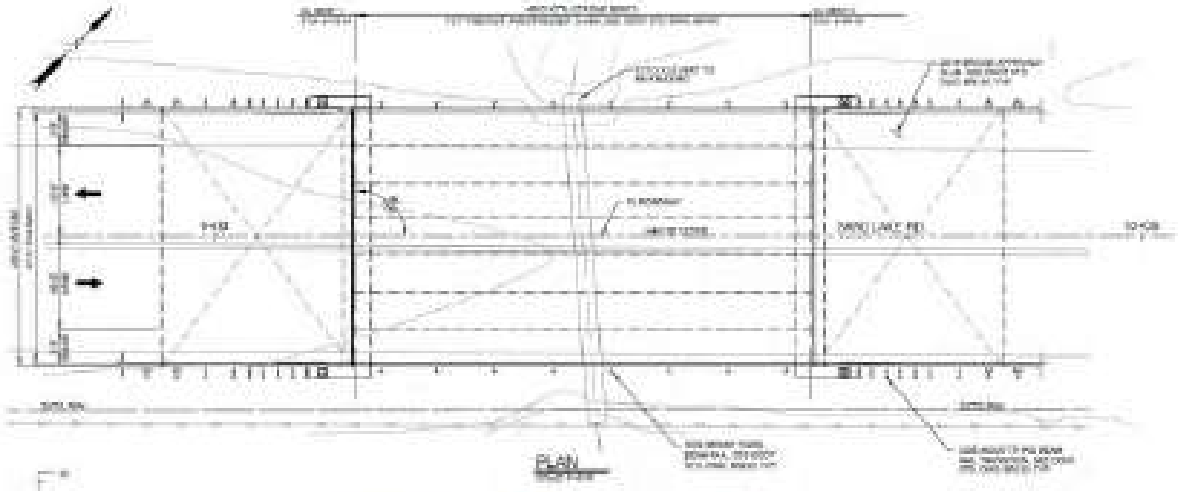


Fig. 1 – Plan View of Proposed Bridge

Superstructure

The new bridge will replace the existing culvert on Sandlake Road at the Reneke Creek Crossing. The bridge will be a 49'-0" long bridge (CL Bent to CL Bent). This length allows for shorter abutments and wingwalls. This will reduce the overall project footprint and simplify construction. The proposed bridge length will also be sufficient to provide an adequate hydraulic opening for high flow events and a channel sufficient for fish passage. The new bridge will be 28'-0" wide. Approach slabs will be placed at each end of the bridge to provide a smooth transition from the bridge deck to the roadway. The cross section will consist of two 10'-6" lanes, 3'-3" shoulders with thrie-beam railing on side mounted steel posts on each side of the bridge. A 2" asphalt concrete wearing surface will be used on the bridge deck. Additionally, an asphalt concrete wearing surface will be used on the approach slabs per Standard Drawing BR165. The bridge will be placed to match the raised roadway profile.

Substructure

The bridge substructure will consist of a pile cap abutment supported by driven pipe-piles. The piles will be driven to the depth specified by the geotechnical engineer to achieve the design bearing capacities. Cast-in-place wing walls will be placed to prevent erosion of the roadway prism.

Cost

The bridge construction costs for the recommended alternative are estimated to be $650,000. These do not include contingency, construction engineering, mobilization, or inflation.

Roadway Height Alternatives Assessment

To assess the cost of roadway raising relative to the reduction in flood risk, two minimum roadway elevations were considered: 17 or 18 feet. Based on hydraulic modeling of the bridge opening, both minimum elevations will alleviate the roadway overtopping issues and reduce road closures.

17-foot Roadway Elevation



Fig. 2 – 17' Elevation Roadway Reconstruction Extents

An increase in the minimum roadway elevation to 17 feet will require reconstruction of 515 feet of roadway; 284 feet to the north of Reneke Creek and 231 feet to the south. This alternative keeps the project construction footprint about 250 feet to the south of the Sitka Sedge State Park entrance. This alternative will provide approximately 1.5 feet of freeboard over Reneke Creek during the 100-year flood event. The initial estimate for this work is $189,000. This does not include contingency, construction engineering, mobilization, or inflation.

Advantages	Disadvantages
Smaller fill quantities	Reduced freeboard
Lower overall cost	Less room for gravel deposition
	Lower capacity for potential sea rise

18-foot Roadway Elevation



Fig. 3 – 18' Elevation Roadway Reconstruction Extents

An increase in minimum roadway elevation to 18 feet will require a roadway reconstruction of 780 feet; 375 feet to the north of Reneke Creek and 405 feet to the south. This alternative keeps the project construction footprint about 175 feet to the south of the Sitka Sedge State Park entrance. This alternative will provide approximately 2.5 feet of freeboard over Reneke Creek during the 100-year flood event. The initial estimate for this work is $275,000. This does not include contingency, construction engineering, mobilization, or inflation..

Advantages	Disadvantages
Additional 1-foot of freeboard	250 feet of additional roadway reconstruction
More room for gravel deposition	Extends shoulder closer to Reneke Creek ditch
More hydraulic capacity	Higher overall cost

Proposed Roadway Elevation

Given that the stream has a history of gravel deposition events and the relatively low increase in cost to go from a roadway elevation of 17' to 18', the 18' roadway elevation is the preferred alternative. This will provide additional freeboard resulting in a better ability to pass flows and debris in the event that the channel aggrades at the structure.

1. SWLS 1 foot. This surface was developed following a survey conducted January 27, 2023 to support the SSTW restoration project. It includes 1 small area within the Reneke Creek watershed around its southern culvert. This survey also surveyed the culvert material, shape, and invert elevations (SWLS, 2023).

2. PhoDAR. This DEM is a combination of a photogrammetrically-derived model and LiDAR data with manual adjustments. The aerial photography used in the photogrammetrically-derived model was taken on September 18, 2015. The LiDAR is dated to 2009. Manual adjustments were made in some areas of dense vegetation. However, the PhoDAR includes some artificially elevated artifacts outside of the vegetation zone adjustments (OPRD, 2017).

Additional terrain edits were performed in HEC-RAS to enforce low ground in channels as well as high ground elevations along roadways and Beltz Dike.

4.2.2 Boundary Conditions

HEC-RAS requires flow as input at the upstream model boundaries and river stage at the downstream boundary. Reneke Creek at and upstream of Sandlake Road is fluvially-dominated. Streamflow estimates for Reneke Creek were developed using scaled flows from a nearby watershed as detailed in section 4.1. The downstream boundary of the model is located in Sand Lake approximately 2,000 feet downstream of Beltz Dike, consistent with the SSTW restoration project 2D HEC-RAS model. This downstream boundary condition is tidal. The tides data used was water level data measured outside of the Beltz Dike February 1-13, 2017. This data is appropriate for use during the 100-year event scenario because of the high tides (11.9 ft NAVD and 11.3 ft NAVD) included in the dataset that are concurrent with the hydrograph peak. Tidal water level data used is shown in **Figure 5**.

Figure 6 shows the tidal datum for the South Beach tidal gage. The South Beach NOAA tide station reports a Mean Higher High Water (MHHW) of 7.57 ft NAVD and a maximum observed tide of 11.6' NAVD (NOAA 2023). Daily high tides are generally below the 9.6 ft invert elevation of the active culvert at Reneke Creek/Sandlake Road, but extreme tides and high water levels in Beltz Marsh do interact with the crossing and Reneke Creek channel to some degree (SWLS, 2023).

4.2.3 Model Layout

As noted in section 4.0, ESA utilized its working 2D HEC-RAS model of the SSTW project, adding detail as needed to evaluate the culvert replacement at Reneke Creek. The full model layout was developed for the SSTW project, then was revised for a more detailed analysis of Reneke Creek and its culvert replacement. **Figure 7** shows the full model extent with the Reneke Creek area circled. This portion of the model was extended upstream to better capture Reneke Creek inflows upstream of the broader, alluvial floodplain.

