

### 3.0 GEOTECHNICAL

#### 3.1 Design Procedure for Earthen Embankments

The following represents the typical procedure for the geotechnical design and analysis of levee embankments. The procedures stated herein, although considered typical, are in no way implied to eliminate engineering judgment.

##### 3.1.1 General Design Guidance

USACE Publications:

- EM 1110-2-1913, Design and Construction of Levees, Apr. 00
- EM 1110-2-1901, Seepage Analysis and Control for Dams, Apr 93
- DIVR 1110-1-400, Soil Mechanic Data, Dec. 98
- ETL 1110-2-569, Design Guidance for Levee Underseepage, May 05

Computer Software:

- Slope Stability Program based on “MVD Method of Planes” (Method of Plane Program and plotting program is available by contacting New Orleans District. Point of Contact is Paul R. Oakland at [Paul.R.Oakland@usace.army.mil](mailto:Paul.R.Oakland@usace.army.mil)).
- Slope Stability Programs based on “Spencer’s Procedure”

NOTE: While there are references in this document to specific, proprietary computer programs, these are included only as representative of the function and quality of calculations. Other programs which can perform like analyses and provide output in similar format are acceptable.

##### 3.1.2 Field Investigations

For levee design, centerline and toe borings should be taken every 500 feet (OC), with borings alternating between 5” undisturbed and general type soil borings or CPTs.

Borrow borings are typically taken at 500 feet OC. Consult geologists when developing boring programs.

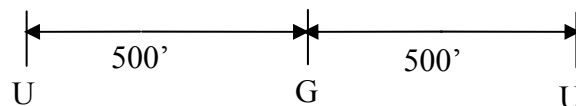


Figure 3.1 Boring spacing

### **3.1.2.1 Strengthlines**

The guidance outlined herein assumes test results are from 5" diameter undisturbed samples; unconsolidated-undrained triaxial (Q) tests are the predominant tests and are supplemented by unconfined compression (UCT) tests. The methods of analysis should be both Spencer Method and Method of Planes using the factors of safety outlined in Table 3.1. Strengthlines should be drawn such that approximately one-third of the tests fall below the strengthline and two-thirds plot above the strengthline. A line indicating the ratio of cohesion to effective overburden pressure ( $c/p$ ) of 0.22 should be superimposed on the plot. The  $c/p$  line may be used to assist in determining the trend of the strengthline. A plot of centerline strengths under an existing embankment and another plot under natural ground to be used for toe strengths should be drawn.

### **3.1.2.2 Slope Stability Design Criteria**

Criteria in Table 3.1 is based on criteria presented in EM 1110-2-1902 Slope Stability, 2003, for new embankment dams adapted for southeast Louisiana hurricane and storm damage reduction system. In accordance with EM 1110-2-1902 acceptable factors of safety for existing structures may be less than for new dams, as referenced in paragraph 3-3 Existing Embankment Dams, only when the existing structures have performed satisfactorily under the design or higher load condition. Given the unique soil conditions of southern Louisiana, the potential complexity of the levee and floodwall features, and the required intricacy of the slope stability software programs now being implemented, designers must take extreme care when utilizing software programs for these geotechnical designs. Engineers must spend appropriate time and effort in verifying that software program input correctly models the problems to be solved and that the resulting output provides a reasonable design with the most critical failure surfaces (i.e. when using SLOPEW program, critical failures surfaces and factors of safety shall be analyzed both with and without utilizing the optimization option).

\* Note that risk-based approaches are currently being developed for future incorporation in these criteria.

**Table 3.1 – Slope Stability Design Factors of Safety.**

Analysis Condition	Required Minimum Factor of Safety	
	Spencer Method <sup>1</sup>	MOP <sup>2</sup>
End of Construction <sup>3</sup>	N/A	N/A
Design Hurricane <sup>4</sup> (SWL)	1.5	1.3
Water at Project Grade (levees) <sup>5</sup>	1.4 (1.5) <sup>6</sup>	1.2
Water at Construction Grade (levees) <sup>5</sup>	1.2	N/A
Extreme Hurricane (water @ top of I-Walls) <sup>5</sup>	1.4 (1.5) <sup>6</sup>	1.3
Extreme Hurricane (water @ top of T-Walls) <sup>5a</sup>	1.4 (1.5) <sup>6</sup>	1.2
Low Water (hurricane condition) <sup>7</sup>	1.4	1.3
Low Water(non-hurricane condition) <sup>8</sup> S-case	1.4	1.3
Water at Project Grade Utility Crossing <sup>9</sup>	1.5 (1.4)	1.3 (1.2)

**NOTES:**

1. Spencer method shall be used for circular and non-circular failure surfaces since it satisfies all conditions of static equilibrium and because its numerical stability is well suited for computer application. These factors of safety are based on well defined conditions where: (a) available records of construction, operation, and maintenance indicate the structure has met all performance objectives for the load conditions experienced; (b) the level of detail for investigations follow EM 1110-1-1804, Chapter 2, for the PED phase of design; and (c) the governing load conditions are established with a high level of confidence. Poorly defined conditions are not an option, and the Independent Technical Review must validate that the defined conditions meet the requirements in this footnote.

2. Method of Planes shall be used as a design check for verification that levee and floodwall designs satisfy historic district requirements. Analysis shall include a full search for the critical failure surface per stratum since it may vary from that found following the Spencer method.

3. Given the non-critical nature of the required factors of safety for the End of Construction case, analysis of this load case is not required.

4. Applies to analyses failing toward the protected side for the SWL condition (100-yr return period, 90% assurance, is authorized as the current design hurricane loading condition). Stability is analyzed for the as-constructed section with water a SWL using drained strengths expressed in terms of effective stresses for free-draining materials and undrained strengths expressed in terms of total stresses for materials that drain slowly. For water at SWL against T-Walls, Method of Planes analysis is required as a design check only. See T-Wall criteria later in this chapter for specific details.

5. Applies to analyses failing toward protected side of the as-constructed levee or

floodwall section for different water load cases under a short term hurricane condition. Stability for levee and floodwall systems are analyzed using drained strengths expressed in terms of effective stresses for free-draining materials and undrained strengths expressed in terms of total stresses for materials that drain slowly.

(a) For water at the top of as-constructed T-Walls, Method of Planes analysis is required as a design check only. See T-Wall criteria later in this chapter for specific details.

6. The required factor of safety shall be increased from 1.4 to 1.5 when steady-state conditions are expected to develop in the embankment or foundation. (The higher FOS only applies to the freely-draining sand stratum that can obtain the steady state condition).

7. Applies to flood side where low hurricane flood side water levels are quickly lowered. Method of Planes analysis is required as a design check only for T-Walls. See T-Wall criteria later in this chapter for specific details. This analysis represents a short-term rapid drawdown situation that may occur when a hurricane passes so that winds are in a direction away from the levee. Criteria are from EM 1110-2-1902, Table 3-1, and note 5, considering potential erosion concerns. Stability is analyzed for the as-constructed levee section using drained strengths expressed in terms of effective stresses for free-draining materials and undrained strengths expressed in terms of total stresses for materials that drain slowly.

8. Applies to flood side and protected side. Method of Planes analysis is NOT required for T-Wall designs. This analysis represents a long-term water level drawdown where steady state conditions prevail. Stability is analyzed for the as-constructed levee section using drained strengths expressed in terms of effective stresses. (S-case type analysis for normal loading condition; non-hurricane loading.)

9. Applies to flood side and protected side for levees and I-Walls for water at Project Grade. For the flood side analysis, low water elevation is low water produced by hurricane conditions. Stability is analyzed using drained strengths expressed in terms of effective stresses for free-draining materials and undrained strengths expressed in terms of total stresses for materials that drain slowly. The lower FOS (in parenthesis) may be used for levees that have received their final levee lift.

### 3.1.2.3 Reserved

Table 3.2 – Reserved.

### 3.1.3 Levee Embankment Design

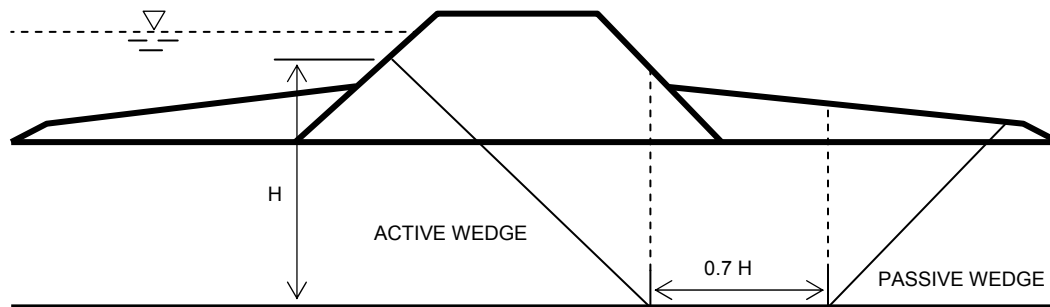
A. Using centerline borings, toe borings, CPTs, and applicable test results, determine stratification, shear strength, and unit weights of materials and separate alignment into soils and hydraulic reaches. Soil parameters and stratification to be used for design must be reviewed for approval by senior engineer.

B. Using cross sections of existing conditions, determine minimum composite sections for similar topography for each reach.

C. Using consolidation test data, determine stratification for settlement purposes. Verify that the assumed gross section minus the total settlement is greater than or equal to the required net section or determine the number of subsequent lifts during project life to maintain grade higher than design grade. Also future subsidence and sea rise should be investigated.

D. Using both the Spencer Method and the Method of Planes (Stability with Uplift program which will be provided by the Government) and design undrained shear strengths, determine the Factor of Safety of the gross section. Compare Factor of Safety to established design criteria. At a minimum, the following analyses shall be performed:

If inadequate, design stability berms, reinforcing geotextile, soil improvements, or some other means to produce an adequate Factor of Safety with regard to the current design criteria. The designer should check the final design section determined by the Method of Planes and the Spencer Method and present the Factors of Safety for both analyses. The minimum distance between the active wedge and passive wedge should be  $0.7H$  as shown in Figure 3.2, where  $H$  is the vertical distance of the intersection of the active wedge with the ground surface and its intersection with the failure surface.



**Figure 3.2 Minimum distance between active and passive wedges (embankments)**

E. Typical assumed values (in lieu of test results) for undrained soil parameters are shown in Tables 3.3 and 3.4.

**Table 3.3 – Typical values for embankment fill.**

Soil Type	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)
Compacted Clay (90%)	110	400	0
Compacted Clay from Bonnet Carrie (from dry borrow pit placed on land)	115	600	0
Uncompacted Clay (from dry borrow pit placed on land)	100	200	0

All sections of HSDRRS levees (central portion, wave berms and flood/protected side stability berms) shall be designed and constructed utilizing compacted clay.

