Designers Responses to 90% IEPR Comments.

Comment # 1

The designer's direction from USACE Portland District Staff has been, from the beginning of the review process, that the analysis and design of the levee must follow EM 1110-2-1913. The majority of the water loading occurs on the face of the channel and not on the levee bank.

The Section A. shown on sheet C12 is at the deepest section of the riverside of the new levee. A review of the grading plan sheet C9 shows that much of the new levee is being constructed on higher ground. With Mean Higher High Water at roughly 9 feet NAVD88 the amount of tidal influence is a follows:

Station Range (feet)	Tidal Influence Range (feet)	Notes
61+00 to 63+00	0 to 6	Adjacent tidal channel
63+00 to 67+30	0	Riverside berm
67+30 to west end	0 to 3	Existing grade is 7-9 feet NAVD88

We are comfortable proceeding with this project using the EM 1110-2-1913.

Comment # 2

Please reference the revised Geotechnical report dated 6-20-2011. For Comments #2 through #10

We concur.

Comment # 3

Finding # 1-3

Response-See Section 4.2 of the revised report. Soil parameters used in the slope stability and seepage analyses were based on laboratory testing, boring explorations, SPT N-value correlations, and our experience with similar soil conditions. Laboratory testing indicated undrained shear strength, or cohesion, of approximately 460 psf (3.2 psi) for both the foundation soil and the re-compacted borrow material. The values can be derived from the plots of the unconsolidated undrained triaxial compressive tests (UU tests) which show the stress path during the soil test and are shown in Appendix A. The plots indicate p-values (one half of the difference between vertical and horizontal stresses) of 3.2 psi or 460 psf at the time of failure. Our analysis assumed an undrained shear strength of 460 psf for the compacted borrow which is consistent with the laboratory testing. For the analysis we reduced the shear strength of the foundation material to 350 psf, which is conservative relative to the undrained shear strength measured from the laboratory UU testing and equal to lowest corrected strength value from the field vane tests.

To evaluate strength of the soils under drained conditions, GeoDesign performed direct shear and consolidated drained triaxial compression tests (CD tests). Internal friction angles of greater than 30 degrees were computed based on the slope of the direct shear strength versus the normal pressures measured during the direct shear tests for both the compacted burrow material and the native material. A CD test performed on a sample of foundation soil indicated a friction angle of 23 degrees and a cohesion value of 140 psf. The strength values from the CD test were determined by from the angle and intercept of the Mohr's Circle, which is a graphical representation of the stress along the major and minor axes at a given point, in this case the point of failure.

In our analysis we used the drained strength parameters of the foundation material measured from the CD testing which indicate lower strengths than the drained strength parameters measured from the direct shear testing. For the embankment material we used an angle of friction of 28 degrees. This value is less than value indicated from the direct shear testing and consistent with published values for compacted clay (Holtz and Kovacs, 1981).

Strength parameters under consolidated undrained conditions (CU) were estimated based on the strength test results from unconsolidated undrained (UU) and consolidated drained (CD) laboratory test results and our experience with similar soils. The parameters are shown in Table 1.

Table 1. Soil Strength Parameters

Soil Unit	**Saturated Unit Weight (γ_{sat}) (pcf)**	**Cohesion (c) (psf)**	**Internal Friction Angle (ϕ) (degrees)**
Unconsolidated Undrained Strength Parameters			
Compacted Levee Fill	105	460	0
Foundation Soil	90	350	0
Consolidated Undrained Strength Parameters			
Compacted Levee Fill	105	300	14
Foundation Soil	90	225	11
Drained Strength Parameters			
Compacted Levee Fill	105	140	28

Foundation Soil	90	140	23

Finding # 4

See section 3.3.2 of the revised report. . Corrected field vane shear testing indicates the soil has an undrained shear strength of approximately 0.18 to 0.25 tsf. The field vane measurements were corrected using an empirical correction factor as given by Bjerrum (1972) and using the measured plasticity index and a time to failure of 10^4 minutes which is typically associated with embankment failures involving large construction equipment. An unconsolidated undrained triaxial compression test indicated the soil had a cohesion of 460 psf.