For 2D hydraulic models, the model domain is divided into a mesh composed of cells (**Figure 7**). In HEC-RAS, the water surface elevation is calculated at each model cell and velocity is calculated at the faces between cells. As such, cell size and cell face orientation are important to consider to accurately represent flow patterns within channels and the floodplain. Breaklines were added to the mesh to ensure that roads, berms, Beltz Dike, channels, and swales were properly captured within the model mesh. Additionally, the mesh resolution was assigned to 20 ft within the Reneke watershed to appropriately capture flows within channel and across the floodplain.

4.2.4 Roughness

HEC-RAS requires an estimation of the roughness of the landscape as characterized by the parameter Manning's *n*. For the 2D hydraulic model, the National Land Cover Database (NLCD) 2019 Land Cover was used (**Table 1**). (MRLC, 2021)

Table 1. Roughness categories and final Manning's *n* values

Type	Manning's *n*
Open Water	0.035
Barren Land Rock-Sand-Clay	0.03
Evergreen Forest	0.15
Developed, Open Space	0.035
Woody Wetlands	0.07
Developed, Low Intensity	0.08
Grassland-Herbaceous	0.04
Developed, Medium Intensity	0.12
Shrub-Scrub	0.05
Mixed Forest	0.12
Deciduous Forest	0.1
Emergent Herbaceous Wetlands	0.045
Developed, High Intensity	0.15
Pasture-Hay	0.045

5.0 ODFW Coordination and Conclusions

ESA met with ODFW at the project site on June 29, 2023 to review stream and habitat conditions upstream and downstream of Sandlake Road at current and previous channel locations. Proposed stream configurations and their associated benefit for fish passage and rearing habitat was discussed, along with preliminary hydraulic modeling results to indicate feasibility and sustainability of proposed crossing locations.

ODFW agreed with ESA's recommendation to place the proposed crossing structure at the southern end of the site. ODFW observed this southern location has a natural low spot for Reneke Creek to cross Sandlake Road. ODFW also observed the existing channels and their rearing habitat in the southern end of the site. ODFW expressed concern in losing that quality existing habitat if the proposed crossing was placed in the northern portion of the site and the stream was relocated through only the northern portion.

ODFW provided more extensive notes regarding channel alignment. ODFW identified the current alignment of Reneke Creek running adjacent to Sandlake Road as non-viable due to its negative impacts on fish habitat and water quality exacerbated by lack of vegetation, potential for riparian restoration, channel confinement, and disconnection from the floodplain. ODFW noted there are several options for proposed stream alignments and provided feedback on two viable alignments based on the site visit and hydraulic analysis results.

Figure 8 shows stream profiles identified by ESA based on low points in the topography. ODFW noted that a stream channel realignment following stream profile P1 in **Figure 8** may have the lowest risk of increased erosion, but would require extensive channel construction and riparian restoration efforts to construct. Additionally, that alignment would have to have viable ecological uplift. **Figure 9** shows the draft design channel alignment, "DesignChan1," proposed by ESA based on hydraulic modeling. ODFW noted that this alignment maximizes the use of the existing riparian vegetation and in-stream habitat. However, ODFW also expressed concern over possible erosion at bends, noting that it would be necessary to demonstrate that erosion would be minimal and would not require future bank stabilization. The proposed stream/road crossing location and potential associated stream improvements are discussed in greater detail below.

6.0 Proposed Stream/Road Crossing Location, Freeboard Analysis, and Potential Associated Stream Improvements

As described in Section 3.0, one existing stream/road crossing for Reneke Creek is a small, blocked culvert on the northern portion of the site. This existing crossing is non-functional as the culvert has been filled in by gravels and is undersized to pass peak flows. Reneke Creek redirects southwest through the roadside ditch to a low point and another existing culvert at the southern end of the site (active culvert). This active existing crossing location is the natural low point within the Reneke Creek area and the low point in Sandlake Road. Even prior to the northern culvert becoming blocked, overbank flows from Reneke Creek would have flowed to this low point to cross Sandlake Rd.

This southern existing crossing location (**Figure 8**, Profile P1) was identified as a viable stream realignment associated with locating the new crossing structure at the low point in Sandlake Rd. Overbank flows from many locations along Reneke Creek eventually drain to this low point. Hydraulic modeling described below demonstrates that 100-year peak flows could be passed through a new crossing structure, and freeboard could be achieved by raising the road at the existing sag curve without impacting the Sitka Sedge entrance driveway. Upon consideration of existing quality habitat in the southern end of the site and coordination with ODFW (Section 5.0), the southern crossing location was selected as the proposed stream/road crossing location. Building the new crossing structure in this location would also replace the problematic culvert and portion of the road that overtops during high water. If a different location were chosen for the new crossing structure, it would likely still be necessary to replace this culvert and repair and/or raise the roadway at another time.

Two additional potential crossing locations were identified and considered (**Figure 8**, Profiles P2 and P3). These potential crossing locations are north of the site and would redirect Reneke Creek to flow around Beltz Marsh, connecting with Sand Lake on the outboard side of Beltz Dike. Profile P3 was evaluated as a possible historical alignment on the far northern edge of the alluvial fan that would connect to an existing culvert crossing to the north. Profile P2 would run straight through the center of the alluvial fan planform. These two alignments were dismissed for several reasons including private property impacts, scale of realignment and associated riparian improvements that would be needed, the relative elevation of the channel and roadway in the location of P2, and the level of roadway and infrastructure impacts that would be needed to provide capacity and freeboard.

6.1 Proposed Stream/Road Crossing Design and Freeboard Analysis

Oregon Department of Transportation (ODOT) provides design requirements for crossing structures for design recurrence interval (storm event) and clearance (freeboard). The standard design event for culverts and bridges is

the 50-year storm event. However, ODOT notes that designs should be analyzed for the base flood (100-year storm event). ODOT also requires the 100-year storm event be used for facilities in FEMA floodplains. (ODOT, 2014) Although Sandlake Road is not located within the FEMA floodplain, the 100-year storm event is used for the design criteria for the proposed crossing as a conservative design approach.

ODOT also provides requirements for clearance under bridge structures. Clearance, also known as freeboard, is "the vertical distance between the design flood water surface and the bottom of the bridge slab, box or beam." ODOT also notes, "The normal minimum clearance is 1.0 foot above the design flood elevation or 3 feet if drift or debris is a concern…If practical, 1 foot of clearance above the 100-year flood is provided." (ODOT, 2014) Washington Department of Transportation (WSDOT) provides more stringent freeboard requirements of up to 3 feet above the 100-year storm event. (WSDOT, 2023) For a conservative design approach, ESA recommends the proposed crossing provide 3 feet of freeboard above the 100-year water surface elevation, with a minimum of 1 foot of freeboard. Hydraulic modeling results of the draft design channel alignment (**Figure 9**) at the proposed crossing location results in a 100-year water surface elevation of 13.0 feet.

A preliminary bridge cross section located at the low point in Sandlake Road, showing the estimated 100-year water surface, the existing roadway profile, and a proposed roadway profile are shown in **Figure 10**. This preliminary bridge cross section assumes a bridge deck and roadway depth of 2.5 feet, requiring an 18' NAVD88 roadway elevation to provide three feet of freeboard under the 100-year flow. This crossing location, channel opening dimensions, and peak water surface elevation was provided to DEA for the roadway and structural engineering alternatives analysis described in ***DEA's Type, Size, and Location – Sandlake Road Bridge at Reneke Creek*** (DEA, 2024).