**Table 3.4 - Typical values for Silts, Sands, and Riprap**

Soil Type	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)
Silt	117	200	15
Silty Sand	122	0	30
Poorly graded sand	122	0	33
Riprap	132	0	40

- Notes. 1) Weight of riprap may vary based on the filling of the riprap voids over time.  
 2) Undrained soil parameters for the S-Case are:  
     Silt: Cohesion = 0 psf,  $\phi = 28$   
     Clay: Cohesion = 0 psf,  $\phi = 23$

F. If embankment material is to be taken from the protected side in an adjacent borrow pit or if an adjacent canal exists, stability of the embankment must be checked to determine the allowable distance of the pit away from the embankment and the allowable depth of the pit. Typical allowable factors of safety for an adjacent borrow pit or canal are 1.5 with a flooded pit, and 1.3 with a dry pit. These analyses should be performed with flood side water at the Still Water Level. Factors of safety are applicable for both Method of Planes and Spencer's Method.

G. At pipeline crossings, the allowable Factor of Safety shall be 1.5 for the gross section for a distance of 150 feet on either side of the centerline of the pipeline or

an appropriate distance determined by engineering assessment. This analysis should be performed with flood side water at the Still Water Level.

### 3.1.4 Seepage Analysis

#### 3.1.4.1 Definitions

**Stage, or water surface elevation (wse)** – the height of water against a levee or floodwall. Water height is measured as the vertical distance in feet (meters) above or below a local or national elevation datum.

**Design Water Surface Elevation (DWSE)** – the stage or water level to be used in deterministic analyses such as the geotechnical, structural stability, and seepage analyses. For the HPS, the DWSE is found from the authorized water surface elevation (AWSE) and its associated uncertainty at the selected confidence limit, where uncertainty is represented by normal distribution, and the confidence limit is 90%:

AWSE = best fit for 50% confidence level  
DWSE = 90% confidence level

**Project Grade** – this represents the net grade of the levee or floodwall, and is sometimes referred to as top of protection, top of levee, or net levee grade. The project grade includes increases above the DWSE to account for wave action/runup, minus the overbuild that is provided for primary consolidation.

#### 3.1.4.2 Design Assumptions and Considerations

1. The HPS Seepage Design Criteria will be applied exclusively to the design of levees and floodwalls that protect areas where there would be **very high consequences** should the levees or floodwalls fail during a flood or hurricane event. Very high consequences entail losses of human life and/or major damage to exceptionally valuable property or critical facilities.

2. The HPS Seepage Design Criteria will be used only where the uncertainty of subsurface conditions and soil properties is “*small*.” To reduce the uncertainty of subsurface conditions to a “*small*” level, it is necessary to perform more than the minimum number of subsurface explorations. The minimum number of explorations is commonly described as a series of three explorations, boreholes or soundings, performed approximately every 1,000 feet (~300 meters) – see ETL 1110-2-569. In addition to performing additional borings and/or soundings, the subsurface explorations should be coupled with data from geophysical testing or other supplemental investigations such as Cone Penetrometer Testing (CPT) designed to explore the variability in subsurface conditions. To reduce the uncertainty of soil properties to a “*small*” level, it is also necessary to perform laboratory tests to characterize soil unit weight properties. Further, post-

construction monitoring of piezometric levels need to be performed, where feasible, in order to qualify for a “*small*” level of uncertainty.

3. The design water surface elevations are associated with the 100-year flood/hurricane events. The fact that the DWSE has a 90% confidence level results in a design water surface elevation that is more conservative than has been used previously in many instances. In addition, the water surface elevations used for design of the HPS are associated with surges and waves produced by hurricane loadings, and will be sustained at peak levels for durations of hours rather than days or weeks.

4. Due to the short time frames associated with hurricane events, the inability to work in hurricane winds, and the general inaccessibility of much of the hurricane protection system during a hurricane, there will be no opportunity to conduct levee patrols or to flood-fight levee or floodwall distress to prevent failure.

5. To the extent possible, design criteria should reflect observed performance of levees and floodwalls that have been subjected to severe storm loadings.

6. One of the lessons from Hurricane Katrina is the need to provide ductility to the design of the HPS levees and floodwalls in order to avoid the brittle failures which occurred when the floodwalls were overtopped. To this end, it is understood that a developing design principle is that regardless of the level of protection being provided, there will always be the potential for a larger storm to create a stage or water level that would reach all the way to the top of the levee or flood wall, and to even overtop the levee or floodwall. Accordingly, levees and flood walls should be designed to withstand water levels reaching all the way to the top with a least a small margin of safety.

### 3.1.4.3 Calculation of Underseepage Factors of Safety

The factors of safety for underseepage at the landside levee toe are computed as follows:

$$FS_g = \frac{\gamma' \times z_t}{\gamma_w \times h_o} \quad , \text{ which is the same as } \quad FS_g = \frac{i_{cr}}{i_e}$$

where:

$FS_g$  = apparent underseepage factor of safety

$\gamma'$  = average effective (or buoyant) unit weight of soil =  $\gamma_{sat} - \gamma_w$

$\gamma_w$  = unit weight of water

$\gamma_{sat}$  = total, or saturated, unit weight of soil blanket

$z_t$  = landside blanket thickness

$h_o$  = excess head (above hydrostatic) at toe

$i_{cr}$  = critical exit gradient =  $\gamma' / \gamma_w$

$i_e$  = exit gradient =  $h_o / z_t$



**Table 3.5(a) -- Criteria for Safety against Erosion and Piping at the Toe of the Levee**

Levee/Wall Application	Minimum Factor of Safety at Levee or Wall Toe <sup>1</sup> for Still Water Level Shown	
	DWSE (Design Water Surface Elevation) <sup>2</sup>	Project Grade <sup>3</sup>
Riverine	1.6	1.3
Coastal	1.6	1.3

NOTES:

1. Minimum factors of safety are based on steady state seepage conditions. Water surfaces in excess of Project Grade, particularly for hurricane loadings, are likely to be of such short duration that steady state conditions will not develop for this extreme condition. Safety is adequately addressed by the criteria for water surface at DWSE and Project Grade.
2. Design water surface elevation (DWSE) is the water level used in deterministic analyses, such as the geotechnical, structural stability, and seepage analyses.
3. Project Grade, sometimes referred to as "top of protection" or "net levee grade," is higher than the design water surface elevation to account for wave run-up, minus overbuild for primary consolidation.
4. Where factors of safety do not satisfy the criteria in Table 3.5(a), relief wells, seepage berms, cutoff walls, or other remediation measures shall be designed to satisfy the criteria shown in Table 3.5(a). If a seepage berm is used, the berm shall also satisfy the criteria for safety at the toe of the berm shown in Table 3.5(b).

**Table 3.5(b) -- Criteria for Safety against Erosion and Piping at the Toes of Seepage Berms for all Riverine and Coastal Levees<sup>1</sup>**

<b>Seepage berm width divided by height of levee<sup>6,7</sup></b>	<b>Minimum Factor of Safety at Toe of Berm<sup>2,3</sup> for Still Water Level Shown</b>	
	<b>DWSE (Design Water Surface Elevation)<sup>4</sup></b>	<b>Project Grade<sup>5</sup></b>
4 <sup>8</sup>	1.5	1.2
8	1.3	1.1
12	1.1	1.0
16 or more	1.0	1.0

**NOTES:**

1. Where a berm is designed to satisfy the criteria for safety at the toe of the levee shown in Table 3.5(a), the factor of safety at the toe of the berm shall satisfy the criteria shown in Table 3.5(b). Factors of safety for intermediate berm widths shall be interpolated between values shown.
2. Minimum factors of safety at the berm toe are based on steady state seepage conditions. Water surfaces in excess of Project Grade, particularly for hurricane loadings, are likely to be of such short duration that steady state conditions do not develop for this extreme condition. Safety is adequately addressed by the criteria for water surface at DWSE and Project Grade.
3. Minimum allowable factors of safety decrease with increasing berm width because damage at the toe of a wider berm poses a smaller threat to the integrity of the levee. With a wider berm, a longer time would be required for erosion to work back to the toe and threaten the integrity of the levee.
4. Design water surface elevation (DWSE) is the water level used in deterministic analyses, such as the geotechnical, structural stability, and seepage analyses.
5. Project Grade, sometimes referred to as "top of protection" or "net levee grade," is higher than the design water surface elevation to account for wave run-up, minus overbuild for primary consolidation.
6. Berm width is measured from levee toe to berm toe.
7. Levee height is defined as the difference in elevation between Project Grade and the prevailing ground surface elevation in the vicinity of the landside levee toe.
8. Where seepage berms are required, the minimum berm width shall be four times the height of the levee.

### 3.2 I-Wall Design Criteria

This section applies to I-Walls that serve as or impact hurricane flood protection.

#### 3.2.1 General Design Guidance

USACE Publications:

- EM 1110-2-2502, Retaining and Flood Walls, Sept. 89
- EM 1110-2-2504, Design of Sheet Pile Walls, Mar. 94
- EM 1110-2-1913, Design and Construction of Levees, Apr. 00
- EM 1110-2-1901, Seepage Analysis and Control for Dams, Apr 93
- DIVR 1110-1-400, Soil Mechanic Data, Dec. 98
- ETL 1110-2-569, Design Guidance for Levee Underseepage, May 05

Computer Software:

- CE Sheet Pile Wall Design/Analysis Program, “CWALSHT”
- Slope Stability Program based on “MVD Method of Planes” (Method of Plane Program and a plotting program is available by contacting New Orleans District. Point of Contact is Paul R. Oakland at [Paul.R.Oakland@usace.army.mil](mailto:Paul.R.Oakland@usace.army.mil).)
- Slope Stability Programs based on “Spencer’s Procedure”

Walls shall be constructed using the latest datum from Permanent Benchmarks certified by NGS - NAVD 88 (2004.65).

The following is a summary of protection heights for various wall systems. Maximum heights refer to exposed height of the protected side of the wall.

- I-Walls – 4 foot maximum height
- T-Walls – Typically 4 foot and greater in height
- L-Walls / Kicker Pile Walls – 8 foot maximum height

Seepage, global stability, heave, settlement and any other pertinent geotechnical analysis shall be performed in order to ensure that the overall stability of the system is designed to meet all Corps criteria.

Geotechnical engineers shall minimize the height of the wall system by designing the largest earthen section that is practical and stable for each individual project.

Flood wall protection systems are dedicated single purpose structures and will not be dependent on or connected to (non-Federal) structural or geotechnical features that affect their intended performance or stability.

In an I-wall, the steel sheet piling is a pile acting to control seepage and provide support to the structure.

I-walls (steel sheet piling) should not be capped until the foundation primary consolidation has occurred from the embankment loading and/or foundation settlement is negligible.

The following criterion is based on experience associated with Hurricane Katrina where some I-walls performed well and others performed poorly. I-walls shall be limited to 4 feet maximum exposed height measured from the protected side. Where existing walls exceed this maximum, fill should be added on the protected side to minimize stick-up and differential fill across the wall should be limited to 2 feet unless additional analysis is performed. I-walls are acceptable as tie-ins to levee embankments. Site and soil conditions will dictate their use in these applications.