Finding # 5

No parameters for consolidation, drained strength, and undrained unconsolidated shear strength were derived from “thick-walled” samplers. Only one direct shear test was performed on a sample of the foundation material using a thick walled sampler. Results of the testing were very similar to direct shear test performed from a Shelby Tube sampler; however, in disturbance would have resulted in conservative strength parameters. The drained strength parameters used in the revised analysis were based on a consolidated drained test performed on a sample taken from a Shelby Tube. Both consolidation tests were performed on a thin walled Shelby tube (a typo incorrectly identified one sample as being from at 5 feet in boring B-4 where an SPT sampler was driven; however, the sample was testing was performed on a sample Shelby tube at 5 feet in boring B-5 as verified by the original paperwork).

Comment # 4

Section 4-2a of EM 1110-2-1913 states “Almost any soil is suitable for constructing levees, except very wet, fine-grained soils or highly organic soils.” The report and specifications indicate that drying of the embankment soils to within 10 percent of optimum will be required and prohibits the use of soils with organic contents greater than 10 percent. Further, the adjacent levee which will be removed and will be replaced by the proposed levee appears to be constructed of similar materials at similar slopes and performed as intended during since it was constructed.

Additional analyses have been performed with reduced soil strength parameters. Soil parameters in the revised analysis are based on laboratory testing performed by an USACE accredited laboratory and published values, and are likely conservative as the lowest values from each type of testing on any one sample are used.

EM 110-2-1913 indicates that slopes shall be no greater than 2H:1V depending on the results of the stability analysis, and the stability analysis indicates the slopes are stable.

Please refer to the Construction QA Plan Appendix DES90-5. If the quantity of on-site material is not sufficient to construct the levee, or if sufficient moisture conditioning and compaction cannot be achieved, it will be necessary to import fill. Imported fill material used to construct the levee should generally be fine grained, free of organics and other deleterious material, and sufficiently impermeable in its compacted condition. The imported material should have the following properties:

- A classification of ML, MH or CL in accordance with the Unified Soil Classification System.
- A minimum of 90 percent by dry weight of particles passing the Standard No. 200 US Sieve.
- A minimum shear strength of 460 psf at a compacted density equal to 90 percent of the maximum dry density as determined by ASTM D 698.
- Have an organic content of less than 10 percent by dry unit weight.

Comment # 5

We used Seep/W to compute the pore water pressure on the proposed levee during steady state conditions from flood stage. The pore water pressures computed during our seepage analysis were used as input for our steady state analyses performed in Slope/W. In our seepage analyses, we used a saturated hydraulic conductivity of 3.3×10^{-8} feet/second and published equations for the soil-water characteristic curves (hydraulic conductivity and water content functions) for silt from Green and Corey (1971) for the embankment material, as shown in Appendix B. We selected this soil-water characteristic curve because it had a relatively low permeability and high volumetric water content compared to other functions for silt in the published literature. Because of the high water content and high plasticity of the soils, in our opinion this function most closely represents the compacted embankment material. However, to evaluate the sensitivity of the solution to different hydraulic conductivity and water content functions we also performed analysis using other functions including the Clay/Silt Functions from Fredlund and Xing (1994), and computed a similar factor of safety.

The vertical hydraulic conductivity of the native silt was estimated as $2.5\text{x}10^{7}$ feet/second based on published values for similar soils. However, to evaluate the sensitivity of the solution to hydraulic conductivity, we also performed an analysis using a vertical hydraulic conductivity of $3.3\text{x}10^{-8}$ for the foundation soils and obtained a similar factor of safety. For the purposes of our analysis we assumed that the vertical hydraulic conductivity was equal to 1/3 of the horizontal hydraulic conductivity to account for layers having a higher hydraulic conductivity than others. Our analysis used a volumetric water content of $0.7 ft^3/ft^3$ for foundation soils, which was computed based on the laboratory measured average moisture content of 88 percent for the soil samples. We performed the analysis using volumetric water contents ranging between 0.5 and 0.7 ft^3/ft^3 for the foundation soils and used a value of 0.7 to be conservative.