ESA considered locating the new crossing structure at the northern existing blocked culvert location. However, the channel invert in this location is just two feet below the roadway elevation near the top of the buried culvert. Even with a full channel width of conveyance, the 100-year modeled water surface elevation would overtop the current road elevation. The roadway would need to be raised to provide freeboard inside a new crossing structure, and raising the road in this location would likely impact the adjacent driveways at SSNA. **Figure 11** shows a potential bridge cross section at the location of the existing blocked culvert. The channel in this location experiences gravel deposition and channel aggradation and may continue to require maintenance. Furthermore, overbank flows from areas upstream of the road crossing would still flow down to the low point, and a separate culvert and roadway improvement project may still be needed at the road's low point.

6.2 Potential Creek Realignment Options

The proposed stream/road crossing location at the low point in Sandlake Road has an existing stream alignment that directs Reneke Creek in a roadside ditch to the topographic low point (**Figure 8**, Profile EX). This current stream alignment is not preferred due to erosion of the Sandlake Road embankment and its poor habitat and water quality. Potential realignment options were identified and assessed using topographic assessment, hydraulic analysis, field observation, and coordination with ODFW fish biologists. The goals in realigning Reneke Creek in the vicinity of Sandlake Road include:

- Moving the creek away from the roadway embankment and out of the roadside ditch. The straight ditch channel has caused erosion on the embankment requiring channel widening and repairs. The ditch is exposed to direct sun with little to no shade provided by the roadside vegetation.

- Realigning the creek to cross the roadway more efficiently through a new crossing structure. The current right angle where the existing ditch meets the culvert is hydraulically inefficient and prone to erosion. Within the remnant forested wetland area, there is an existing channel form aligned perpendicular to the roadway, as well as multiple other flow paths that drain to the culvert and low point.

- Improving habitat conditions including instream habitat complexity and riparian vegetation for fish and other species. Extend the high-quality habitat observed downstream of the roadway to the upstream side of the road, taking advantage of relict channels, swales, and dense native vegetation. Consider broader wetland habitat restoration and reforestation on the land south of Reneke Creek's current alignment.

- More frequently activating relict channels within the remnant forested wetland area that appear to be a part of Reneke Creek's historical alluvial fan. It may be possible or preferrable to reroute Reneke Creek away from the roadside ditch to connect to one of the existing channel flow paths rather than excavating a new channel through the forested wetland. With future gravel depositions and potential beaver activity, it's likely that Reneke Creek could shift main flows into any one of these channels, or occupy multiple channels in a braided or anastomosing system similar to the channels downstream of the roadway.

Within the lower area closest to the proposed replacement crossing structure, it would be beneficial for habitat conditions to route Reneke Creek through the remnant forested wetland. On-the-ground observation within the forested wetland revealed several existing flow paths in the form of swales or remnant channels. A topographic assessment using LiDAR elevation data was conducted to analyze potential channel alignments leading into the proposed stream/road crossing. **Figure 12** shows multiple potential channel alignments following existing low points in the terrain. Lower Alignment 1 (**Figure 12**) is approximately 50 feet from Sandlake Road. This may not provide adequate buffer distance from the roadway embankment. Lower Alignments 2-4 would all provide a good buffer in moving the channel away from the roadway, and selection likely depends on the proposed upper channel realignment.

Potential upper channel alignments were identified and assessed using hydraulic analysis. Water levels, velocities, and flows were analyzed to understand where and when the floodplain activates during a flood event. **Figure 13a** shows the floodplain activates approximately midway between Sandlake Road and the tree line upstream of the cleared pasture. **Figures 13b and 13c** show floodplain activation occurs when Reneke Creek exceeds about 70-100 cfs of conveyance. This corresponds to approximately a 2-year flood event.

Potential channel realignment at or downstream of existing floodplain activation maintains existing natural flow patterns, retains part of the existing riparian corridor, and limits the extent of construction compared to a realignment starting further upstream. Two potential upper channel alignments at or downstream of the floodplain activation were assessed. **Figure 14** shows upper channel alignments A and B. Alignment B diverges from Reneke Creek's existing alignment sooner and follows the floodplain activation area. Alignment A maintains more of Reneke's Creek existing alignment and continues Reneke Creek southwestward following its natural bend. Alignment A includes more of the existing Reneke Creen riparian habitat than Alignment B. Alignment A also results in a longer Reneke Creek, providing more potential habitat area than Alignment B.

Combining the lower and upper channel realignment assessments results in a preliminary draft design channel alignment as shown in **Figure 9**. Reneke Creek stream realignment downstream (west) of Sandlake Road was considered and determined to be unnecessary due to the existing high-quality habitat and developing channel complexity at the interface of Reneke Creek and Beltz Marsh.

The details of channel realignment will be reviewed and discussed with stakeholders including OPRD and TCPW. Reportedly, additional sediment deposition occurred during a flood event in December 2023 that may be initiating a channel avulsion. Additional field observation of recent changes may influence the decision on a new channel alignment. Such sediment deposits are likely to continue occurring in the future and will need to be considered in design. In subsequent discussions with OPRD and ODFW it may be worth considering a realignment starting further upstream than the path shown in Figure 9, or at least consider lowering the south bank of Reneke Creek in some locations to allow more frequent floodplain activation and to favor future natural channel realignment further away from the roadside ditch area.

6.3 Potential Associated Stream Improvements

Along with general realignment of Reneke Creek away from the roadside ditch, additional stream habitat improvements and channel modifications should be considered, including:

- channel filling and bank stabilization at the point of diversion to prevent the channel from returning to the roadside ditch, as noted by ODFW (section 5.0).
- lowering the bank or berms along the south side of Reneke Creek to allow for more frequent floodplain activation into the pasture area to the south.
- wetland habitat and vegetation restoration in the area along the realignment which could be considered restoration of the forested conditions that existed in this area as shown in the 1953 aerial photo.
- Extending the complex and high-quality habitat conditions observed downstream of Sandlake road through the crossing and upstream of the roadway, including allowing or initiating beaver activity or beaver dam analogues.

7.0 Conclusions and Next Steps

ESA proposed the crossing location and channel realignment considerations described herein. Next steps include:

- coordination with TCPW to confirm the proposed crossing location,
- direct DEA to proceed with roadway and bridge structure preliminary design,
- coordination with OPRD to confirm channel realignment and refine associated stream improvements, and
- advance design concept to 30% Design for Stream, Roadway, and Structure including 30% Plans and TS&L Report.