### 3.2.2 Geotechnical Design Guidance

#### 3.2.2.1 Global Stability Analysis

I-wall/ Embankment Slope Stability. The Method of Planes and Spencer’s Method shall be used for slope stability analysis. See Table 3.1 for required factors of safety. The system shall be designed for global stability utilizing the “Q” shear strengths for the following load cases: No Tension Crack, With Tension Crack (See Table 3.6 and following Notes).

**Table 3.6 - Global Stability Criteria**

Tension Crack Depth	FOS <sub>min</sub> WL to Top of Wall
None	1.3
CWALSHT or pressure comparison (see note 1)	1.3

**NOTES:**

1. Methods for determining crack depths, particularly for penetrating thin layers of sand, were not well developed at this time. The crack depth is important for computation of seepage, global stability, uplift and piping, and pile tip penetration. For the present design, use the CWALSHT program to determine the tension crack depth by both the fixed and sweep methods utilizing a FOS of 1.0. Use the deeper/lower elevation from the two analyses. If the crack ends only a few feet above the tip, then assume crack extends to tip. If the computed CWALSHT crack depth is above the sheet pile tip, compare the hydro-static water pressure to the at-rest lateral earth pressure ( $\gamma_w h_w$  vs.  $\gamma_s h_s K_o$ ; where  $\gamma_s$  is the saturated unit weight of soil) and assume the crack will propagate to a point of equivalence. The crack may be assumed to be deeper, as described in paragraph Piping and Seepage Analysis, but shall be limited in depth to a point no deeper than the sheet pile tip. Also, because saturated granular soils will not sustain a crack, the designer must develop if the crack will propagate through a thin sand layer to an underlining clay stratum.
2. For global stability, full hydrostatic head shall be used to the depth of the crack at the face of the I-wall (flood side). Protected side piezometric conditions used for stability analysis shall be based on seepage evaluation as described in paragraph Piping and Seepage Analysis below.
3. To model a tension crack that extends to the sheet pile tip, perform the following for global slope stability. For a full clay foundation, remove all soil above the tension crack tip on the flood side of the wall. Check failure mechanisms in the vicinity of the tip at locations above and below the sheet pile tip for failure surfaces that are the most critical. Failure surfaces with lower factors of safety may exist if weaker layers are present near the sheet pile tip.

#### 3.2.2.2 I-Wall Sheet Piling Tip Penetration

Wall Stability. Use the CWALSHT program to determine the required tip by the fixed surface wedge method or Coulomb earth pressure coefficient method and the sweep search method with factors of safety applied to **both** active and passive soil parameters. The deeper computed tip elevation shall be used for design. The sweep method may not run for all cases. If the sweep method does not reach

equilibrium, base the tip elevation on the fixed surface wedge method or Coulomb earth pressure coefficient method. No wall friction or adhesion shall be used in the determination of active or passive earth pressures.

Factor of Safety with Load Cases - (CWALSHT program determines depth of tension crack)

“Q” – shear strengths

a. Cantilever Wall.

FOS = 1.5; Water to Still Water Level (SWL) plus wave load shall be furnished by the hydraulic engineer.

b. Bulkhead Wall.

For walls with fill differential of greater than 2 feet from one side of the wall to the other, a bulkhead analysis should be performed.

FOS = 1.5; Low Water for Hurricane conditions, bulkhead analysis if applicable.

c. Design check.

This is not typical hurricane design case but shall be checked to ensure a bracket of load envelopes and critical loads are considered.

(Case 1.) FOS = 1.3; Water to Top of Wall plus no wave load.

“S” – shear strengths

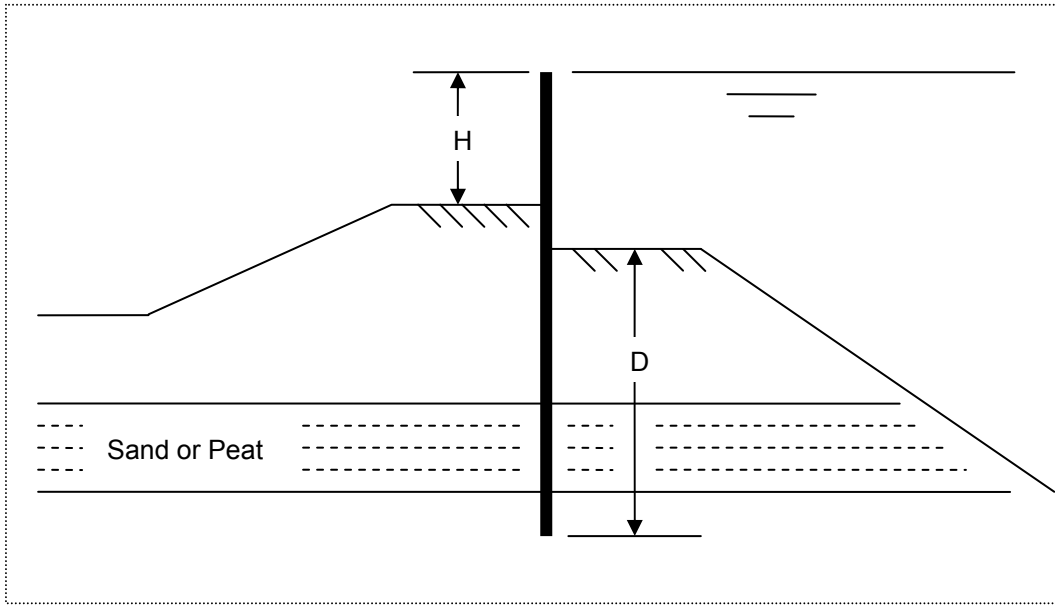
d. FOS =1.5; Normal low water (not Low Water for Hurricane conditions bulkhead analysis) if applicable.

Minimum Tip Penetration. In some cases, especially Q-case penetrations derived for low heads, the theoretical required penetration could be minimal. In order to ensure adequate penetration to account for unknown variations in ground surface elevations and soil, the embedded depth (D) of the sheet pile as shown in Figure 3.3 shall be the greatest penetration of:

a. 3 times the exposed height (H) on the protected side of the wall as shown in Figure 3.3. The embedment of wall shall be based on the lower ground elevation against the wall as shown on the figure below. In the case shown, the lowest ground surface against the wall is on the flood side.

b. 10 feet below the lower ground elevation.

c. Additional depth determined by engineering judgment such selecting appropriate loading cases, penetration to head ratios and stickup ratios, and for extending sheet piling through very shallow sand or peat layers.



**Figure 3.3 Minimum tip penetration depth**

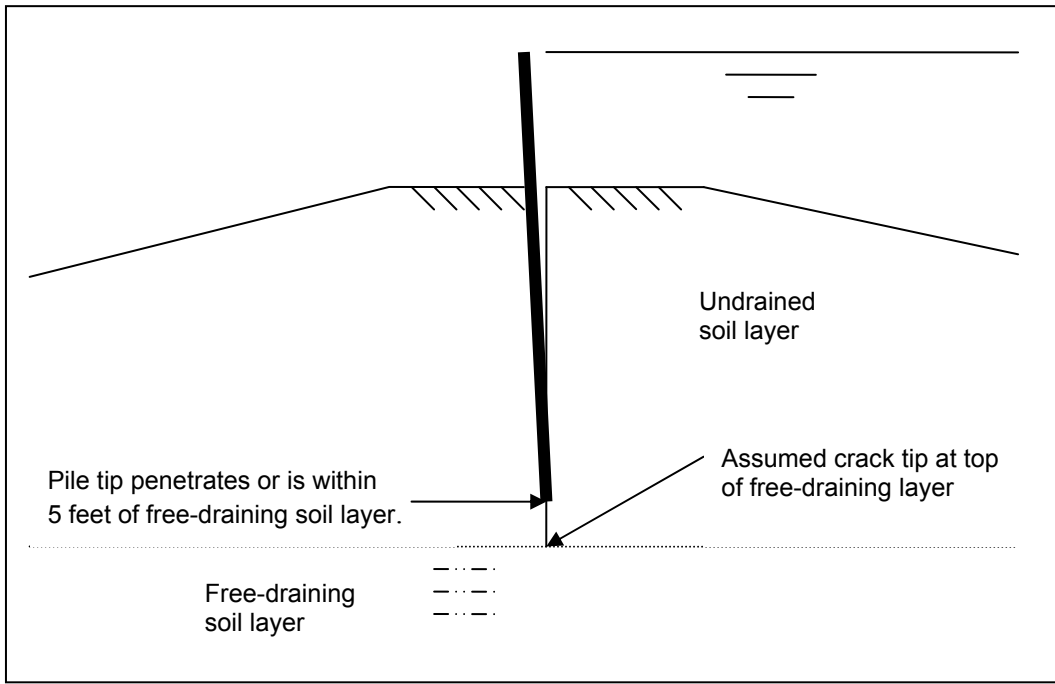
### 3.2.2.3 Piping and Seepage Analysis

Piping. The I-wall must be designed for seepage erosion (piping) along the wall. Analysis shall be based on water to the top of the wall. This analysis can be performed by various methods such as flow nets, Harr's method of fragments, Lane's weighted creep ratio, or finite element methods. Lane's weighted creep ratio, while useful in some circumstances, may not be the most accurate method available to designers. Engineering judgment should be exercised in selecting the most appropriate method of seepage analysis. The seepage analysis shall consider the tension crack which will shorten the seepage path. When the levee and foundation are constructed entirely of clay, the potential for developing a steady state seepage condition along the sheet piling is negligible. However, this should be checked by the designer and engineering judgment should be used to determine if the sheeting piling needs to be extended to meet this criteria.

If an aquifer is present close to the sheet pile tip, or if the sheet pile penetrates the aquifer, a standard seepage analysis as per DIVR-1110-1-400 shall be used to design the seepage resistance of the embankment. In this case, the vertical distance between the tip and the aquifer would be considered to be the flood side blanket thickness. The head at the levee toe can then be calculated using DIVR-1100-1-400 to check for exit gradient and heave.

If the computed crack depth is within 5 feet of an aquifer, the crack shall be assumed to extend to the aquifer (see Figure 3.4). For specific cases where the geology of the foundation is well known and the designer is confident that the sand strata is more than 2.0 feet below the tip of the sheet pile, the crack shall

extend only to the depth calculated from Table 3.6. A well know geology shall have field investigations (boring and/or CPT data) spaced closer than 100 feet.



**Figure 3.4 Computed crack depth near an aquifer**

Seepage. Seepage analysis should be checked in accordance with the applicable portions of EM 1110-2-1901, DIVR 1110-1-400, and ETL 1110-2-569.