It has been suggested that the horizontal conductivity of compacted clay may be several times higher than the vertical conductivity because of the effects of soil fabric or the flow between lifts. However, laboratory tests performed on clay with good bonding between lifts has shown the vertical and hydraulic conductivity can be essentially identical (Boynton and Daniel 1985). To account for anisotropy in the field as a result of lack of bonding between lifts or some layers with higher permeability than others we analyzed the embankment with a vertical hydraulic conductivity equal to ¾ of the horizontal hydraulic conductivity. However, the stability of the embankment was not sensitive to differences between the horizontal and vertical permeability of up to 25 percent. The results of our Slope/W analysis are shown in Appendix C for steady state seepage.

Under steady state conditions our seepage analysis conservatively assumed that the water level on the upstream side of the levee to be at 15.2 feet (NAVD 88). For the analysis of the downstream slope, we estimated a pieziometric surface of 3 feet MSL, which is near the base of the existing drainage channel at the toe of the levee. Our seepage analysis computed an exit gradient of 0.15 was computed at the base of the drainage ditch, and an exit gradient at the toe of the levee to be 0.43 during steady state seepage conditions. This is less than the maximum permissible gradient of 0.5 at the toe of the levee, recommended by Engineer Technical Letter (ETL) 1110-2-529 Design Guidance for Levee Underseepage.

The Design Levee Elevation for Diking District 11 (aka Minimum Crest Height) is based on the 1933 high water as recorded plus 3 feet freeboard. This is 7.7 feet 1929(MSL) + 3 feet = 10.7 Adjusted to NAVD88 (See Bench Mark Narrative Below), this is 11.99 feet + 3 feet = 14.99. The proposed top of levee (after 1 foot of settlement) is 15.2 feet NAVD88. The road section is on top of that. The seepage and slope stability analysis were performed with the riverside water elevation at the top of levee (15.2 feet NAVD88). This is conservative.

Comment # 6

Finding # 1

The crest of the slope was established at 12 feet to allow for a vehicle to access the levee for purposes of inspection or flood fighting. The proposed levee geometry (3H:1V on the landward side and 2H:1V on the slope on the riverward side) maximizes the slope stability under the current space requirements since there is a drainage ditch at the toe of the slope on the landward side. The Section 6-1 states “A 1V:3H is the steepest slope that can be conveniently traversed with conventionally mowing equipment and walked on during inspections.” However, Section 6-1 also states minimum that levee sections are often dictated by space requirements, in certain situations such as urban environments. Significant portions of the existing levee are built at a 2H:1V on the riverward side, including the section that the proposed improvements are replacing. Consequently, replacing a limited section of the existing levee with a new levee with similar slopes is not going to have a significant effect on the convenience of maintaining the levee system, and widening the slopes would impinge on an existing, stream channel in the proposed wetland. The slopes meet the minimum levee requirements for flood-fighting, maintenance, inspection, and for general safety conditions, of a 10 feet crown width and 1V on 2H slope as stated Section 6-2 c of EM 1110-2-1913.

Section 6-1 states that low levees and levees to be built of good material resting on proven foundations may not require extensive stability analysis. The foundation is soft; however, extensive analysis was performed of the levees. Further, the existing levees which were constructed between 1939 and 1941 do not show evidence of slope instability that would compromise the function of the levee. See response to Comment #7 for additional discussion.

Finding 2. The Design Levee Elevation for Diking District 11 (aka Minimum Crest Height) is based on the 1933 high water as recorded plus 3 feet freeboard. This is 7.7 feet 1929(MSL) + 3 feet = 10.7 Adjusted to NAVD88 (See Bench Mark Narrative Below), this is 11.99 feet + 3 feet = 14.99. The proposed top of levee (after 1 foot of settlement) is 15.2 feet NAVD88. The road section is on top of that. The seepage and slope stability analysis were performed with the riverside water elevation at the top of levee (15.2 feet NAVD88). This is conservative.