8.0 References

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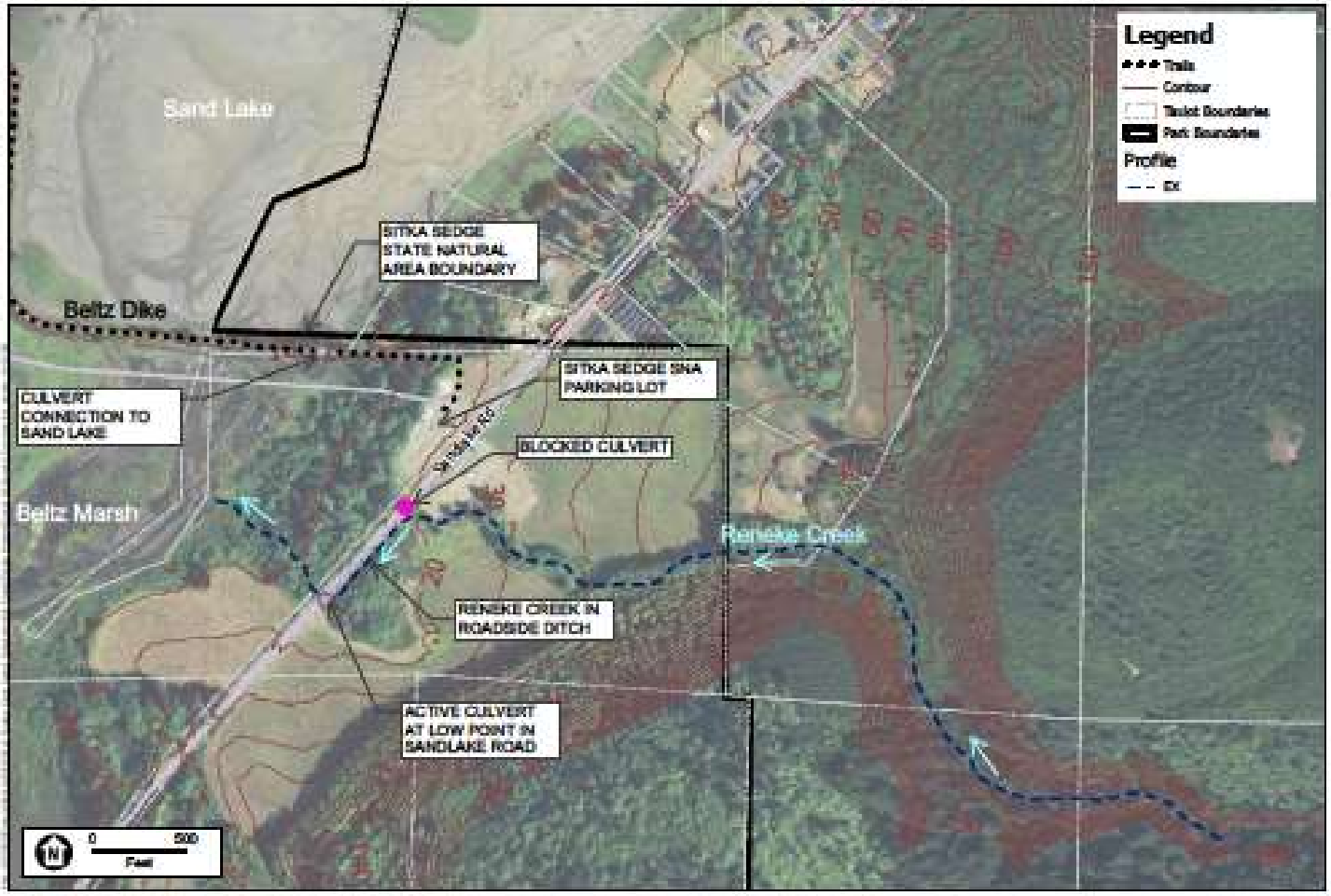
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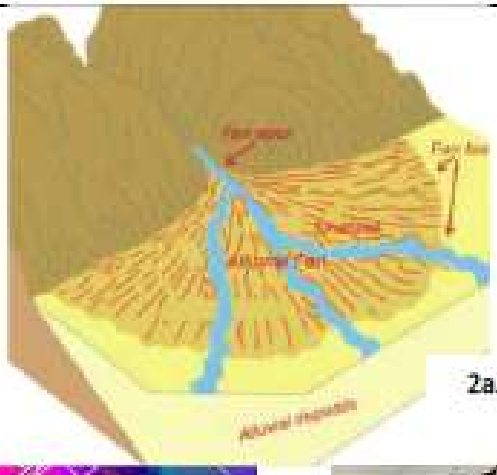


SOURCE: ESA, 2023

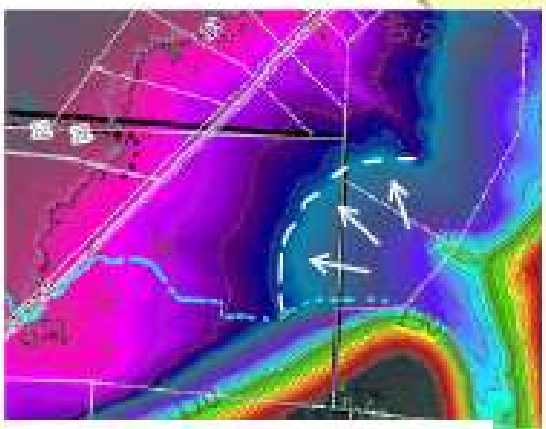
Reneke Creek Culvert Replacement

Figure 1
Renelle Creek Site Overview

ESA



2a. Schematic representation of Alluvial Fan (Ferentinou, et al, 2011)



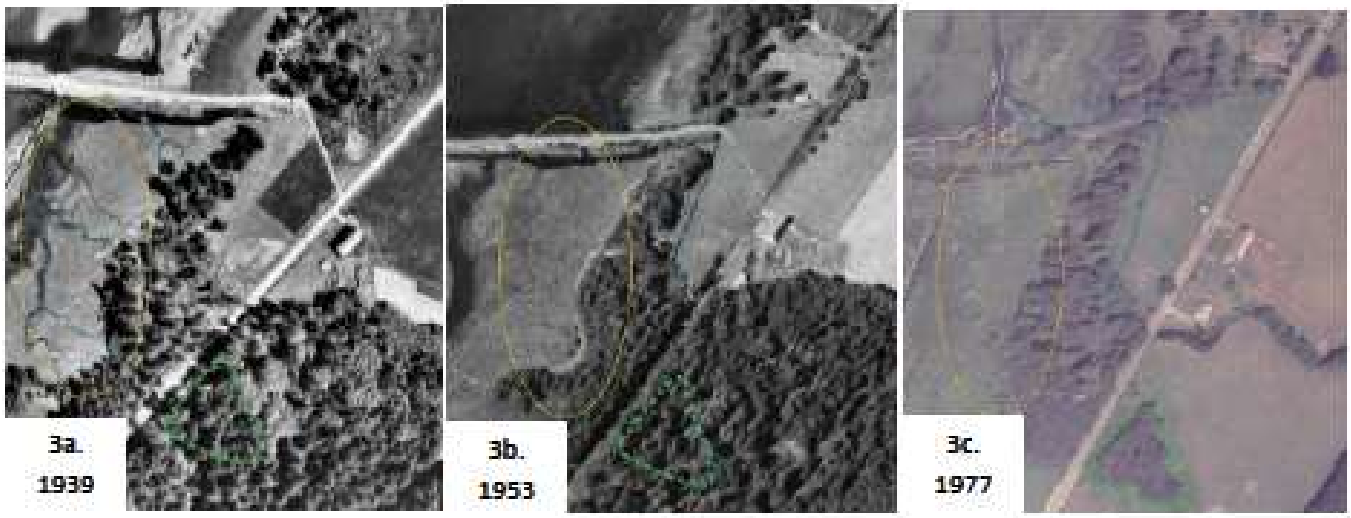
2b. Reneke Creek Alluvial Fan LiDAR Analysis



2c. Reneke Creek alignments across alluvial fan

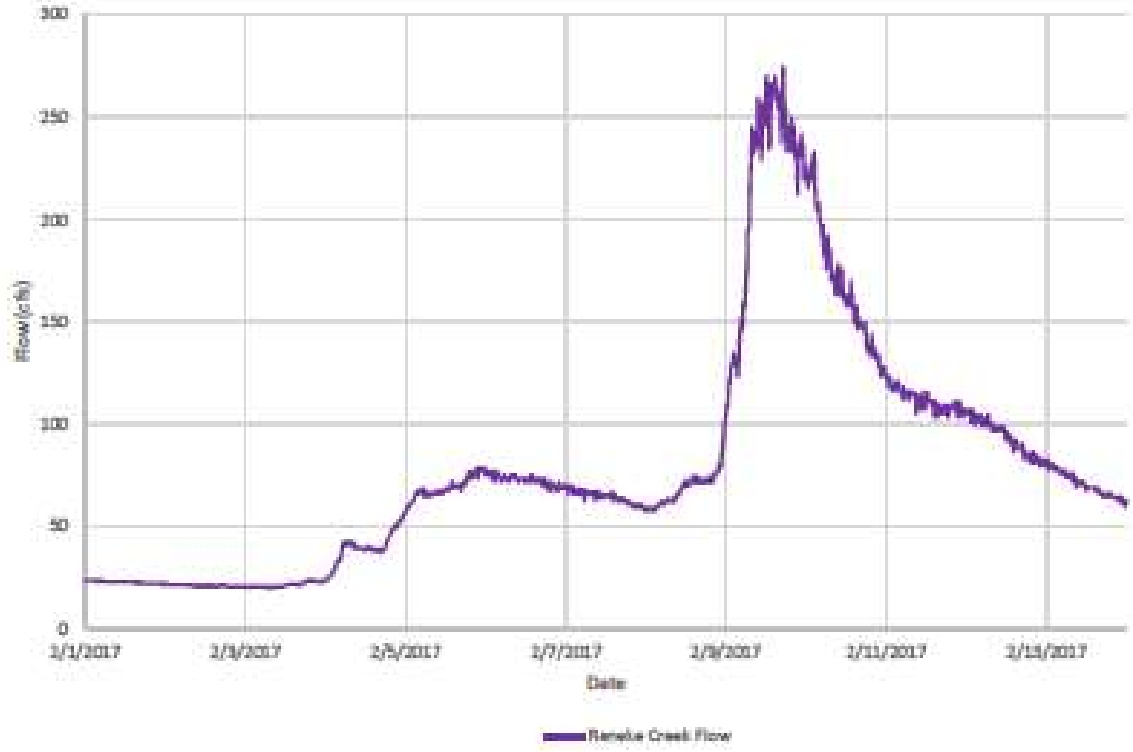
SOURCE: ESA (2025)

