APPENDIX A

CONCEPTUAL GEOTECHNICAL ASSESSMENT OF CONTAINMENT BERM

Ash Creek Associates, Inc.
Environmental and Geotechnical Consultants

Memorandum

Date: August 7, 2006

To: Mr. John Verduin, Anchor Environmental, LLC
Ms. Anne Summers, Port of Portland
Mr. Marcel Hermans, Port of Portland

From: Stuart Albright, P.E., Ash Creek Associates
Steve Dickenson, PhD

Re: Preliminary Seismic and Geotechnical Evaluations
Terminal 4, Slip 1 CDF
Portland, Oregon
1165-00

We have completed the preliminary seismic and geotechnical analysis for the Confined Disposal Facility (CDF), to be constructed at Terminal 4, Slip 1, on the Willamette River in Portland, Oregon. This study incorporates over 90 years of geotechnical data available for Terminal 4 as well as the most recently available data for seismic risk. The approaches taken for the analyses of seismic hazard and dynamic soil response, as well as the static and seismic design are consistent with recent CDF designs completed in the EPA Region 10 (St. Paul (City of Tacoma, 2003) and Port of Tacoma Slip 1 CDFs (Occidental Chemical and Port of Tacoma, 2003)..

The purpose of our work was to provide seismic and geotechnical engineering assessments of the site, as well as to provide recommendations for the CDF. Our scope of work for this project included:

- A review of general geologic literature and previous geotechnical reports in the project vicinity;
- Review of past subsurface explorations;
- Geotechnical engineering analyses;
- Probabilistic seismic hazard analysis; and
- Preparation of this report.

This report includes the following:

- A description of the CDF configuration including a discussion of the materials proposed for constructing the berm;
- A probabilistic seismic hazard analysis including a discussion of general regional seismicity and site specific hazards;
- A discussion of site geotechnical conditions developed from extensive past subsurface explorations;
- An evaluation of liquefaction potential associated with design earthquakes; and

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- A berm stability analysis with cases for:
    - Short Term Static Stability (conditions representative of the CDF during the filling operations)
    - Long Term Static Stability (conditions representative of the CDF after full filling)
    - Seismic Stability (Pseudostatic and Deformation Based Procedures)
    - Post Earthquake Stability (using residual strength models)

The results of this study indicate that the CDF structure as proposed is protective of the contaminated dredged sediment placed in the CDF. The structure will adequately protect and contain the dredged material during static conditions representative of normal operations and during design earthquake scenarios.

1.0 CDF Configuration and General Assumptions

Consistent with the Engineering Evaluation and Cost Analysis (EE/CA) (BBL 2005), we have assumed that an at-grade CDF berm will be constructed at the mouth of Slip 1. The CDF will consist of three main components: (1) a CDF containment berm, (2) dredged fill, and (3) a surface layer. The berm configuration assumed for our analysis incorporates a 2H:1V slope for the inward faces. This configuration is consistent with containment berms recently constructed for the St. Paul and Port of Tacoma Slip 1 CDFs. The majority of the outward face is also constructed at a 2H:1V slope although the portion between 2.8 feet NGVD and -3.2 feet NGVD will be constructed at a 5H:1V slope. The crest of the structure will be constructed to elevation 33 feet NGVD and is assumed to be 20 feet wide. The cross section is shown on the generalized stability cross section, Figure 1. The base of the structure will be placed at depths ranging up to 10 feet below existing grades depending on depths of softer alluvium. For the majority of the berm structure, the removal of loose material will likely be less than 5 feet. We have assumed that 10 feet will be removed below the outer toe of the berm and replaced with structural fill.

Along the pierhead line just upstream of the berm will be a new structure to replace the current Berth 405 structure in Slip 1. The berth for the new structure could be dredged to elevation -46 feet NGVD in the future. Therefore, the berm is designed with this elevation at the toe. This elevation was selected to correspond with the deepest dredging requirements that are likely under current and potential uses for Terminal 4.

Behind the berm, contaminated dredged sediments will be placed to elevation 12 feet NGVD or below. Between elevation 12 and 23 feet NGVD, non-contaminated dredge sediment will be placed. The upper 10 feet of the CDF will be filled with imported granular materials.

The berm material will consist of a mix of sand and gravel. The berm will be constructed using training terraces consisting of quarry spalls or riprap, which will also serve as slope protection. An approximate gradation for the proposed imported material is given in the table below. This gradation has been used successfully at other CDF containment berms in Region 10.

Table 1.1a: Preliminary Berm Fill Gradation

Screen Size	Percent Passing
4inch	90 to 100
3/4inch	50 to 75
U.S.NO.4	35 to 55
U.S.No,10	25 to 45
U.S.No.40	10 to 25
U.S. No. 200	0 to 4

The dredged sediments will likely consist of a mixture of silts and sands. This material may be placed through a variety of methods including hydraulic placement. We have assumed that the resulting placed sediments will consist of relatively soft silts with low shear strength.

The fill to be placed over the contaminated dredged sediments will consist of granular soils. For the purposes of our analysis, we have assumed that these soils will consist of moderately silty sands.