Finding 3. The levee sections on sheets C12 and C16 will be updated to show the required 15 foot buffer. The buffer is shown on the plan view C9 as a boundary. It is labeled as the Dike Planting Zone and it needs to be labeled in addition as the boundary of the 15 ft width for access, inspection, vegetation management and flood fighting.

Comment # 7

Finding # 1

Section 4.2.1 of our revised report provides additional discussion on soil parameter selection. Laboratory testing indicated undrained shear strength, or cohesion, of approximately 460 psf (3.2 psi) for both the foundation soil and the re-compacted borrow material. The values can be derived from the plots of the unconsolidated undrained triaxial compressive tests (UU tests), which show the stress path during the soil test and are shown in Appendix A. Our analysis assumed an undrained shear strength of 460 psf for the compacted borrow, which is consistent with the laboratory testing. For the analysis we reduced the shear strength of the foundation material to 350 psf, which is conservative relative to the undrained shear strength measured from the laboratory UU testing and equal to lowest corrected strength value from the field vane tests.

To evaluate strength of the soils under drained conditions, GeoDesign performed direct shear and consolidated drained triaxial compression tests (CD tests). Internal friction angles of greater than 30 degrees were computed based on the slope of the shear strength versus the normal pressures measured during the direct shear tests for both the compacted burrow material and the native material. A CD test performed on a sample of foundation soil indicated a friction angle of 23 degrees and a cohesion value of 140 psf. The strength values from the CD test were determined by from the angle and intercept of the Mohr's Circle.

In our analysis we used the drained strength parameters of the foundation material measured from the CD testing, which indicate lower strengths than the drained strength parameters measured from the direct shear testing. For the embankment material we used an angle of friction of 28 degrees. This value is less than value indicated from the direct shear testing and consistent with published values for compacted clay (Holtz and Kovacs, 1981).

Strength parameters under consolidated drained conditions (CD) were estimated based on the strength test results from unconsolidated undrained (UU) and consolidated undrained (CU) laboratory test results and our experience with similar soils.

Finding #2

It has been suggested that slope stability analysis should be performed on a cracked embankment. However, the existing embankment was constructed to a similar elevation as the proposed embankment crest, and GeoDesign staff did not observe embankment slope failures or cracking that could be associated with placement of fill on a soft foundation. The slope stability analysis in our revised report already based on a number of conservative assumptions. Specifically, we performed multiple tests using different soil testing techniques to obtain both drained and undrained soil strengths and used the lowest value of any test in our analyses, we assumed the water elevation at the top of the levee (not the 100 year flood elevation), we used conservative slope geometries, we assumed that the water level for rapid drawdown analysis would be at an elevation of 3 feet, even though the drainage ditch was filled with water during our site visit and would likely be filled with water during flood stage conditions. The analysis with cracked embankment conditions is not part of the USACE requirements, and it is our opinion that performing additional analysis with additional conservative assumptions that are not supported by field observations is unnecessary.

The computed slope stability analysis results appear to be empirically confirmed by the performance of the existing levee embankment and field observations. As documented in the periodic inspection performed by HDR in January of 2011 (HDR, 2011), the riverward slopes along the Clatsop 11 segment of the levee system are 2H:1V. GeoDesign also estimated the slopes of the existing levee using a clinometer to be approximately 2H:1V over significant portions of the levee. Construction documents do not indicate the source material for the proposed levee; however, the existing levee located north of the proposed levee appears to be constructed of native silt material similar to that proposed for the new levee.