Reneke Creek Culvert Replacement
Figure 2
Reneke Creek Alluvial Fan Analysis



SOURCE: ESA (2023)

Renwick Creek Culvert Replacement
Figure 3
Historical Aerials

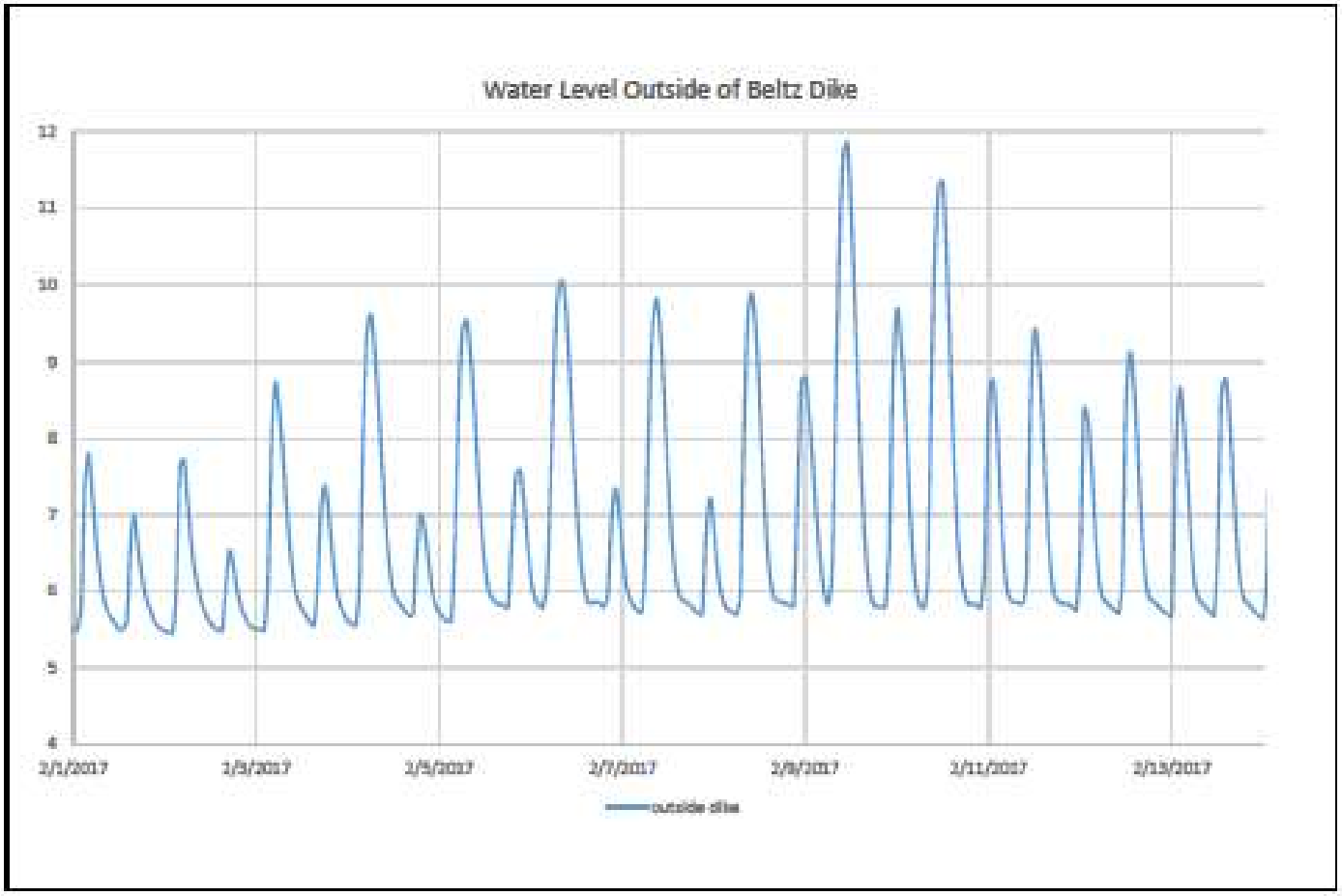


SOURCE: ESA (2023)