2.0 Seismicity

In accordance with the USEPA approved EE/CA (BBL 2005) and the Action Memorandum (USEPA 2006) the CDF and the containment berm were evaluated for stability against a contingency level seismic event. The contingency level event (CLE) represents an earthquake with a 10 percent probability of exceedance in 50 years (i.e., 475-year return period). During the CLE, waterfront facilities may suffer significant damage that would impair operations, and major repair work would likely be required, but no catastrophic failure would develop. Although design components, such as a CDF containment berm, may suffer substantial deformation, containment of the contaminated sediments would not be jeopardized.

The Action Memorandum (USEPA 2006A) requires the following design-level geotechnical seismic analysis for the Terminal 4 RAA and the CDF containment berm stability:

- Detailed characterization of seismic sources (know regional faults) in the vicinity of the Terminal 4 RAA for development of a site-specific seismic hazard analysis
- Development of ground motions from seismic sources considering site-specific geotechnical considerations
- Evaluation of liquefaction potential for CDF containment berm, foundations soils, dredge sediment, and surrounding site soils potentially contributing to instability of the CDF during the design-level earthquake, including evaluation of liquefaction-induced deformations and lateral spreading
- Evaluation of slope stability and deformation for both pseudo-static and post-earthquake conditions
- Development of contingency plan for post-earthquake inspection and repair.

2.1 Regional Seismic Setting

The seismicity of the Portland Metropolitan area, and hence the potential for ground shaking, is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone. Descriptions of these potential earthquake sources are presented below.

The CSZ is located offshore and extends from Northern California to British Columbia. Within this zone the oceanic Juan De Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between the two plates is dipping to the east, and therefore becomes deeper toward Portland. At the easternmost portion of the interface zone that is thought to be capable of generating strong ground motions the interface between these two plates is located at a depth of approximately 20 kilometers (km). Quantifying the seismicity and hazard posed by the CSZ is subject to several uncertainties, including the size of the maximum credible earthquake as described by the moment magnitude (M_w) of the event, the rate of seismicity associated with CSZ earthquakes of various magnitudes, and the nature of the ground motions associated with CSZ earthquakes. (Moment magnitude is used by seismologists to measure larger earthquakes and is based on fault displacement and area of fault rupture.) Geologic evidence of previous CSZ earthquakes has been observed within coastal marshes along the Oregon coast and in off-shore landslide deposits (turbidites). This paleoseismic data has been used to infer the size of pre-historic earthquakes as well as their rate of recurrence. Sequences of interlayered peat and sands have been interpreted to be the result of large ($M_w > 8$) subduction zone earthquakes occurring at intervals on the order of 300 to 500 years with the most recent event taking place approximately 300 years ago.

The intraplate zone encompasses the portion of the subducting Juan De Fuca Plate located at a depth of approximately 30 to 50 km below western Oregon. Very low levels of seismicity have been observed within the intraplate zone in Oregon. However, much higher levels of seismicity within this zone have been recorded in Washington and California. Several reasons for this seismic quiescence were suggested by Geomatrix (1995) and these include changes in the direction of subduction between Oregon and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone includes the 1949 Olympia M_w 7.1, 1965 Puget Sound M_w 6.5, and 2001 Nisqually M_w 6.8 earthquakes. Based on the data presented within the Geomatrix (1995) report, an earthquake of M_w 7.25 has been chosen to represent the seismic potential of the intraplate zone.

The third source of seismicity that can result in ground shaking within the greater Portland area is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in western Oregon is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (M_w 5.6) and Klamath Falls (M_w 6.0) earthquakes are examples of rather shallow crustal earthquakes.

2.2 Site Specific Seismic Hazard

We completed a Probabilistic Seismic Hazard Evaluation (PSHA) using the most up to date information from agencies such as the United States Geological Survey (USGS), Department of Geology and Mineral Industries (DOGAMI), and the Oregon Department of Transportation (ODOT). This information has been supplemented with seismic hazard data from numerous other technical resources. On the basis of the PSHA analyses we have defined the two primary seismic sources considered for design purposes to include: (1) a Mw 9.0 mega-thrust earthquake along the CSZ having a source-to-site distance of roughly 100 km; and (2) a Mw 6.2 shallow, crustal event with a source-to-site distance of 14 km. The relative contributions of the two closest faults, the Portland Hills Fault and the East Bank Fault, to the cumulative seismic hazard are so small for the return period of interest (475 years) that their contributions would not have a significant affect on the cumulative seismic hazard. In light of the low slip rates and corresponding low rates of seismicity estimated for these faults, and based on input from DOGAMI personnel who are actively studying these faults (Madin, 2006), these two potential seismic sources have not been incorporated in the current analyses. The design team has selected the following scenarios for subsequent analysis of dynamic soil response, soil liquefaction, and design for the CDF berm:

1. M_w 9.0 CSZ event resulting in bedrock ground motions of 0.14g beneath the site.
2. M_w 6.2 crustal source resulting in bedrock ground motions of 0.20g.
3. The intraslab (or intraplate) source has been shown to contribute the least to bedrock peak acceleration and spectral accelerations (0.2 and 1.0 second), and therefore omitted from further consideration in our analyses.