A periodic inspection performed by HDR in January 2011 rated the slope stability of the levee as acceptable and stated that during the inspection no slides, sloughs bulges, or depressions (excluding cattle damage) were noted on the levee slopes. It was noted in the HDR report that intermittent thick vegetation inhibited the inspection of this item and could be hiding minor issues. A member a GeoDesign's geotechnical staff also walked the levee to observe signs of instability and sloughing and did not observe any significant failures. We note that significant lengths of the levee were obscured by vegetation during our July 2011 site visit. However, the area immediately to the north of the proposed new section is currently being used as pastureland and was vegetated by short grasses (less than 2 inches in height). The riverward slopes were measured at approximately 2H:1V and GeoDesign staff did not observe evidence of slope failure.

Finding # 3

Under steady state conditions our seepage analysis conservatively assumed that the water level on the upstream side of the levee to be at 15.2 feet (NAVD 88). For the analysis of the downstream slope, we estimated a pieziometric surface of 3 feet (NAVD 88), which is near the base of the existing drainage channel at the toe of the levee.

Finding #4

Section 4.2 of our revised report for additional discussion on the seepage analysis. We used Seep/W to compute the pore water pressure on the proposed levee during steady state conditions from flood stage. The pore water pressures computed during our seepage analysis were used as input for our steady state analyses performed in Slope/W.

The rapid drawdown analysis was performed in Slope/W using the “3-stage” undrained strength method proposed by Duncan, Wright and Wong and adopted by USACE (manual for Design and Construction of Levees EM-1110-2-1913) for soils with low permeability (hydraulic conductivities less than $1x10^{-4}$ cm/sec or $4x10^{-5}$ in/sec). The purpose of the first stage of analysis is to compute the effective stress to which the soil is consolidated prior to drawdown. The consolidation stresses are used to estimate the undrained shear strengths for second-stage computations, with the reservoir lowered. The third computations analyze the stability after drawdown, using the lower of the drained or undrained strengths to ensure a conservative factor of safety is used. The revised rapid drawdown analysis results (analyzed riverside) are shown in Appendix C of the revised report.

Two rapid drawdown analyses were performed. The first analysis assumed sufficient time had occurred to allow consolidation of the soils and consolidated undrained parameters were used during drawdown (c = 300 psf and phi= 14 degrees for the embankment and c = 225 psf and phi= 11 degrees for the foundation material). The second analysis was performed using unconsolidated undrained strength parameters during drawdown (c = 460 psf and phi= 0 degrees for the embankment and c = 350 psf and phi= 0 degrees for the foundation material). For the analyses, the piezometric lines were set at 15.2 feet (NAVD 88) before drawdown and 3 feet after drawdown. A friction angle of 23 degrees and a cohesion 140 psf were used for the drained strength parameters during the analysis.

Finding # 5

The sections used in the analyses were provided by the site civil engineer and represent the as-built condition prior to settlement. After settlement occurs the slopes will be slightly more flat and not as high. However, stability analysis performed on the slopes presented in the figures is conservative.

Finding # 6

The report has been revised and the rapid drawdown stability analysis has been performed on the riverward side. Please see section 4.2 and Appendix C of the revised report for results of the revised analysis.

Comment #8

A copy of our calculations is attached in Appendix D of the revised report for settlement calculations. The exact amount and placement schedule of the stockpile material over levee footprint is unknown, making it difficult to predict the effect of stockpiling on the total pore pressures and settlement of the embankment. However, monitoring of the settlement will be performed using settlement plates and piezometers will be in place to measure pore water pressures and help determine will settlement is complete. If more settlement occurs that is predicted additional levee fill will be placed to achieve the design levee elevation.

Comment # 9

Finding # 1

The revised report and specifications clearly state that all stockpiled material and any other loose or unsuitable material will be removed from the levee foundation prior to levee construction. The grading plan notes will be updated to reflect this.

Finding # 2

The onsite tidal channel construction that has occurred under Phase 1, is located immediately south of the new levee (from levee Sta. 61+00 to 69+00)(see sheet C9). It has been excavated to elevations ranging from 4 to 5 feet. No significant areas of permeable soils, organic areas or abandoned underground utilities have been found. We feel that that provides a sufficient exploration for those conditions for that area, since the test holes have shown relatively homogeneous, impermeable conditions. The west end of the levee from Sta. 69+00 to the end will be sampled to locate for such conditions by 3 test holes.