Reneke Creek Culvert Replacement

Figure 4

Reneke Creek Inflow Hydrograph, 100-year Storm Event

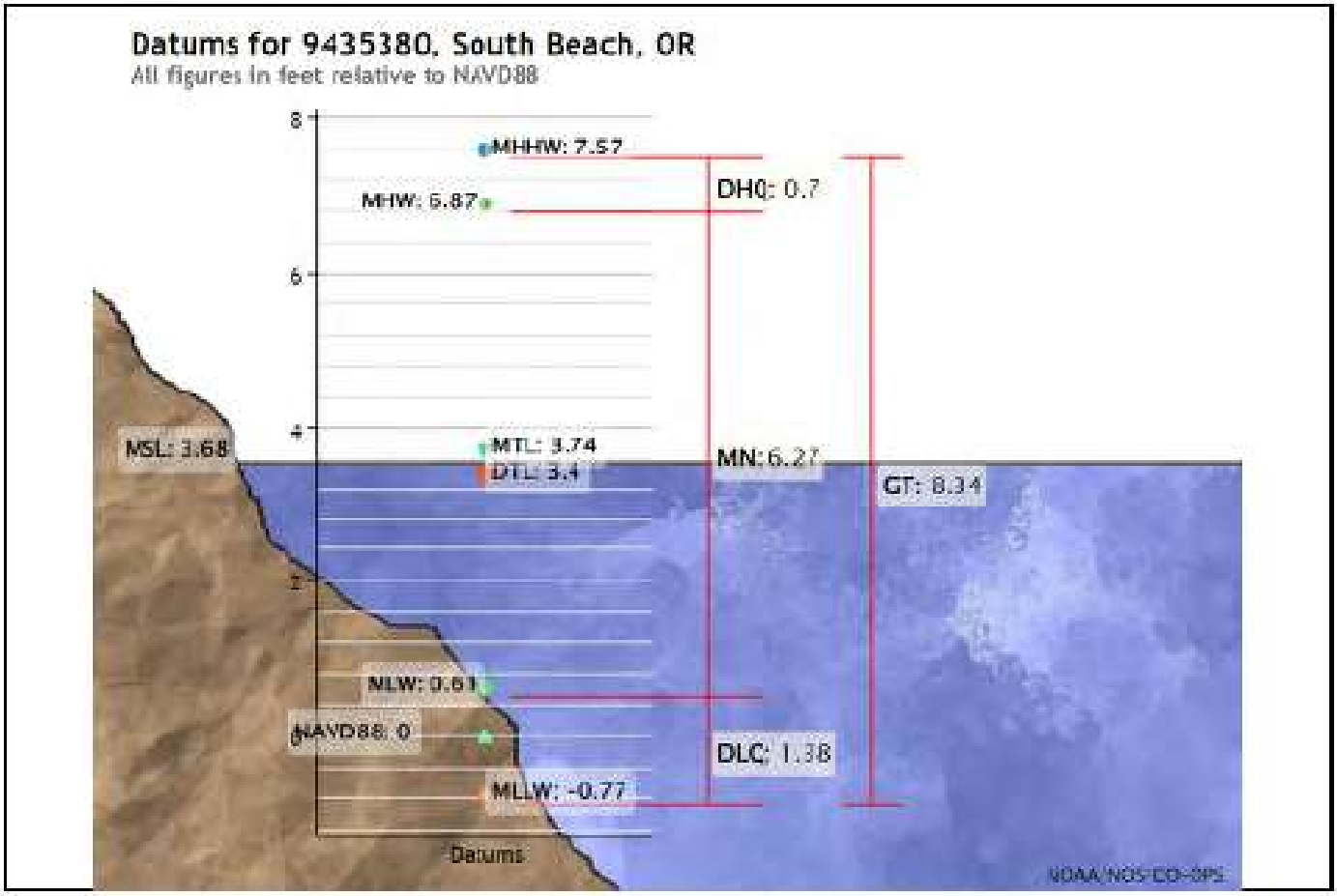


SOURCE: ESA (2023)

Renake Creek Culvert Replacement

Figure 5

Downstream Boundary Condition



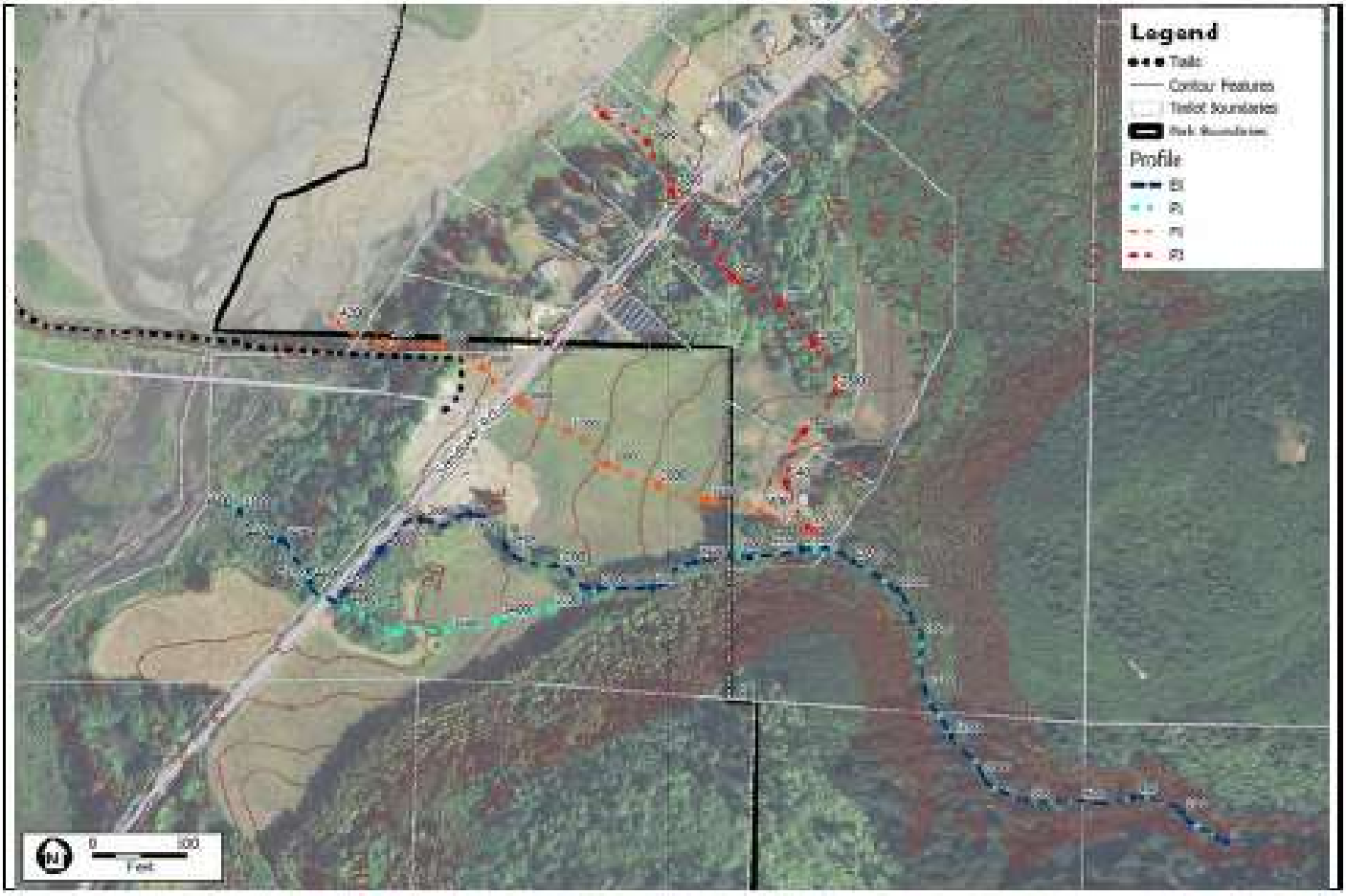
SOURCE: ESA (2023)

Reneke Creek Culvert Replacement
Figure 6
South Beach, OR Tidal Datums



SOURCE: ESA (2023)

Reneke Creek Culvert Replacement
Figure 7
Model Extent



SOURCE: ESA (2023)