### 3.2.2.4 Heave Analysis

If applicable, heave analysis should be checked. The required factor of safety for a total weight analysis is 1.20. The tension crack shall be considered in this analysis. For tension cracks to the sheet pile tip elevation, the pressure at the sheet pile tip should be based on the full hydrostatic head. The factors of safety for computing heave are defined as:

$$FS_h = \frac{\gamma_{sat} \times z}{\gamma_w \times hw};$$

$\gamma_{sat}$  = saturated unit wt. soil

$\gamma_w$  = unit wt. of water

$z$  = overburden thickness

$hw$  = pressure head



**3.2.2.5 Deflections**

The determination of allowable deflection has not yet been made and will be finalized after further evaluating the E-99 test wall and IPET results. Until that time, deflections will be considered to be satisfactory when the exposed I-wall heights are limited to 4 feet as described in Section 3.2.1 General Design Guidance.

**3.3 Pile Capacity**

Piles shall be designed in accordance with EM 1110-2-2906. The following are typical values used by MVN:

- For cohesion vs. adhesion, MVN uses Figure 4-5a on page 4-15
- Limited overburden stresses to 3500 psf for both the "Q" and "S" case.
- No tip bearing for Q-case in clays where cohesion is less than 1000psf
- Typical values for SM = 30° and SP= 33° for no shear testing
- S-Case in clay should be evaluated in all design cases.
- Pile batter shall not be considered in the determination of skin friction capacity.

Recommended factors of safety for MVN projects are shown below. In addition, see Structural Section for additional Factor of Safety for specific load cases.

**Table 3.7 -- Recommended minimum FOS**

	<b>With Pile Load Test</b>	<b>W/O Pile Load Test</b>
Q-Case	2.0	3.0
S-Case	1.5	1.5

**3.3.1 Concrete and Timber Piles**

Typical Values MVN uses for Concrete and Timber piles are shown (see EM for range of values page 4-12 and 4-13):

**Table 3.8 – Q-Case Pile Design Values**

<b>Q-Case</b>					
Type	phi	Kc	Kt	Nc	Nq
Clay	0	1	1.0	9	1.0
Silt	15	1	0.5	12.9	4.4
Sand	30	1.25	0.7	0	22.5

**Table 3.9 – S-Case Pile Design Values**

<b>S-Case</b>					
Type	phi	Kc	Kt	Nc	Nq
Clay	23	1	0.7	0	10
Silt	28	1	0.5	0	19.5
Sand	30	1.25	0.7	0	22.5

### 3.3.2 Steel Piles

For granular soil to steel use delta value approx 2/3 phi

For Steel H-piles. Note: 1/2 of the surface area is soil against steel and the other half is soil against soil. For end bearing use the area of the steel or approximately 60% of the end block area.

### 3.4 T-Wall and L-Wall/Kicker Pile Wall Design Criteria

This section applies to T-Walls and L-Walls that serve as or impact hurricane flood protection.

#### 3.4.1 General Design Guidance

USACE Publications:

- EM 1110-2-2502, Retaining and Flood Walls, Sept. 89
- EM 1110-2-2906, Design of Pile Foundations, Jan. 91
- EM 1110-2-2504, Design of Sheet Pile Walls, Mar. 94
- EM 1110-2-1913, Design and Construction of Levees, Apr. 00
- EM 1110-2-1901, Seepage Analysis and Control for Dams, Apr 93
- EM 1110-2-2100, Stability Analysis of Concrete Hydraulic Structures, Dec 05
- DIVR 1110-1-400, Soil Mechanic Data, Dec. 98

- ETL 1110-2-569, Design Guidance for Levee Underseepage, May 05

Computer Software:

- CE Sheet Pile Wall Design/Analysis Program, “CWALSHT”
- Slope Stability Program based on “MVD Method of Planes” (Method of Plane Program and plotting program is available by contracting New Orleans District. Point of Contact is Paul R. Oakland at [Paul.R.Oakland@usace.army.mil](mailto:Paul.R.Oakland@usace.army.mil) .
- Slope Stability Programs based on “Spencer’s Procedure”

Walls shall be constructed using the latest datum from Permanent Benchmarks certified by NGS as NAVD88 (2004.65). See Section 9 Surveys for additional information.

The following is a summary of protection heights for various wall systems. Maximum heights refer to exposed height of the protected side of the wall.

- I-Walls – 4 foot maximum height
- T-Walls – Typically 4 foot and greater in height
- L-Walls / Kicker Pile Walls – 8 foot max. height and no unbalanced loads

T-Walls are the preferred walls where there is the potential for barge/boat impact loading or unbalanced forces resulting from a deep-seated stability analysis.

L-Walls may also be used where there is the potential for barge/boat impact loading; however, they shall not be used where an unbalanced force is present resulting from a deep-seated stability analysis.

Seepage, global stability, heave, settlement and any other pertinent geotechnical analysis shall be performed in order to ensure that the overall stability of the system is designed to meet all USACE criteria.

Geotechnical Engineers shall minimize the height of the wall system by designing the largest earthen section that is practical and stable for each individual project.

Flood wall protection systems, are dedicated single purpose structures and shall not be dependent on or connected to other (non-Federal) structural or geotechnical features that affect their intended performance or stability.

In an L-Wall, the steel sheet piling is a pile acting to control seepage and provide support to the structure.

The foundation support piles shall be designed such that settlement is limited to an acceptable amount and differential settlement is negligible. Vertical movement of the cap should be less than 0.50” and horizontal deflection of the cap should be limited to 0.75”. Deviations shall be approved in advance by the

USACE engineer of record. Where levees will be raised or new embankment constructed, the adverse effects of foundation consolidation must be considered which includes drag forces on both the sheet pile cut-off and support piles. In addition, these drag forces must be considered in settlement calculations.

### **3.4.2 Geotechnical Design Guidance**

#### **3.4.2.1 Global Stability Analysis**

Stability. Spencer's Procedure shall be used for slope stability analysis incorporating Factors of Safety (FOS) for two (2) Load Conditions according to Table 3.1.

- Condition 1 - water at Still Water Level (SWL)
- Condition 2 - water at the top of the wall

When feasible, stability berms shall be designed to counter unbalanced forces within the foundation beneath the floodwall due to unacceptable FOS. The unbalanced force is determined as the additional resistive horizontal force necessary to achieve the required FOS. Determination of the magnitude, direction, and location of the unbalanced force is described in Section 3.4.3, T-Wall Design Procedure.

#### Stability Analysis Results:

(Case 1) If there are no unbalanced forces, the structure is required to carry only the net at-rest loads acting above the base. These loads must be carried axially by the foundation piles below the base. Therefore, for a T-Wall, the sheet piling section and tip elevation, below the base, is determined only by seepage analysis or erosion control. See Section 3.4.3 for specific T-Wall design procedure. For an L-Wall, the sheet piling section and tip elevation, below the base, is not only determined by seepage analysis or erosion control, it must also resist tension and compression forces acting in conjunction with the foundation kicker pile.

(Case 2) If there are unbalanced soil loads, see Section 3.4.3 for specific T-Wall design procedure. L-Walls are not allowed where unbalanced loads exist.

#### **3.4.2.2 T-Wall Sheet Piling Cut-off Tip Penetration**

Sheet pile tip elevations shall meet criteria for seepage control and at a minimum, shall extend 5 ft. beneath the T-wall base.

Engineering judgment shall be used to determine the final penetration such as extending through very shallow sands or peat layers. When two T-Wall sections with different ground surface, base slab and required sheet pile tip elevations are to be constructed adjacent to one another, a minimum overlap of 50 feet of the deeper required sheet pile tip elevation shall be incorporated. For relatively short

reaches of floodwall with differing sheet pile requirements, such as for Pump Station Fronting Protection, the worst case required sheet pile penetration shall be used for every floodwall part of those structures.

If unbalanced forces exist, as determined by the global stability analysis, then the sheet pile tip will be determined by the anchored bulkhead analysis above.

#### **3.4.2.3 L-Wall Sheet Piling Tip Penetration**

Sheet pile tip elevations shall meet criteria for seepage control and at a minimum, shall have either a 3 to 1 penetration ratio of wall height to depth or shall extend 10 ft. beneath the L-wall base, whichever is greater.

Sheet pile tip elevation shall provide required compression and tension resistance required from T-wall analysis (see below).

Engineering judgment shall be used to determine the final penetration such as extending through very shallow sand or peat layers.

The ultimate tension and compression capacity of the sheet pile shall be the allowable shaft resistance on both sides of the sheet using the projected flange line, except in the upper 10' below the slab. In this top 10', only the protected side of the sheet pile shall be considered effective. A Factor of Safety of 3.5 shall be applied to the ultimate capacity to arrive at the allowable capacity due to reduction inherent when installing sheet piling with vibratory hammers. A Factor of Safety of 2.5 may be used in both compression and tension when a pile load tests is performed.

#### **3.4.2.4 T-Wall and L-Wall Pile Foundation Tip Penetration**

This section applies to PPC, Steel H and Pipe sections.

Pile lengths will be based on soil boring data from existing contracts or, if time permits, new borings. If existing pile test data are available, they can be used to determine pile lengths. For tension and compression ultimate capacity, FOS = 2.0 with static pile test data, FOS = 2.5 with pile dynamic analysis (PDA) or FOS = 3.0 without pile test data. (See table in Structural Design Analysis section for additional FOS.)

#### **3.4.2.5 Piping and Seepage Analysis**

Piping (cutoff-wall tip elevation). T-walls and L-Walls must be designed for piping erosion along the base of the pile founded wall. Analysis shall be based on water to the top of the wall. This analysis can be performed by various methods such as Lane's weighted creep ratio, flow nets, Harr's method of fragments, or finite element methods. A design procedure used for evaluating piping erosion

for clays, silts, and sands directly beneath pile-founded L-walls and T-walls for hurricane protection is to use Lane's weighted creep ratio for a seepage path along the sheet pile wall. Engineering judgment should be exercised in selecting appropriate weighted creep ratio values for this analysis and using the weighted creep length based on flow path through the different foundation materials.

Seepage. Seepage analysis through the foundation should be checked in accordance with the applicable portions of EM 1110-2-1901, DIVR 1110-1-400, and ETL 1110-2-569. For computing the seepage gradient Factor of Safety see Section 3.1.4.

### **3.4.2.6 Heave Analysis**

If applicable, heave analysis should be checked. Safety Factor for Total Weight analysis is 1.2. For computing heave Factor of Safety see Section 3.2.2.4.

### **3.4.3 T-Wall Design Procedure**

#### **3.4.3.1 Description of Design Method**

This design method evaluates the improvement in global stability by including the allowable shear and axial force contributions from the foundation piles together with the soil shear resistance in a limit equilibrium slope stability analysis (Spencer method of analysis). This procedure has the ability to account for both the reinforcing effect the piles have on the foundation soils and ability to determine safe allowable shear and axial forces for the piles. This design procedure is a supplement to existing Hurricane and Storm Damage Reduction System design criteria and EM 1110-2-2906, which shall govern for design aspects not specifically stated herein.