3.0 Geotechnical Conditions

We reviewed the contents of 24 geotechnical reports prepared for past projects within Terminal 4. These data were screened for applicability to the project relative to proximity and exploration methodology. Over 80 borings and 10 cone penetrometer tests (CPTs) were included in this review. Of the borings reviewed, 11 were found to have been within the general CDF area and completed with modern drilling equipment. The most significant data available from the borings consisted of standard penetration test (SPT) blowcounts. The SPT test results were summarized and corrected for rod length, overburden pressure, and hammer efficiency. For all corrections, mid range values as recommended by the Federal Highway Administration (FHWA) were utilized.

The following soil units were encountered in the explorations reviewed:

Loose to Medium Dense Sand Fill. In general, the upland areas adjacent to the CDF were constructed of loose to medium dense sand fills. The thickness of the fill layer ranges from approximately 17 to 35 feet. Gradation testing of the sand fills indicates fines contents ranging from approximately 5 to 15%.

Soft Surface Sediments. The floor of Slip 1 is covered by soft clay, silt, and sand sediments. Based on the sediment cores completed for the EE/CA, the soft sediment layer generally ranges from about 0 to 3 feet in thickness.

Medium Dense Sand. The majority of Slip 1 is underlain by a dark grey, medium dense to dense, medium to coarse sand. This sand is consistent with Willamette River alluvium. Based on past laboratory testing, the fines content of this sand ranges from 3 to 8%. The upper 5 to 10 feet of this formation can range to loose, likely owing to ongoing alluvial processes. Below this disturbed material, the density of the sand is relatively uniform. Based on a review of 138 corrected SPT values, the average blowcount value obtained in this formation was 21 blows per foot (bpf) with a standard deviation of 9.3 bpf. The distribution of blowcounts indicates little to no variation with depth. Only one SPT sample had a measured blowcount of less than 10 (indicative of loose sand) and seven samples had blowcounts of more than 30 (indicative of dense sand). With very little variation, this formation can be modeled as a medium dense, relatively clean sand.

Troutdale Gravel. Dense, partially cemented deposits of gravel and sand were encountered below the alluvial sands. This deposit likely consists of the Troutdale Formation.

4.0 Dynamic Soil Response and Surface Ground Motions

The results of the probabilistic seismic hazard evaluation outlined in Section 2.2 and subsequent application of empirical procedures for estimating the characteristics of strong shaking during the design scenario earthquakes are

applicable for bedrock conditions. The potentially significant influence of soil deposits on the characteristics of the ground motions must be evaluated for analyses of geotechnical hazards associated with soil liquefaction and slope stability. Dynamic soil response analyses have been performed for three critical sections along the CDF berm; (1) centerline, (2) mid-slope at the location of the bench, and (3) at the toe of the berm. The geotechnical profile of soils and underlying rock materials was constructed after review of pertinent reports at Terminal 4, local sites along the Willamette River, and regional geologic mapping. Dynamic soil properties were obtained from in situ data at the Terminal 4 site, regional seismic hazard mapping projects performed by or for DOGAMI, or estimated from correlations with other geotechnical parameters such as SPT or CPT penetration resistances.

The three soil profiles selected for analysis were modeled using Pro-Shake, a commercially available software package based on the widely used program SHAKE (Schnabel et al, 1972). The soil and rock profiles included the soil layers outlined in Section 2.2 as well as the underlying deposits of Troutdale gravel, Sandy River mudstone, a weathered section of Columbia River basalt, and a base layer of relatively intact and fresh Columbia River basalt. A suite of bedrock ground motion time histories was selected on the basis of the source mechanism and magnitude of the event, source-to-site distance, as well as the intensity, duration, and frequency content of the individual motions. All motions are considered representative for the design earthquake scenarios (Mw 6.2 with source to site distance of 14 km, and Mw 9.0 at 100 km distance). Three acceleration time histories were selected for the Mw 6.2 crustal event, and 5 time histories were employed for the Mw 9.0 event (3 actual recordings and 2 synthetic motions generated for CSZ earthquakes).

The dynamic soil response analyses demonstrate that all three of the soil profiles lead to an amplification of bedrock ground motions. The computed peak ground surface accelerations for three sections of the CDF berm are listed in Table 4.1a. These PGA values reflect the average of three analyses for the design crustal event and five analyses for the Cascadia subduction zone event. The dynamic soil response analyses have been performed using a 1D, total stress model therefore the influence of sloping ground conditions on the site response has not been accounted for. The results of the 1D SHAKE analyses will be modified empirically to approximately account for the 2D nature of the CDF berm. This empirical adjustment may increase the PGA values by roughly 10% to 20% along the bench and crest centerline sections of the CDF berm. The ground response along the toe of the berm will not be significantly affected by 2D ground motions effects. The amplification ratios (ground surface/bedrock) for each earthquake scenario have also been evaluated as a function of frequency via the acceleration response spectra. The maximum spectral amplification ratios ranged from 3.0 to 3.5 at periods of 1.0 to 1.5 second. The resulting ground motions were used in analyses of soil liquefaction hazards and slope stability.