Reneke Creek Culvert Replacement

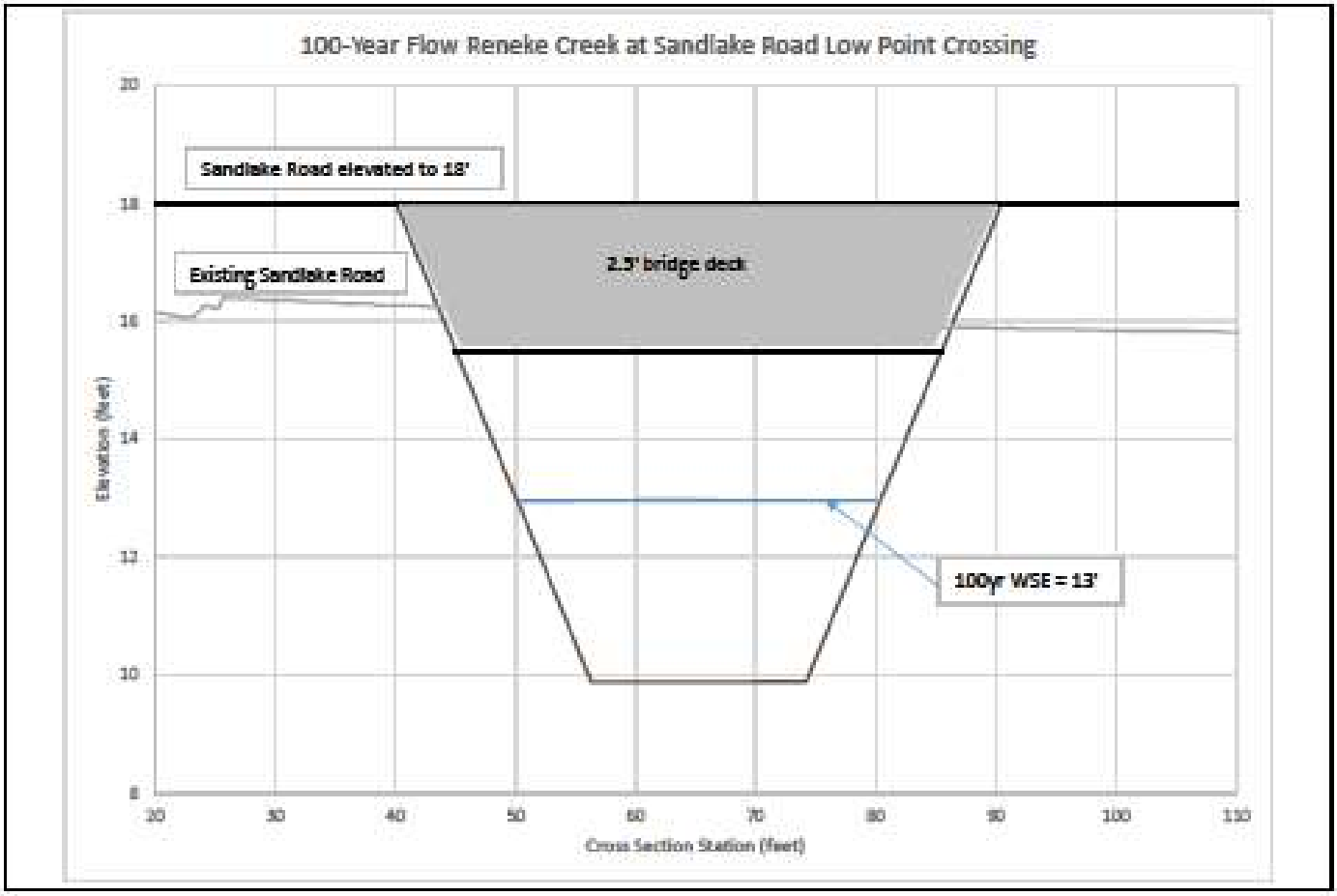
Figure 8

Reneke Creek Existing and Potential Historical Alignments



SOURCE: BSA (2020)

Rereka Creek Culvert Replacement
Figure 9
Draft Design Channel Alignment

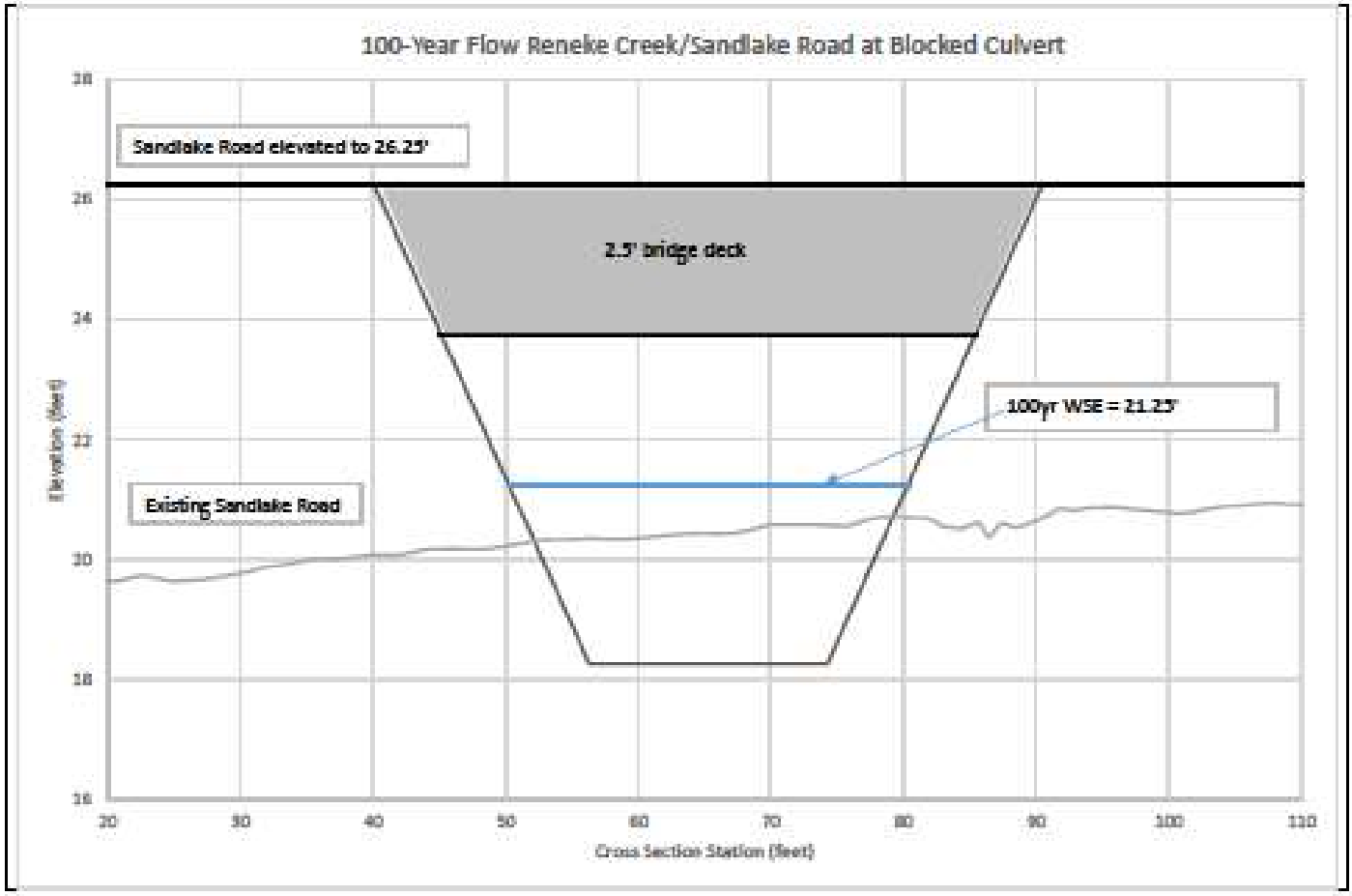


SOURCE: EGA (2025)

Reneke Creek Culvert Replacement

Figure 10

Proposed Bridge Cross Section – Modeled 100-Year Flow at Low Point in Sandlake Rd