The design procedure requires an initial pile layout to get started. The initial pile layout is designed similarly to the current MVN procedure in that slope stability is checked for the T-wall configuration neglecting piles, and also the water loads directly on the wall, and a balancing force is computed to achieve the required global factor of safety (termed the unbalanced force). A portion of the unbalanced force is applied to the pile cap and a CPGA analysis is completed.

The initial CPGA based design is verified by applying the unbalanced force as an equivalent "Distributed Load" to the foundation piles in an Ensoft Group Version 7.0 model (Group 7). Loads are also applied to the wall base and stem and the axial and shear responses for each pile are then compared with the allowable pile forces found from load tests or from computations. Limiting axial and lateral loads according to load test data helps minimize deflection to tolerable limits. Deflections of the T-wall computed from the Group 7 analysis are also compared to allowable deflections and bending moments and shear are checked to verify that they are within allowable pile limits.

Note that all CPGA designs shall include unfactored service loads and the Group 7 input shall include unfactored soil properties.

### 3.4.3.2 General

The procedure is applicable to T-Walls. Adaptations for drainage structures, floodgates, and extended foundations are discussed in Section 3.4.3.4. Fronting walls, constructed separate from existing structures (i.e. Pump Stations), present other analysis concerns that are discussed in Section 3.4.3.5.

For any design the subsurface characteristics must be properly identified. This includes stratigraphy, material properties and groundwater conditions. Material properties for wall design include unit weight, shear strength (drained or undrained depending on loading condition), and horizontal soil modulus. To complete the pile design, proper group reductions must also be considered. Reductions of soil stiffness are applied for pile group effects as noted in this chapter. No reductions are recommended for cyclic loading for several reasons:

- Analyses to date indicate that wall and soil loadings are transmitted axially to the foundation piles and changes in the lateral soil stiffness do not significantly impact the design.
- The Young's modulus of the soil between the wall base and the critical failure surface is reduced in this design procedure based on the global stability. Where global stability factors of safety are below one, the soil stiffness in this zone is neglected. Where the factors of safety exceed criteria, full soil stiffness is used. The soil stiffness is linearly proportional between these limits when the computed factor of safety is between one and the required factor of safety. In this way the soil stiffness is already being reduced and further reduction is felt to be too conservative.
- In most instances the T-walls are above normal water levels and are not routinely subjected to wave, tide or pool fluctuations and the associated large number of loading cycles.

### 3.4.3.3 Design Steps

#### Step 1. Initial Slope Stability Analysis

1.1. Determine the critical **non-circular failure** surface from a slope stability analysis for loading to the SWL and to the Top of Wall using a software program capable of performing Spencer's method with a robust search procedure (hereinafter termed Spencer's method). Sufficient deterministic and finite element analyses have been completed on varying soil profiles to assure that the non-circular surfaces shall govern the stability assessment. Furthermore,

numerical modeling has indicated that soil displacement is nearly horizontally along the critical failure surface. The slope stability analysis should be performed with only water loads acting on the ground surface flood side of the heel of the T-wall because these are the loads that the foundation must resist to prevent a global stability failure. The analysis should not include any of the water, soil, or surcharge loads acting directly on the structure because these loads are presumed to be carried by the piles to deeper soil layers.

Global stability of T-walls includes the foundation materials on the protected side of the wall. If those materials were removed the walls would be required to support a larger unbalanced load. If the foundation on the protected side of the T-wall (like an existing slope towards an inland ditch or canal) is not stable or does not satisfy required factors of safety it must either be improved to meet criteria or be partly removed from the global stability model when calculating the unbalanced load. Landward berms and channel local slope stability analysis shall satisfy the applicable FOS listed in this chapter in order to be included in the global stability analysis.

1.2. If the factor of safety of the critical failure surface is greater than required (see Section 3.1.2.2), a structural analysis of the T-wall system shall be completed using a group pile analysis program (like CPGA or Group 7) using only the water loads and at rest pressures applied directly to the structure. If the lowest factor of safety is less than required, then proceed to Step 2. The factor of safety and defining failure surface coordinates should be noted for use in Step 2.

1.3. As stated in Step 1.1 above, only non-circular failure planes shall be investigated and shall be horizontal along the critical failure surface. This horizontal distance is referred to as the neutral block. **The neutral block shall have a minimum dimension of the greater of 0.7 H or the base length of the T-Wall or structure. H is defined as the vertical distance from the failure surface to the intersection of the failure plane with the ground surface (see Figure 3.5).**

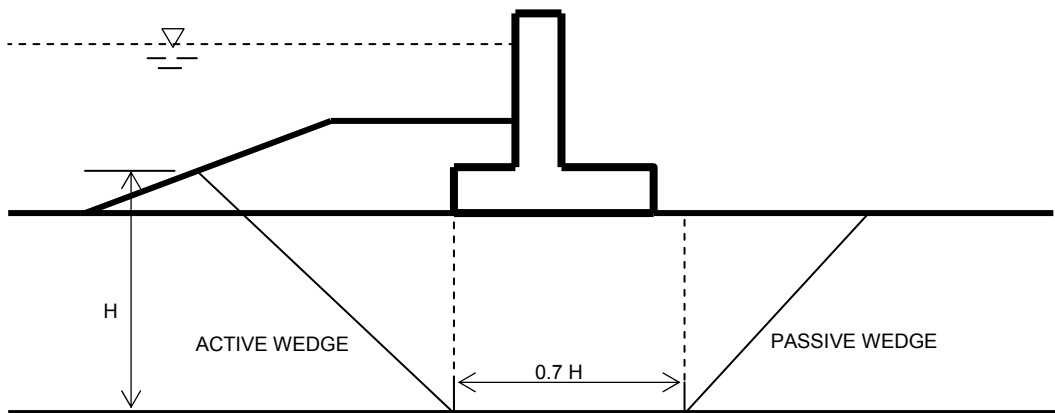


Figure 3.5 -- Typical Failure Plane Beneath a T-Wall



1.4. Designers shall also perform a Method of Planes (MOP) analysis as a design check. This is required regardless if an unbalanced load exists or not. The MOP Factors of Safety are 1.3 for water at the Still Water Level (SWL) and 1.2 for water at Top of Wall (TOW). MOP results (including final factors of safety, failure surface geometries, and any unbalanced loads) shall be compared to the Spencer's analysis that utilize a FOS of 1.5 with Water at SWL and 1.4 with Water at TOW. The Spencer's method remains the design tool.

### Step 2. Unbalanced Force Computation

2.1. Determine the unbalanced forces (for both loading to Still Water Level (SWL) and Top of Wall (TOW)) required to achieve the target factor of safety using Spencer's method with a non-circular failure surface search. The unbalanced force shall be applied as a horizontal line load at a location having an X-coordinate at the heel of the wall or simply beneath the base of the wall. The Y-coordinate shall be located at an elevation that is half-way between the ground surface at the heel of the wall and the lowest elevation of the critical failure surface beneath the wall base from Step 1.

The unbalanced load is arrived at through a trial and error process where the load is varied until the desired factor of safety is achieved. The failure surface found in Step 1 is "reanalyzed" with the specified line load so that the largest unbalanced force is computed. The unbalanced load is determined for both conditions: the slip surface with lowest factor of safety and the slip surface with the highest unbalanced load. The unbalanced load and the defining failure surface coordinates should be noted for use in subsequent steps. The largest unbalanced load does not necessarily coincide with the failure surface with the lowest factor of safety; therefore, multiple failure surfaces at various elevations must be analyzed to determine those corresponding unbalanced forces. The unbalanced load is determined for both conditions: the slip surface with lowest factor of safety and the slip surface with the highest unbalanced load. The unbalanced load and the defining failure surface coordinates should be noted for use in subsequent steps.

**2.2. The critical failure plane is defined as the failure surface that produces the greatest unbalanced load. This failure surface is NOT necessarily the failure surface with the lowest factor of safety. Where unbalanced loads are present, all axial pile capacity developed above the critical failure plane shall be disregarded.**

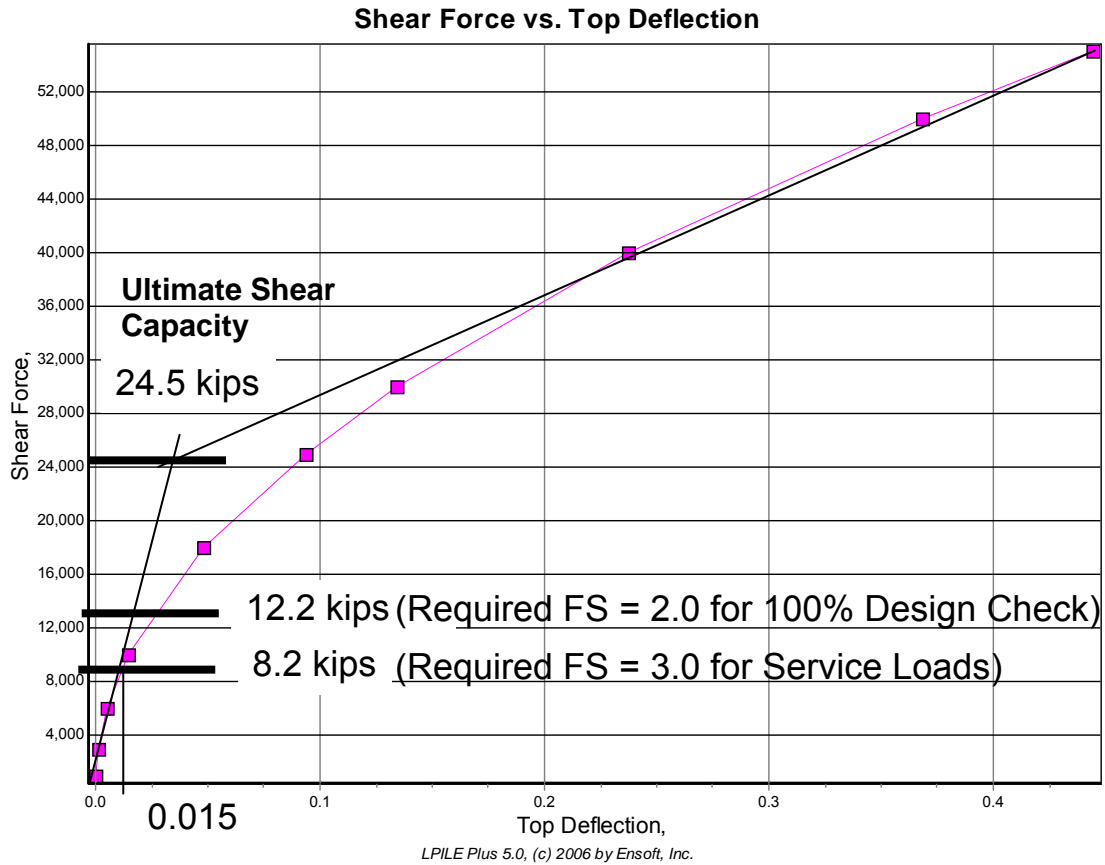
### Step 3. Allowable Pile Capacity Analyses

3.1. Establish allowable single pile axial (tension; compression) capacities. Axial capacity shall be determined according to Section 3.3. Axial capacities must be

determined for tensile and compressive piles. The contribution of skin friction should not be accounted for above the critical failure surface found in Step 2 in the determination of the axial capacity. Allowable axial loads may also be found using data from pile load tests and applying appropriate factors of safety after the ultimate load has been reduced to neglect the skin friction effects capacity above the critical failure surface. No cyclic reductions need to be applied to the capacities.