TABLE 4.0: Computed Peak Horizontal Ground Accelerations

Seismic Source	Peak Horizontal Ground Acceleration (g)			
	Bedrock	Toe of Berm	Bench	Centerline of Berm Crest
Shallow, crustal faulting	0.20	0.33	0.30	0.31
CSZ mega-thrust event	0.14	0.20	0.18	0.17

5.0 Liquefaction Potential

The potential for soil liquefaction during seismic ground shaking is generally associated with loose to medium dense, saturated, sands and very soft, recently deposited, non-plastic silt soils.

Loose sandy soils are present at the mudline in several locations and it is our recommendation that a portion of these liquefiable soils be dredged from the berm footprint area prior to the placement of the CDF berm. Localized lenses of liquefiable soil are probable; however, based on currently available *in situ* geotechnical data, it does not appear that these lenses are vertically and laterally continuous.

Preliminary liquefaction analyses were performed based on standard penetration test (SPT) values available from past geotechnical explorations completed in the vicinity of the CDF. This data was supplanted by CPT data available from the EECA. Ground motions and seismically induces stresses were based on the results of our dynamic site analysis detailed in the preceding section.

Liquefaction modeling was completed using the "simplified procedure" presented in Youd et al. (2001). We calculated factors of safety against liquefaction for all materials above the Troutdale Formation. The analysis was completed for three locations, under the crest of the CDF berm, midway between the crest of the berm and the toe of the berm, and beneath the toe of the berm.

The results of our liquefaction analyses indicate factors of safety of less than 1 (indicative of liquefaction) for much of the upper 35 feet of soil under the toe of the containment berm. For the midpoint analysis, factors of safety of less than 1 were indicated for the upper 17 feet of native sand. The majority of this material beneath the berm is anticipated to be removed prior to constructing the CDF. No indication of liquefaction was indicated for under the crest of the berm.

Although some of the soils present on the site appear to be susceptible to liquefaction, this does not preclude the CDF from safely containing the dredged sediments. The results of our liquefaction analyses were incorporated in our stability analyses which indicate that the berm would remain intact in spite of liquefaction under the outer toe. The post earthquake stability model is discussed in detail in section 6.5.

6.0 Containment Berm Stability

6.1 Methods of Stability Analysis

A number of typical cross sections through the berm were developed and analyzed for deep-seated, global stability. Based on the preliminary analysis, the cross section through the middle of the berm was determined to be the critical section (possessing the lowest factors of safety).

Stability modeling was conducted with GeoSlope's software package SLOPE/W. The software employs a limit equilibrium methodology for calculating a factor of safety against sliding or sloughing. The analysis was completed using Spencer's method which satisfies both moment and force equilibrium.

The cross section was evaluated for four cases: (1) short term static; (2) long term (post-filling) static; (3) long term (post-filling) seismic, and (4) post earthquake static. For each case, we evaluated the slope stability factor based on a random search of circular slip planes. We also searched through the calculated slip planes that pass through the contaminated dredged sediment to determine which of these have the lowest factor of safety. These are referred to as the shallow slip plane and the deep slip plane, respectively. The deep slip plane represents a deep seated

stability failure that might result in release of contaminated sediment. A graphical representation of the results of each of these analyses is shown on Figures 2 through 5.

Preliminary soil parameters used in the analyses were developed based on the results of the geotechnical review. SPT blow counts, CPT cone pressure values, laboratory strength testing, and gradation data were used in concert with published references to develop preliminary strengths and unit weights. Statistical distributions were applied to each value based on a subjective evaluation of the potential variability of assumed and measured data.

For the static and pseudostatic analyses, drained strength parameters were used. These values are appropriate for cases where due to soil permeability, sufficient time exists for pore pressures to dissipate. The strength of the native soils was developed based on correlations with corrected SPT blow counts and CPT cone pressures. The strengths of other soils such as the berm and dredged materials were estimated based on anticipated quality and densities of these soils. The values assumed for non-native soils are comparable to assumed values used in designing previous CDF facilities in EPA Region 10 (City of Tacoma, 2003; Occidental Chemical and Port of Tacoma, 2003). A summary of drained soil parameters employed in our analyses is presented in the following table.

The post-earthquake static analyses were performed to evaluate the margin of safety that may exist during and immediately after seismic loading during the design level events. The shear resistance of the soils was modified to account for the generation of excess pore pressure and possible degradation of strength during cyclic loading. The results of these analyses will be used in subsequent deformation analyses of the CDF and foundation soils.

Table 6.1: Drained Soil Properties, Global Stability Assessment

Soil Unit	Moist Unit Weight (pcf)	Cohesion c' (psf)	Internal Friction Angle (phi')
Berm Fill	135	0	35
Contaminated Dredged Material	120	0	24
CDF Fill	120	0	32
CDF Surface Layer	125	0	35
Native Sands	120	0	35

6.2 Short Term Static Stability

The critical section for the short term static stability reflects the conditions present during filling of the CDF when the entire CDF may be used to decant hydraulically dredged sediments. Our analysis was based on the most critical case for this condition, with the contaminated sediment placed, the water in the CDF to within 2 feet of the crest of the CDF, and with the river at a low water stage. This case is shown on Figure 2.

The slope stability factor of safety relative to a shallow slope movement was 1.43. The factor of safety for slope stability for a deep slope movement that would intersect the dredged materials was 1.88. These values indicate that the berm would be stable during hydraulic filling.