Finding # 3

A slope stability analysis of the levee (see “Levee Stability and Seepage” section of this report) indicates that the fully constructed levee is stable under undrained conditions. Further, stockpiles of greater than 20 feet in height are currently present on the site and the stockpiles have not resulted in slope stability issues. Based on the analysis and field observations, staged construction does not appear to be necessary. However, as precautionary measure, we recommend that two vibrating wire piezometers be installed near the toe of the proposed levee. The piezometers should be read daily while fill is being placed in the vicinity of the piezometers.

Site grading has resulted in the stockpiling of material on the approximate western two-thirds of the site. The grading will likely reduce the amount of total settlement and excess pore water pressures over this portion of site. The exact amount and placement schedule of the material is unknown, making it difficult to predict the effect of stockpiling on the total pore pressures and settlement of the embankment. However, one piezometer should be placed on the eastern portion of the site where no soil stockpiling has occurred. We also recommend a piezometer be placed on the western portion of the site. The piezometers should be placed as close to the landside edge of the toe as practicable, without being in a location they will be at risk of damage or interfere with the levee construction. We understand that levee construction will proceed from east to west. We anticipate the most settlement at the eastern edge of the levee, and staging the construction in this manner will allow for the maximum amount of settlement and pore water dissipation to occur before the completion of construction in the area where the highest amount of settlement is anticipated.

Findings # 4,5,6

The levee fill should be placed in thin uniform lifts and compacted to at least 90 percent of the maximum density and be within 10 percent of the optimum moisture content as determined by ASTM D 698. Each lift of fill should be uniformly compacted in lift thicknesses less of 9 inches or less in loose thickness, except for the first lift which can be thicker in order to protect the subgrade.

Finding # 7

This approach to embankment construction sequencing will be considered when detailing a comprehensive construction staging plan and schedule.

Comment # 10

See plan sheets C6 and C8. The areas of the riverside of the levee that are subject to tidal drawdown (below 9 feet NAVD88) are called out on sheet C6 as slope protection zones. These will be covered with a biodegradable erosion control mat with a tight stake pattern and planted with sedges and bulrushes that are tolerant of mowing, inundation and salinity. The plant list and matting notes are contained on sheet C6 and the mat staking and edge anchor details are shown on sheet C8.

Comment # 11

The culvert and tidegate replacement have been dropped for the current project. The existing tidegate, while undersized by current USACE standards, has functioned adequately. The tributary drainage area to this gate will be reduced by approximately 35 acres as a result of the setback of the levee. The culvert and tidegate and culvert will be upgraded at a later time and will be approved under a separate Section 408 review at that time.

Comment # 12

The Design Levee Elevation for Diking District 11 (aka Minimum Crest Height) is based on the 1933 high water as recorded plus 3 feet freeboard. This is 7.7 feet 1929(MSL) + 3 feet = 10.7 Adjusted to NAVD88 (See Bench Mark Narrative Below), this is 11.99 feet + 3 feet = 14.99. The proposed top of levee (after 1 foot of settlement) is 15.2 feet NAVD88. The road section is on top of that.

Comment # 13-16, 20

These pipes and the tidegate have been dropped from the project. See response for Comment # 11.

Comment # 17

The discussion of residual risk has been added to the Design Summary.

Comment # 18.

The flows are instantaneous flows derived from recorded peak flows on existing stream gages through the USGS Regional Regression Equations. The calculations for the flows will be shown in an updated report. The “average” flow is the result of the equation. The “minimum” and “maximum” flows in the table are the results with the standard error of the regression equation subtracted and added. They were shown for illustrative purposes and were not used in the model.

Comment # 19

The tide station ID numbers have been added to the report.

Comment # 21

The before and after WSE profiles have been shown in Figures A-27 and A-28. (at the back of the Exhibits document).