3.2. Compute allowable shear loads on the pile at the critical failure surface. Lateral shear loads have historically not been computed; instead deflections are calculated at a working stress level and are required to be less than specified limits. For this procedure, in addition to the traditional check of pile cap displacements, allowable lateral loads are now used as a design check. The Ensoft program LPILE or the Corps program COM624G can be used to compute allowable lateral shear in the pile using these steps:

- a. Analyze the pile with a free head at the critical failure surface. To account for overburden pressure, make the top foot a layer with a unit weight equal to the effective stress due to the overburden.
- b. Run a series of progressively higher lateral loads on the pile, with moment equal to zero, and plot load vs. deflection results. The pile will fail when deflections increase greatly with increasing load. The load vs. deflection curve should be terminated at the load at which yield in the pile is reached. Draw lines roughly tangent to the initial and final portions of the curve. The point of intersection of the two tangent lines is the ultimate shear strength. An example of this is shown in Figure 3.6.
- c. Divide the shear load by the same factors of safety used to compute allowable axial capacity from calculated ultimate values.



**Figure 3.6** Example of computation of ultimate shear load in the pile from a load vs. deflection curve developed using LPILE. FOS varies depending on load case.

Step 4. Initial T-wall and Pile Design

4.1. Use CPGA to analyze all load cases and perform a preliminary pile and T-wall design comparing computed pile loads to the allowable values found in the preceding step. For this analysis the unbalanced force is converted to an “equivalent” force applied to the bottom of the T-wall. It is calculated by a ratio derived by computing equivalent moments at the location of the maximum moment in the pile below the critical failure surface. The location of maximum moment is approximated from Figure 6.9 of “Pile Foundations in Engineering Practice” by Shamsher Prakash and Hari D. Sharma as being about equal to the stiffness factor, R, below the ground surface. The equivalent force (excluding the unbalanced force above the base of the T-Wall),  $F_{cap}$ , is calculated as shown below (see Figure 3.7):

$$F_{cap} = F_{ub} \left[ \frac{\left( \frac{L_p}{2} + R \right)}{(L_p + R)} \right] \frac{L_p}{L_u} \quad (1)$$

Where

$F_{ub}$  = unbalanced force computed in Step 2.

$L_u$  = distance from top of ground to the lowest El. of critical failure surface (in)

$L_p$  = distance from bottom of footing to lowest el. of crit. failure surface (in)

$$R = (EI / Es)^{1/4} \quad (2)$$

$E$  = Modulus of Elasticity of Pile (lb/in<sup>2</sup>)

$I$  = Moment of Inertia of Pile (in<sup>4</sup>)

$Es$  = Modulus of Subgrade Reaction (lb/in<sup>2</sup>) below critical failure surface. In New Orleans District this equates to the values listed as  $K_{HB}$ .

$K_{HB}$  is calculated as shown in Section 3.

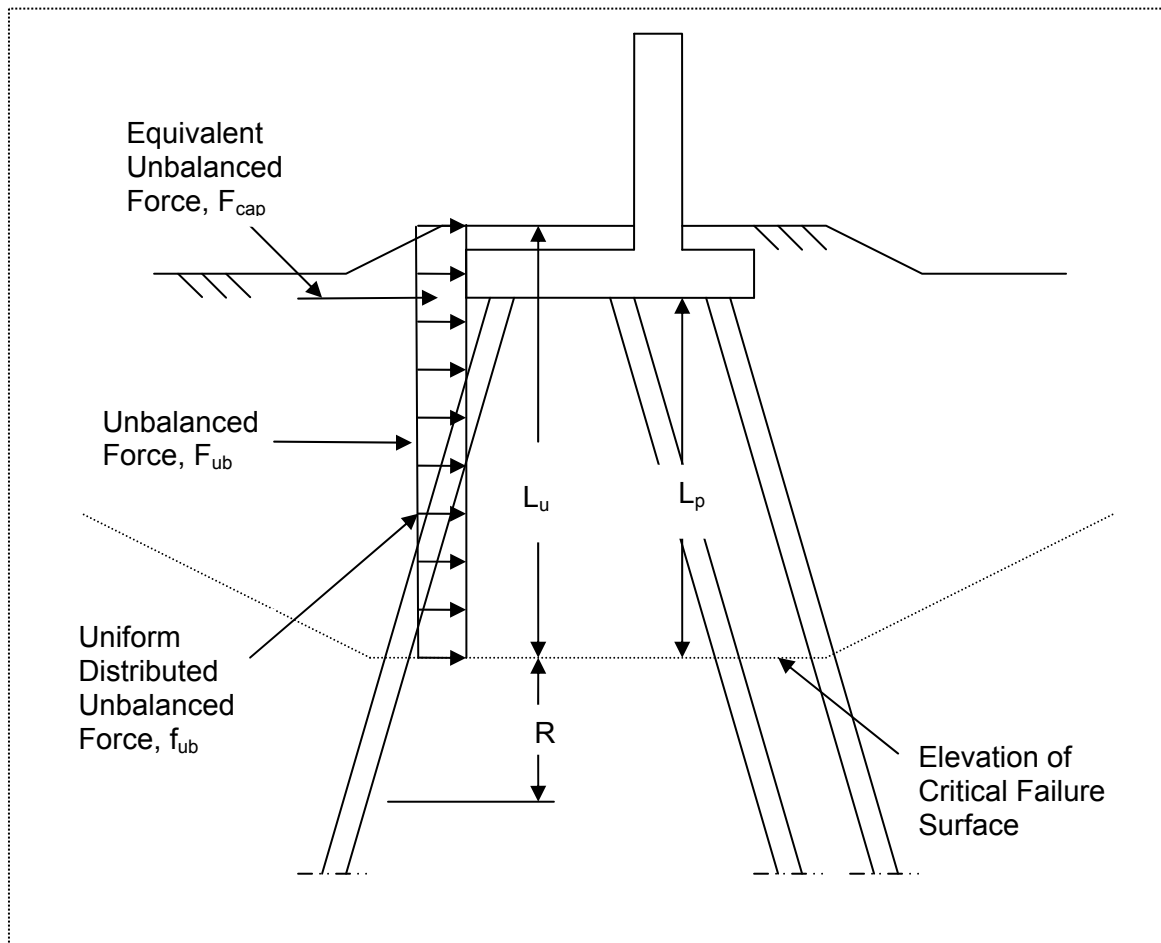


Figure 3.7 Unbalanced Forces.

Comments:

- a. The above procedure does not directly account for the unbalanced force that's transferred down the pile and into the soil below the critical failure surface by lateral soil resistance. This procedure has been found to be adequate for computing axial loads in the piles in order to determine a preliminary pile layout. Forces not accounted for with this procedure will be computed directly in Step 5.
- b. The lowest elevation of the critical failure surface is used, regardless of where the computed failure surface actually intersects the piles. This simplification is made because the soil-structure modeled with this procedure is an approximation and research shows that the presence of the piles will influence the actual location of the critical failure surface so it is something like that shown in Figure 3.7. This procedure is considered to provide acceptable design forces in the piles.

4.2. In CPGA, the top of soil will be modeled at the ground surface, and the subgrade modulus,  $E_s$ , is reduced with reduced global stability factors of safety to account for lack of support from the less stable soil mass located above the critical failure plane. For cases where the global factor of safety without piles is less than 1.0,  $E_s$  is input at an extremely low value, such as 0.00001 ksi (CPGA will not run with  $E_s$  set at 0.0). For conditions where the factor of safety is between 1.0 and the target factor of safety,  $E_s$  is computed by multiplying the percentage of the computed factor of safety between 1.0 and the target factor of safety by the actual estimated value of  $E_s$ . For example, if the  $FS = 1.0$ ,  $E_s$  is input as 0.00001. If the  $FS = 1.2$ , the target factor of safety is 1.5, and the estimated value of  $E_s$  below the foundation is 100 psi,  $E_s$  is input at 40% of the actual estimated value, 40 psi. This accounts for the fact that with higher factors of safety the unbalanced force is a small percentage of the total force, and the soil is able to resist some amount of the lateral forces from the wall. Although  $E_s$  is reduced, the full pile length is considered braced provided the FOS is above 1.0 or the sheet piling is extended as stated in Step 4.4 below. One reduced value of  $E_s$  is used throughout the depth of the pile. There is no distinction in values between the leading and trailing rows.

For certain cases with shallow critical failure surfaces, the procedure in the previous paragraph may not match well with the Group results found in later steps. For these cases, the CPGA model may be created with the ground level set at the level of the critical failure surface and the T-wall suspended above it at the actual footing elevation. The soil modulus at the critical failure surface is used for this model. There is no reliable method to account for factors of safety greater than 1.0 with this method however.

4.3. No reductions to the subgrade modulus are required for cyclic loading. Group reductions based on pile spacing are also applicable. However, for

monoliths containing battered piles, further refinement of the  $E_s$  value for Step 4 calculations may not be required for several reasons:

- The horizontal component of Battered Piles provides most of the lateral resistance.
- The  $E_s$  reduction used in the Step 4.2 conservatively uses the same reduced  $E_s$  for trailing rows as leading rows.
- The governing load cases will be more accurately analyzed in Step 5.

When used, Group reduction factors ( $R_g$ ) to be applied to subgrade modulus shall be computed as shown below:

Subgrade Modulus reductions are computed as follows:

For loading perpendicular to the loading direction:

$$R_{ga} = 0.64(s_a/b)^{0.34} \quad ; \text{ or } = 1.0 \text{ for } s_a/b > 3.75$$

Where:

$s_a$  = spacing between piles perpendicular to the direction of loading (parallel to the wall face). Normally piles should be spaced no closer than 5 feet on center.

$b$  = pile diameter or width

For loading parallel to the loading direction:

For leading (flood side) piles:

$$R_{gbl} = 0.7(s_b/b)^{0.26} \quad ; \text{ or } = 1.0 \text{ for } s_b/b > 4.0$$

For trailing piles, the reduction factor,  $R_{gbt}$  is:

$$R_{gbt} = 0.48(s_b/b)^{0.38} \quad ; \text{ or } = 1.0 \text{ for } s_b/b > 7.0$$

Where:

$s_b$  = spacing between piles parallel to the direction of loading (perpendicular to the wall face. Note:  $s_b$  can be measured 5 pile diameters below the bottom of the cap, making pile rows trailing others battered in the opposite direction to normally be able to be considered as leading piles.