6.3 Long Term Static Stability

The long term static stability case reflects a finished condition for the CDF. For this case, we assumed that the groundwater table within the CDF would approach current levels observed inland of Slip 1. This case is shown on Figure 3.

The factor of safety for our long term static stability analysis was 1.61. The factor of safety for deep slope movements was 2.25. These values indicate that the berm will be stable under normal operating conditions.

6.4 Seismic Stability (Pseudostatic)

The seismic case was developed based on the 475-year return interval event. In accordance with widely accepted analysis methods, we used a value equal to one-half of the peak horizontal acceleration developed from our seismic analysis. We used the maximum value for peak horizontal acceleration computed during our dynamic study. This case is shown on Figure 4.

The factor of safety relative to shallow, surface movement was 0.91. The factor of safety for deep shear surfaces that intersect the dredged sediments was 1.17. This indicates the potential exists for displacement of the berm toe during a design earthquake. However, the remaining berm possesses sufficient residual strength to contain the contaminated dredged materials.

6.5 Post Earthquake Stability

For the post earthquake scenario, we modified the strength parameters used in the static case. This approach is consistent with the design approach used at other Region 10 CDFs (City of Tacoma, 2003; Occidental Chemical and Port of Tacoma, 2003). This included inserting a zone of liquefied sand under the toe of the embankment. This case is shown on Figure 4.

Stress dependent undrained strength values were used to model liquefiable soils in the post-earthquake analysis. The stress dependent strength values were estimated using a correlation with corrected SPT blow counts. Since our liquefaction analysis indicated factors of safety between 1.0 and 1.2 for much of the native sands, we modified the friction angles for this formation based on estimating excess pore pressures. This analysis was extended to the upper dredge fills that were assumed to be uncompacted after placement. A summary of post earthquake soil parameters employed in our analyses is presented in the following table.

Table 6.5: Post Earthquake Soil Properties, Stability Assessment

Soil Unit	Moist Unit Weight (pcf)	Internal Friction Angle (phi')	$S_u(_{liq})/\sigma'_v$
Berm Fill	135	35	NA
Contaminated Dredged Material	120	0	0.1
CDF Fill	120	27	NA
CDF Surface Layer	125	35	NA
Liquefied Native Sands	120	0	0.2
Native Sands	120	30	NA

The factor of safety relative to shallow, surface movement was less than 1. The factor of safety for the deep shear surfaces that intersect the dredged sediments was 1.64. This indicates the potential exists for displacement of the berm toe after a design earthquake. However, the remaining berm possesses sufficient residual strength to contain the contaminated dredged materials.

A more detailed seismic deformation study will be completed. This study will use a Newmark analysis to model permanent slope deformation.

6.6 Statistical Evaluation

For each case evaluated, our statistical evaluations indicate that the probability for a deep movement that would impact the dredged sediments was 0. This analysis indicates that the proposed design more than adequately addresses the potential for variability within the strength of the soils present and proposed for use in the construction of the berm.

7.0 Discussion

Based upon our analysis, the CDF structure as proposed is protective of the contaminated sediments placed within the CDF. The structure will adequately protect and contain the dredged material..

Static factors of safety in excess of 1.5 and seismic factors of safety in excess of 1.1 are broadly considered stable for earth structures in cases where nominal permanent deformations are acceptable. For all cases, the factors of safety against a deep slope movement were far in excess of these values. The berm as designed will prevent the physical release of contaminated sediment..

Our analysis did indicate the potential for permanent deformations along shallow surfaces under a design seismic event. The shallow slope movement is considered to be within tolerable ranges, although such deformations would require rebuilding the outer face of the berm, our analysis indicates that the contaminated sediment within the CDF would not be impacted. The risks associated with shallow surface sloughing are comparable to the risks associated with most waterfront facilities in the Portland area..