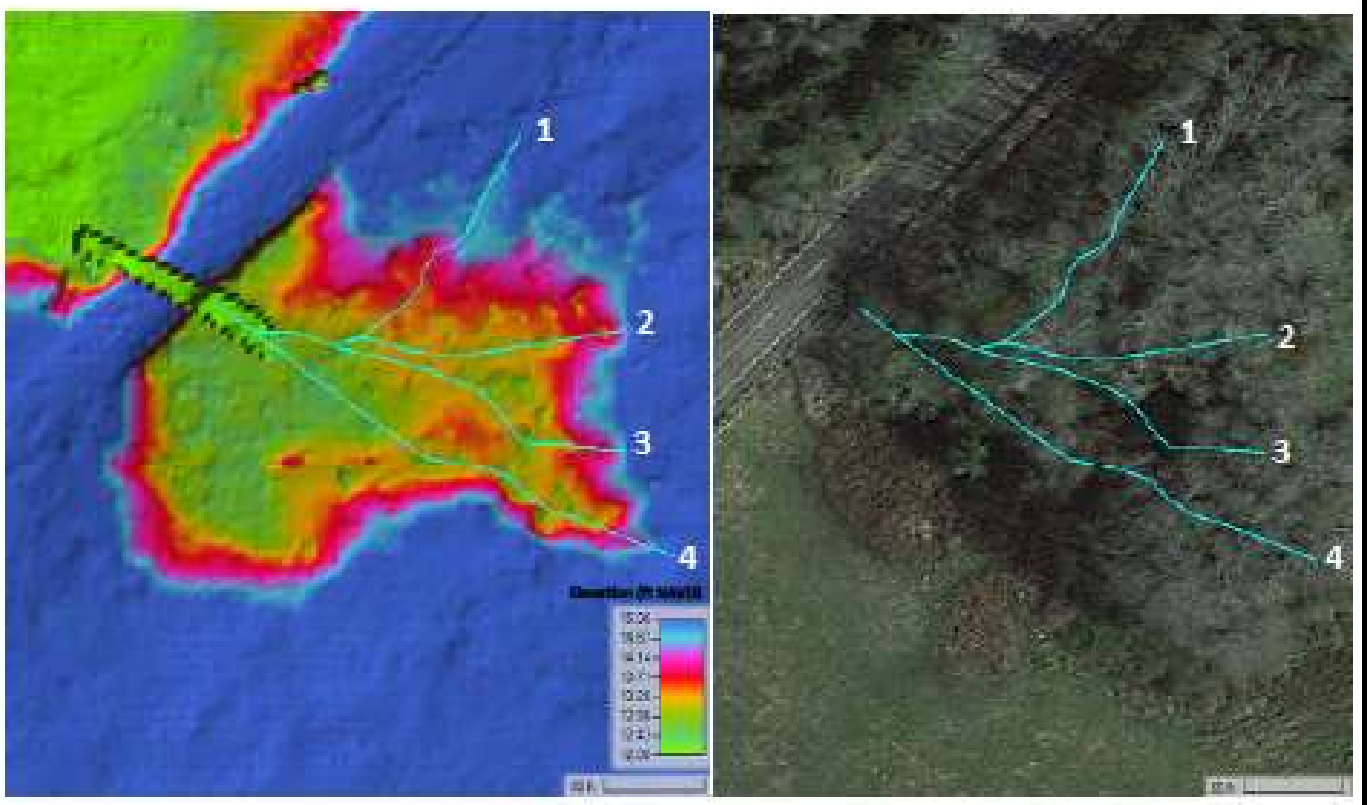


SOURCE: GSA (2020)

Reneke Creek Culvert Replacement

Figure 11

Proposed Bridge Cross Section – Modeled 100-Year Flow at Blocked Culvert Location

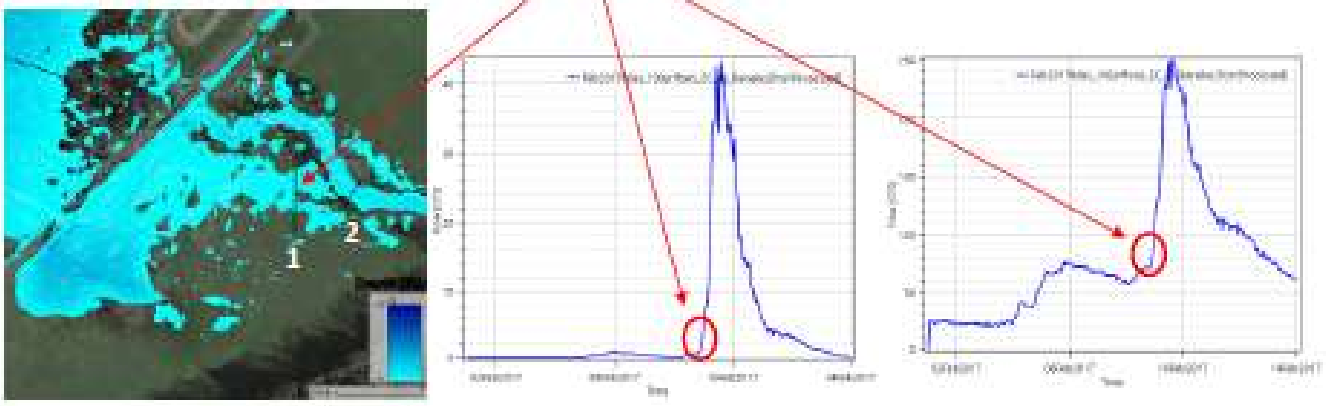


SOURCE: ESA (2023)

Renwick Creek Culvert Replacement

Figure12

Lower Channel Alignment Analysis



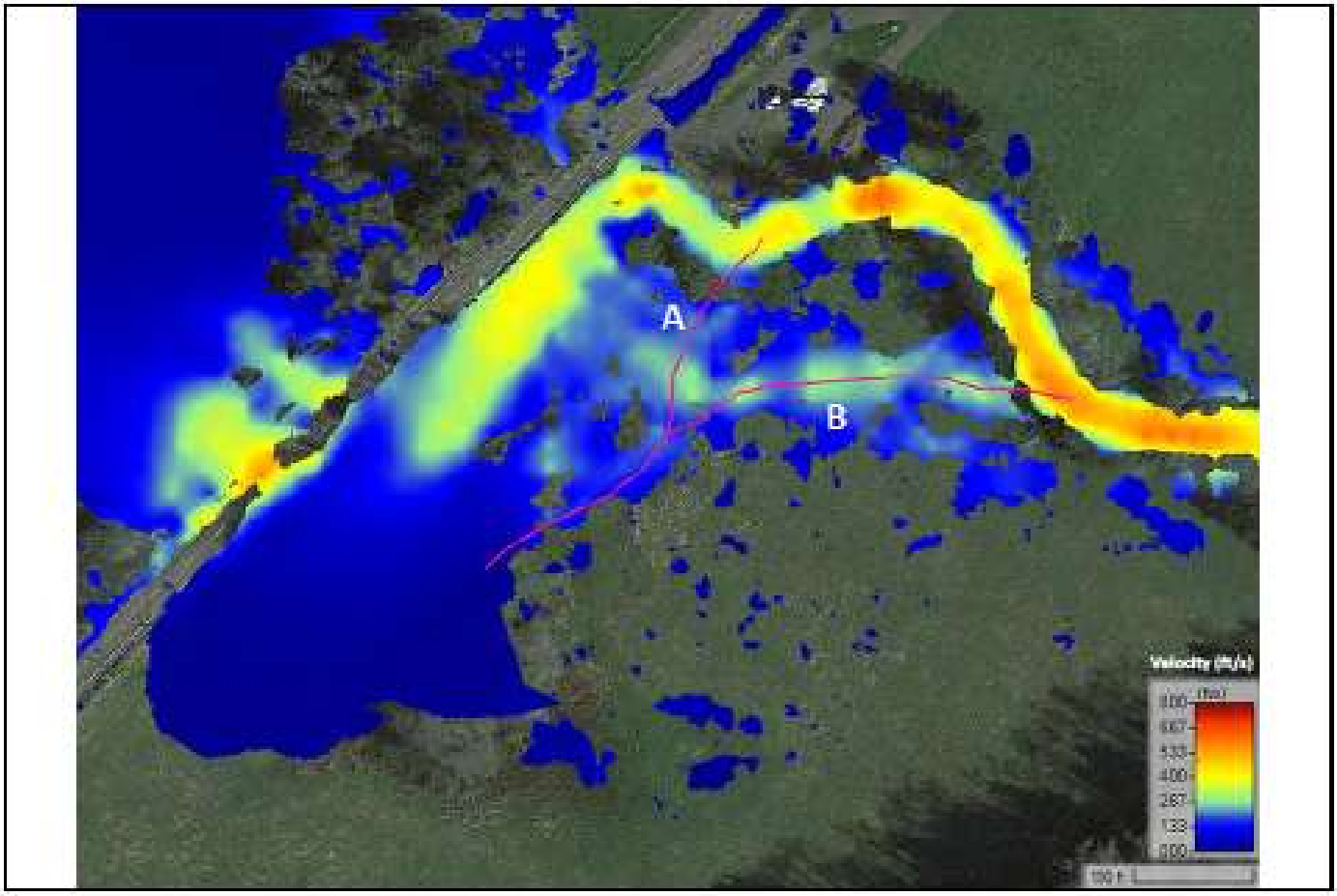
10a.
Maximum Depth

10b.
Flow at Profile 1

10c.
Flow at Profile 2

SOURCE: ESA (2023)

Renake Creek Culvert Replacement
Figure 13
Upper Floodplain Activation



SOURCE: ESA (2023)

Renwick Creek Culvert Replacement
Figure 14
Upper Channel Alignment Analysis