$b$  = pile diameter or width

4.4. Sheet piling shall be included and designed to control seepage. Sheet pile shall be designed for seepage in accordance with Section 3.4.2. When unbalanced loads exist, cutoff sheet piling shall be extended 5 feet below the critical failure plane determined in Step 2. The sheet piling shall be a PZ-22 section or equivalent, structural analysis is not required. The sheet piling curtain wall

provides the added benefit of confining the soil wedge such that the pile shall be considered braced full length about both axis regardless of the stability factor of safety.

4.5 This paragraph addresses the resistance to soil flow of the failure wedge through the pile foundation. Storm surge loading on the soil beyond the relieving base width of the T-wall superstructure results in a passive loading on the foundation piles where the soil tends to push through the piles rather than an active loading where the piles tend to push through the soil. The foundation piles need to be checked for resistance to flow through, which is a function of pile spacing, magnitude of load and soil shear strength, and number of pile rows. Pile spacing perpendicular to the load should generally be limited to no more than seven times the pile diameter. To resist flow-through, the passive load capacity of the piles ( $P_{all}$ ) is checked against the unbalanced loading. In addition, this check will define the upper limit of possible loading on the flood side row of piles and may lead to redistribution of the unbalanced load for later Group 7 analysis. The procedure for performing this check is set up to evaluate this per monolith or by pile spacing (for uniformly spaced piles) as follows:

- a. Compute capacity of the flood side pile row using a basic lateral capacity:

$$\sum P_{all} = \frac{n \sum P_{ult}}{1.5} \quad (3)$$

Where:

$n$  = number of piles in the row perpendicular to the unbalanced load within a monolith. Or, for monoliths with uniformly spaced pile rows,  $n = 1$ .

$\sum P_{ult}$  = summation of  $P_{ult}$  over the height  $L_p$ , as defined in Step 4.1  
 For single layer soil is  $P_{ult}$  multiplied by  $L_p$   
 For layered soils,  $P_{ult}$  for each layer is multiplied by the thickness of the layer and added over the height  $L_p$

$P_{ult} = Rf(9S_u b)$

$S_u$  = soil shear strength

When there are multiple soil strata between the base of the structure and the critical failure plane being analyzed,  $S_u$  shall be calculated as the weighted average of  $S_u$  of each stratum above that failure plane.

$b$  = pile width

$Rf$  = group reduction factor for pile spacing parallel to the load are as follows:

For leading (flood side) piles:

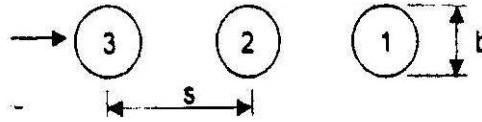
$$R_f = 0.7(s/b)^{0.26} \quad ; \text{ or } = 1.0 \text{ for } s/b > 4.0$$

For trailing piles, the reduction factor,  $R_f$ , is:

$$R_f = 0.48(s/b)^{0.38} \quad ; \text{ or } = 1.0 \text{ for } s/b > 7.0$$

Where:

$s$  = spacing between piles parallel to the loading



**Figure 3.8 Spacing between piles.**

No reduction is considered for the pile spacing perpendicular to the load. Group effects do not need to be considered between pile rows battered in opposite directions (battered away from each other). A trailing row staggered from a leading row may be treated as a leading row, but additional rows should be treated as trailing. The spacing between lead pile and the staggered pile (row spacing), in the direction of the load, shall be equal to or less than the column spacing of the leading piles.

b. Compute the unbalanced unit load on the piles ( $F_p$ ) to check against  $\Sigma P_{all}$ :

$$F_p = wf_{ub}L_p \quad (7)$$

$w$  = Monolith width. Or, for monoliths with uniformly spaced pile rows,  $w$  = the pile spacing perpendicular to the unbalanced force ( $s_t$ )

$$f_{ub} = \frac{F_{ub}}{L_u} \quad (8)$$

$F_{ub}$  = Net unbalanced force per foot from Step 4.1

$L_u$  and  $L_p$  as defined in Step 4.1

If layered soils exist, this check can be made by summing  $P_{all}$  over the length of the pile from the bottom of the wall to the lowest elevation of the critical failure surface ( $L_p$ , fig. 2) (i.e.,  $\Sigma P_{all}$ ) and comparing it to  $f_{ub}$  multiplied by  $L_p$ .



c. The number of piles is adequate to resist flow-through if  $\Sigma P_{all}$  for the flood side piles exceeds  $F_p/2$ . If  $F_p/2$  exceeds  $\Sigma P_{all}$  for the flood side piles, then compute  $\Sigma P_{all}$  for all rows of piles. If  $\Sigma P_{all}$  is less than  $F_p$ , then the pile foundation will need to be modified (decreasing transverse pile spacing and/or increasing pile rows) until this condition is met.

The flow is resisted by the full  $\Sigma P_{all}$  of the floodside row and the balance distributed to all piles behind the flood side row as modified by  $R_f$  for trailing piles. Irregular pile layouts with rows that have far fewer piles than other rows should not have increased load on the pile to account for greater lateral spacing.

4.6. For an additional flow-through mechanism check, compute the ability of the soil to resist shear failure between the pile rows from the unbalanced force below the base of the T-wall,  $f_{ub}L_p$ , using the following equation:

$$f_{ub}L_p \leq \frac{A_p S_u}{FS} \left[ \frac{2}{(s_t - b)} \right] \quad (9)$$

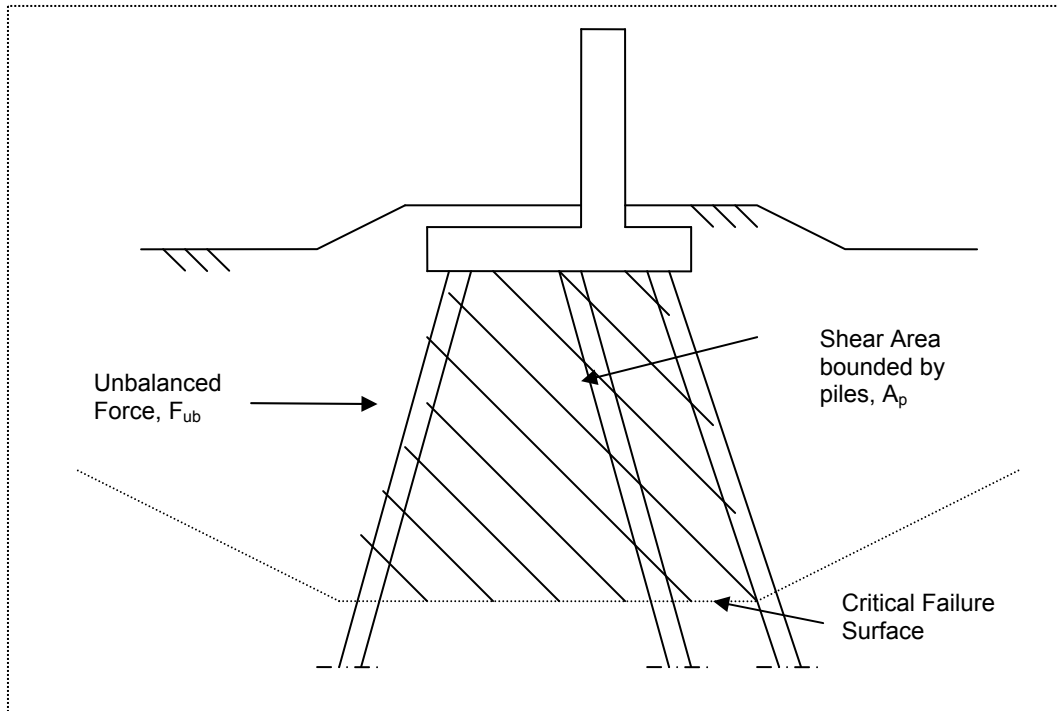
Where:

$A_p S_u$  = The area bounded by the bottom of the T-wall base, the critical failure surface, the upstream pile row and the downstream pile row multiplied by the shear strength of the soil within that area. For layered soils, the product of the area and  $S_u$  for each layer is computed and added for a total  $A_p S_u$ . See Figure 3.9.

$FS$  = Target factor of safety used in Steps 1 and 2.

$s_t$  = the spacing of the piles transverse (perpendicular) to the unbalanced force

$b$  = pile width



**Figure 3.9 Area for soil flow-through shear check.**

Note: The sheet pile is conservatively neglected for this computation.

#### Step 5. Pile Group Analysis (all loads)

5.1. To verify the preliminary CPGA design, Group 7 (Ensoft Group Version 7.0) is used to check pile loads and stresses. All loads, including the unbalanced loading, are applied to the pile foundation. Only load cases controlling deflections and pile loads in Step 4 need to be checked. It is expected that the critical load cases checked will include the unbalanced force found for loading at the SWL and the Top of Wall.

5.2. Water pressures, at rest soil pressures, concrete weight, vessel impact, etc. are applied directly to the structure. The unbalanced load is applied as uniformly distributed along the length of the bearing piles located above the critical failure plane.

5.3. For the pile group analysis, develop a Group 7 model that incorporates the water and soil loads applied directly to the wall base and stem and also include the computed unbalanced force as distributed loads acting on the piles. At this point, the pile foundation has also been adjusted as needed to resist soil flow through as required in Steps 4.5 and 4.6. The total distributed load on the piles ( $F_p$ ) was defined in Step 4.5. Distribution of unbalanced loading onto the rows of piles is as follows:

- If the total ultimate capacity ( $n\Sigma P_{ult}$ ) of the flood side pile row is greater than 50%  $F_p$ , then 50% of  $F_p$  is applied to the flood side row of piles as a uniform load along each pile equal to  $0.5f_{ub}S_t$  (variables are defined in Step 4.5), and the remaining 50% of  $F_p$  is divided evenly among the remaining piles.
- If the total ultimate capacity ( $n\Sigma P_{ult}$ ) of the flood side piles is less than 50% of  $F_p$ , then the distributed load on each pile of the flood side row is set equal to  $P_{ult}$  and the remaining amount of  $F_p$  is distributed onto the remaining piles according to the relative group reduction factors (Rf). Rf values are determined in accordance with Step 4.5 above.

The distribution of load to the piles has a degree of uncertainty. To assure that the piles are not structurally overstressed from combined axial and bending stresses, as well as shear stress, the Group analysis shall also be performed with 100%  $f_{ub}L_p$  applied to the lead pile, but no more than  $\Sigma P_{ult}$  as described previously. Pile allowables shall be increased by a 50% overstress factor. The shear strength in the soil shall also be checked, the allowable shear capacity of the soil shall be the ultimate divided by a FOS = 2.0 (see Step 3.2; in Fig 3.5 the allowable load is 12.2 kips).

5.4. The Group analysis will yield the response of the piles to all the loads applied to the T-wall system. The Group 7 program will automatically generate the p-y curves for each soil layer in the foundation based on the strength and the soil type. Once the Group 7 run is completed, the pile shear and axial force responses are determined from the output file. These forces must be determined from the piles local coordinate system. The pile group reduction factors shown previously in Step 4.4 are the same as used by the Group 7 program, so the program can be left to compute them automatically.