8.0 References

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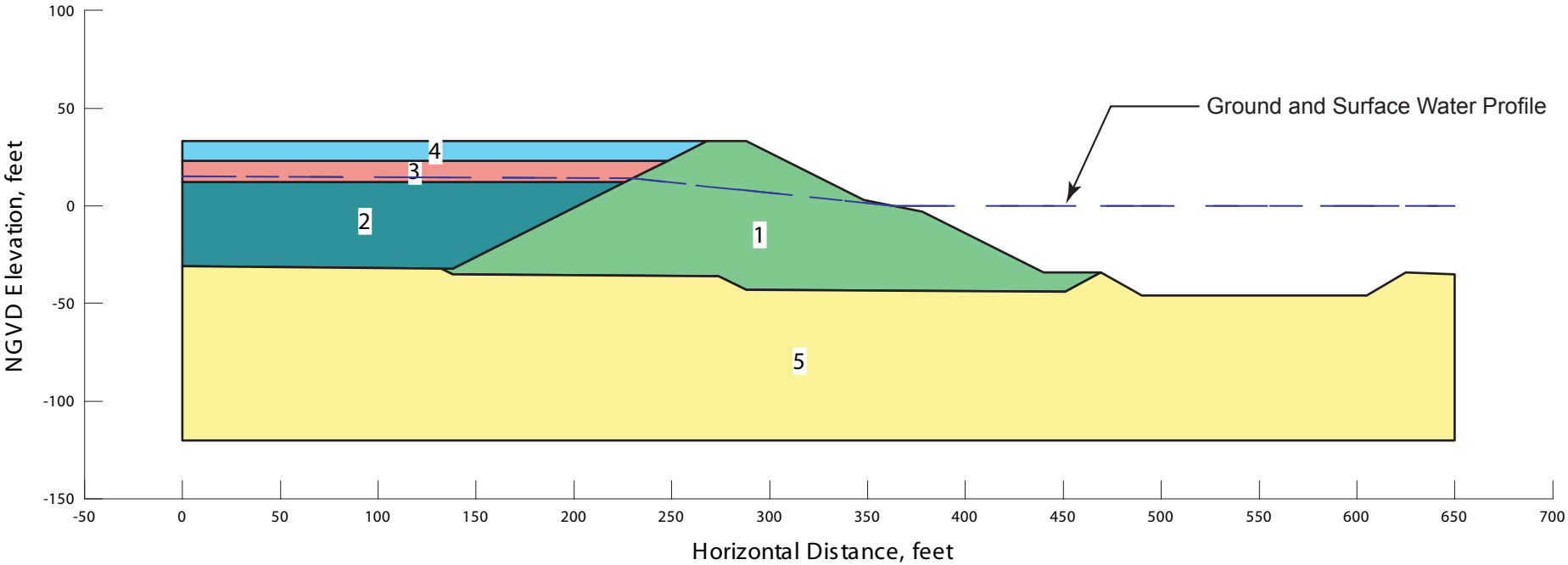


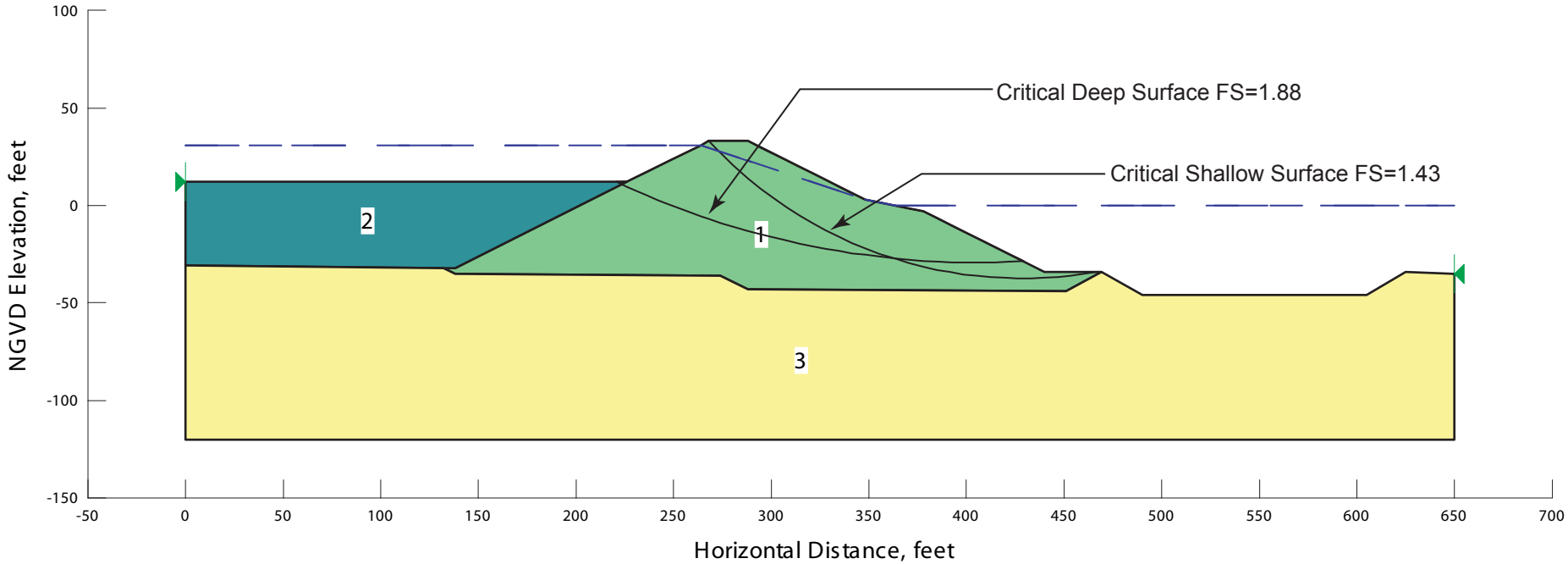
Soil Unit	Material
1	Berm Fill
2	Contaminated Dredged Materials
3	Cell Fill
4	Surface Fill
5	Native Sands

Basic Stability Model

Preliminary Seismic and Stability Evaluations
Terminal 4 Slip 1 CDF
Portland, Oregon

Ash Creek Associates, Inc.
Environmental and Geotechnical Consultants

Project Number	1165-00	Figure
August 2006		1



Soil Unit	Material	Unit Weight pcf	Friction Angle degrees	Cohesion pcf
1	Berm Fill	135	35	0
2	Contaminated Dredged Materials	120	24	0
3	Native Sands	120	35	0

Short Term Operational Stability

Preliminary Seismic and Stability Evaluations
Terminal 4 Slip 1 CDF
Portland, Oregon

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Project Number	1165-00	Figure
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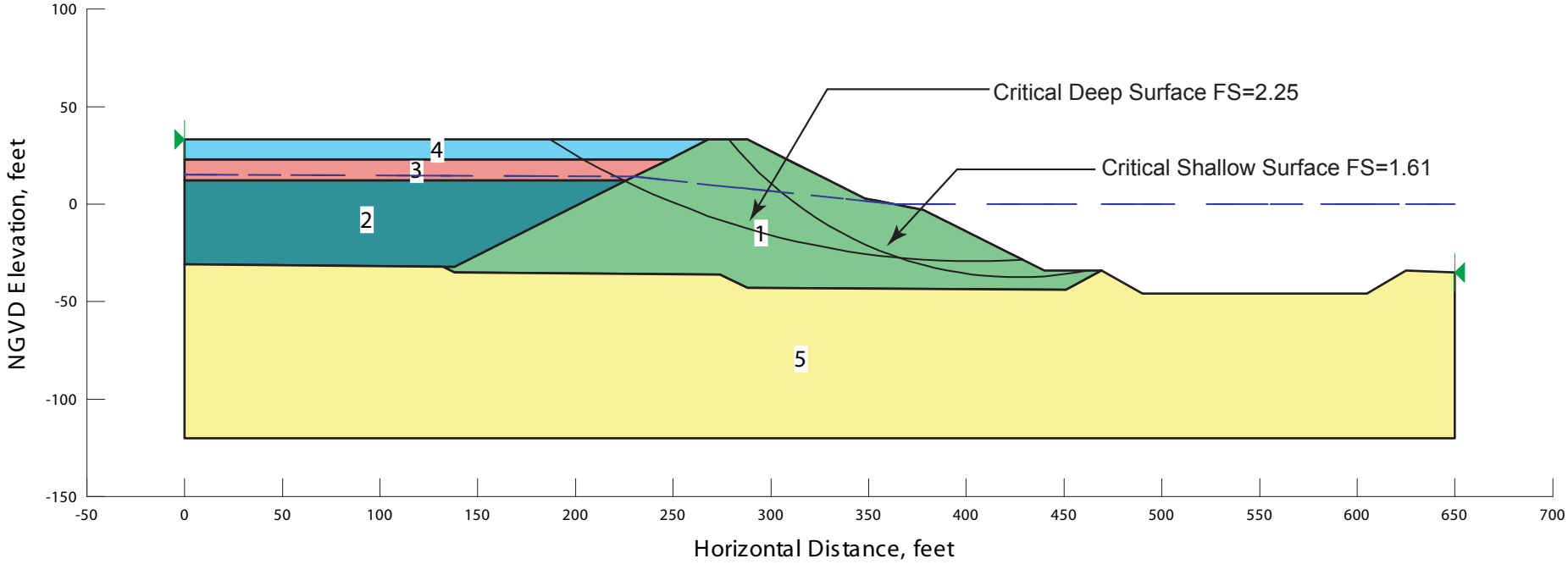

Soil Unit	Material	Unit Weight pcf	Friction Angle degrees	Cohesion pcf
1	Berm Fill	135	35	0
2	Contaminated Dredged Materials	120	24	0
3	Cell Fill	120	32	0
4	Surface Fill	125	35	0
5	Native Sands	120	35	0

Long Term Static Stability

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Terminal 4 Slip 1 CDF
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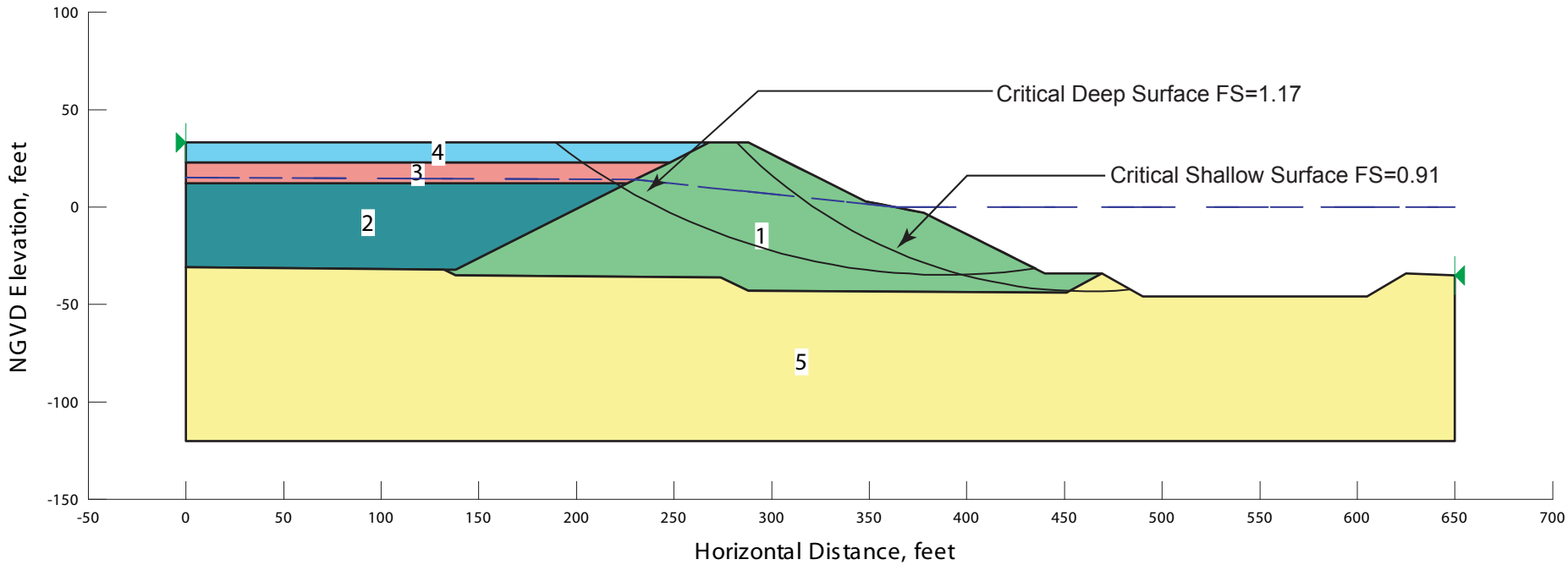

Soil Unit	Material	Unit Weight pcf	Friction Angle degrees	Cohesion pcf
1	Berm Fill	135	35	0
2	Contaminated Dredged Materials	120	24	0
3	Cell Fill	120	32	0
4	Surface Fill	125	35	0
5	Native Sands	120	35	0

Seismic (Pseudostatic) Stability

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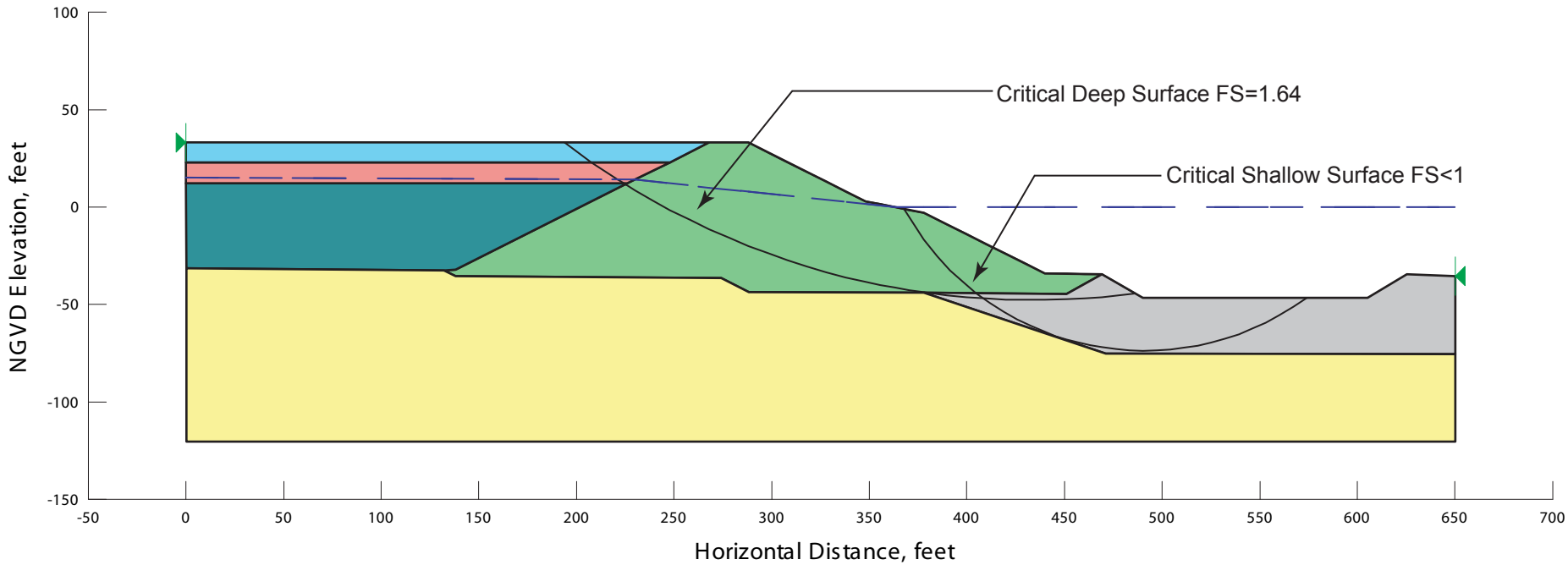

Soil Unit	Material	Unit Weight pcf	Friction Angle degrees	Su/σ'vo
1	Berm Fill	135	35	NA
2	Contaminated Dredged Materials	120	0	0.1
3	Cell Fill	120	27	NA
4	Surface Fill	125	35	NA
5	Liquefied Native Sands	120	0	0.2
6	Native Sands	120	30	NA

Post Earthquake Stability

Preliminary Seismic and Stability Evaluations
Terminal 4 Slip 1 CDF
Portland, Oregon

Ash Creek Associates, Inc.
Environmental and Geotechnical Consultants

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