5.5. This analysis can be made using partial p-y springs to support the piles in the volume of the critical failure mass similar to reductions for the CPGA method found in step 4.2. The partial p-y curves are interpolated on the basis of the unreinforced factor of safety determined in Step 2. If the unreinforced safety factor is less than or equal to 1 then the p-y curves inside the failure circle are zeroed out so that the soil in the failure mass offers no resistance to pile movement. If the unreinforced factor of safety is between 1 and 1.5 the target factor of safety the p-y springs are partially activated based on the percentage that the unreinforced safety factor is between 1 and 1.5 the target factor of safety. Thus, if the unreinforced factor of safety is 1.25 and the target is 1.5, the p-y springs are 50% activated. Fifty percent activation is achieved by reducing the shear strengths in the Group 7 soil layers by 50%.

5.6. Perform structural design checks of the piles and T-wall to ensure that selected components are not overstressed and displacement criteria are met.

Include stress check for the 100%  $f_{ub}L_p$  applied to the lead pile as stipulated in Step 5.3.

5.7. Compare the allowable axial and shear capacity loads from Step 3 to the pile responses. If the axial and shear forces in any pile exceed the allowable pile loads the piles are considered over capacity and the pile design must be reconfigured.

#### GUIDANCE FOR EXTENDED PILE FOUNDATIONS AND SEQUENTIAL PILE FOUNDED STRUCTURES EXPERIENCING UNBALANCED LOADS.

##### **3.4.3.4 Sector Gate and Drainage Structure Foundation Analysis**

Pile foundations for Sector Gate and Drainage Structure monoliths are checked for stability using the same procedure as T-Walls except that limitations are made on the number of piles included in resisting the unbalanced load. The minimum neutral block dimensions described in Step 1.2 are applicable; this includes the full width of the base. The number of piles dedicated to resist the unbalanced load is limited to the greater of those required to satisfy the flow-through as calculated in Steps 4.5 and 4.6. In Step 4.5, the affected piles are limited to that number needed for  $P$  allowable to exceed  $F_p$  applied. In Step 4.6, the number of affected piles is limited by those bound by the Shear area (see Fig 3.6).

##### **3.4.3.5 Fronting T-Walls with Trailing Structures**

Until further analysis proves otherwise, the unbalanced load shall be conservatively resisted by only the fronting wall. Therefore, global stability will be based on the fronting wall. The neutral wedge minimum, specified in Step 1.3 as the greater of  $0.7 H$  or the base width, shall be based on the fronting wall only. It is assumed that a failure plane would penetrate the trailing structure regardless of the structure net downward force and base shear strength capacity. The procedure for T-Walls would be fully applied to the fronting wall w/o considering the trailing structure. The benefit to this approach is that the fronting wall stabilizes the soil under the trailing structure so there is no loss in pile capacity above a critical failure plane. This is significant when considering that many of the existing trailing structures are built on timber piles with minimal capacity. Note that the protected side tail water, where applicable such as the intake basin of a pump station, imposes a dead load. This dead load is relieved by the pile foundation and is not included in the Central Block Resistance for cohesive soils ( $R_b$  in MOP analysis). However, the tail water head creates a downward pressure that should be included in passive driving resistance ( $D_p$  in MOP analysis). One solution to reduce any unbalanced load with sequential structures is to locate the fronting wall further from the pump station such that a stability berm can be built between the two.

### **3.4.3.6 Design Examples**

Examples of this step-by-step design procedure for T-Walls are provided in Appendix E.

### **3.5 Levee Tie-ins and Overtopping Scour Protection**

For a structural alternative on utility crossings, see Structures Section for Details. The tie-in details for T-Walls and L-Walls that terminate into a levee section must follow the latest guidance. See Structures Section for Details.

Scour protection on the flood side and protected side of wall should follow the latest guidance presented in the Structures and Hydraulics Sections.

### **3.6 Utility Crossings**

These guidelines have been prepared after detailed review, analysis and practical application of various methods and the performance of crossings subjected to Hurricane Katrina. These guidelines describe the only acceptable methods for pipeline crossings of levees which qualify as part of a **Federal Hurricane Protection Levee System**. The following is a brief description of the acceptable methods for crossing hurricane protection levees.

#### **3.6.1 Directional Drilling**

Directional drilling consists of inserting the pipeline underground well below the hurricane protection system levee. This can be accomplished before, during or after construction of a project. The required depth is a factor of local soil conditions, design elevation and anticipated long-term consolidation and settlement of foundation soils. Pipelines must also be designed to emerge from underground a safe distance from the limits of the project. Currently utility crossings using this method are reviewed individually upon submittal to MVN of a proposed design by the utility owner. General criteria for installing pipelines by nearsurface directional drilling under levees follows.

##### **3.6.1.1 Layout**

The pipeline entry or exit point, when located on the protected side of a levee, should be set back sufficiently from the protected side toe of the levee such that (a) the pipeline reaches its horizontal level (maximum depth), and/or (b) the pipeline contacts the substratum sands or some other significant horizon, at least 300 feet from the protected side of the levee toe.

When the pipeline entry and/or exit point are located on the flood side of protection, the entry and/or exit points should be positioned such that the pipeline

is (a) landward of the projected 50-year bankline migration, (b) at least 20 feet riverward of the levee stability control line based on the applicable project factor of safety, and (c) at least 10 feet landward of the existing revetment. The purpose of this restriction is to avoid placing a potential source of seepage close to the levee stability control line, and also to help assure the pipeline retains adequate cover.

**3.6.1.2 Design Criteria**

The basic relationship for hydraulic fracture pressure ( $P_f$ ) for undrained conditions is a function of the in-situ minimum principal total stress,  $\sigma_3$ , i.e. the sum of the overburden pressure plus the undrained shear strength ( $s_u$ ) at the point of rupture. (Note: This does not include any side forces on the soil column.)

$$[1] \quad P_f = \sigma_3 + s_u$$

Undrained conditions assume no flow of the borehole fluid into the soil formation. For bores in south Louisiana soils employing a bentonite drilling fluid with good wall cake, it is reasonable to assume that undrained conditions exist. The downhole or borehole mud pressure is composed of hydrostatic pressure (position head) and circulation pressure. The minimum factor of safety against hydraulic fracture shall be 3.0. Factor of safety is defined here as the ratio of the existing overburden pressure (hydraulic fracture pressure  $P_f$ ) to the downhole mud pressure ( $P_m$ ).

$$[2] \quad FOS = (\sigma_3 + s_u)/P_m$$

**3.6.1.3 Guidelines for Permit Review**

This list of general criteria is not intended to be all inclusive. Additional design details may be considered on a case-by-case basis. It is recommended that applicants for directional drilling permits and their designers schedule a meeting with the Corps of Engineers in the early stages of planning to discuss how these guidelines apply to their proposed work. Applications for directional drilling permits beneath levees/floodwalls will be evaluated primarily for their affect upon the integrity of the flood protection system.

Directional drilling will not be allowed in congested urban areas. Exceptions may be considered where population density and land use allow adequate room for expeditious replacement of the flood protection should hydraulic fracture or other damage occur.

Applications for directional drilling permits shall furnish engineering evaluations and computations addressing all the issues presented here and provide specific measures of problem avoidance, dimensions, distances, pressures, weights, and all other pertinent data regarding drilling operations.

Applications for directional drilling permits shall address the ratio of drill diameter versus installed pipe diameter and how seepage through the annular space will be avoided. The applicant should not over-ream the final drill hole, as seepage will potentially result.

Applications for directional drilling permits shall include details demonstrating that the drilling operation will not create a hydraulic fracture of the foundation soil beneath and near the levee. Designers shall provide calculations confirming that the downhole mud pressure during the drilling operation results in a minimum factor of safety equal to 3.0 against hydraulic fracture of the levee foundation within 300-ft of the levee toe. These calculations shall bear the stamp of a registered civil engineer.

Applications shall include a plan for mitigating the potential problem of hydrolock in the borehole due to unanticipated clogging of the return fluid, and the potential loss of drilling fluid return to the surface as a result of other unforeseen downhole problems.

#### **3.6.1.4 Drilling Operations**

The pilot hole cutter head must not be advanced beyond/ahead of the wash pipe more than a distance such that return flow would be lost. Also, the wash pipe ID should be sufficiently greater than the OD (cutting diameter) of the pilot cutter head such that return flow is enhanced. Applications for directional drilling permits shall directly address the methodology to be employed in the effort to keep the return of flow up the drill hole during the entire operation. These requirements are to assure that blockage of the annular space between the wash pipe and drill pipe and associated pressure build-up do not occur.

Drilling mud shall be of sufficient noncolloidal lubricating admixtures to (a) assure complete suspension and removal of sands and other "solids" cuttings/materials, and (b) provide adequate lubrication to minimize bridging by cohesive materials thereby facilitating surface returns flow along the annular space.

The fly cutter used in the prereamer run shall have an OD (cutting diameter) sufficiently greater than the OD of the production pipe such that the hole diameter remains adequate to minimize hang-ups of the production run and thereby, associated stresses on surrounding soils. Applications for directional drilling permits shall also address the increased seepage potential caused by this annular space developed during drilling.

Prereamer runs shall be a continuous operation at least through the down-slope and up-slope cutting sections to prevent undue stress on the surrounding soils during re-start operations.

Shut-off capability in the production pipeline should be provided to immediately cutoff flow through the pipeline should leakage occur.

Positive seepage cutoff or control and impacts of future levee settlements on the pipeline must be addressed and supported approved engineering analyses.

### **3.6.1.5 Construction Schedule**

All work on, around and under levees or flood protection is season sensitive. Some levee/flood wall systems serve as hurricane protection, some are for river flooding and still others are for a combination of these. There may be a season during which the sensitivity of the flood control system will not allow work. Designers should make every effort to discern the alternate methods of providing interim flood protection which may be required during each phase of work.

### **3.6.1.6 Monitoring and Liability for Damages**

Work shall be monitored by Corps representatives. The applicant will reimburse the Corps for all costs, including salaries and per diem, associated with monitoring the entire project. Applicants shall inform the MVN Operations Division permits representative 36 hours in advance of beginning of installation. Drilling beneath levees shall begin during the daylight hours Monday through Friday to facilitate monitoring. The applicant must estimate his work schedule and inform the Corps so that representatives may have adequate time to study the site.

The owner/applicant shall be liable for any damage to the levee resulting from drilling operations. Damage is defined as drilling fluid returns to the surface inside the levee cross-section. The owner/applicant shall replace and/or repair the damaged levee to the Corps of Engineers' satisfaction. Repair may include total replacement of the levee and installation of a grout curtain to the depth of the pipe. Repairs shall be performed in an expedited fashion to Corps specifications.

Applications for directional drilling permits shall include a plan to replace the flood protection should damage occur. A typical sketch of this repair is shown for information only as Figure 3.10.



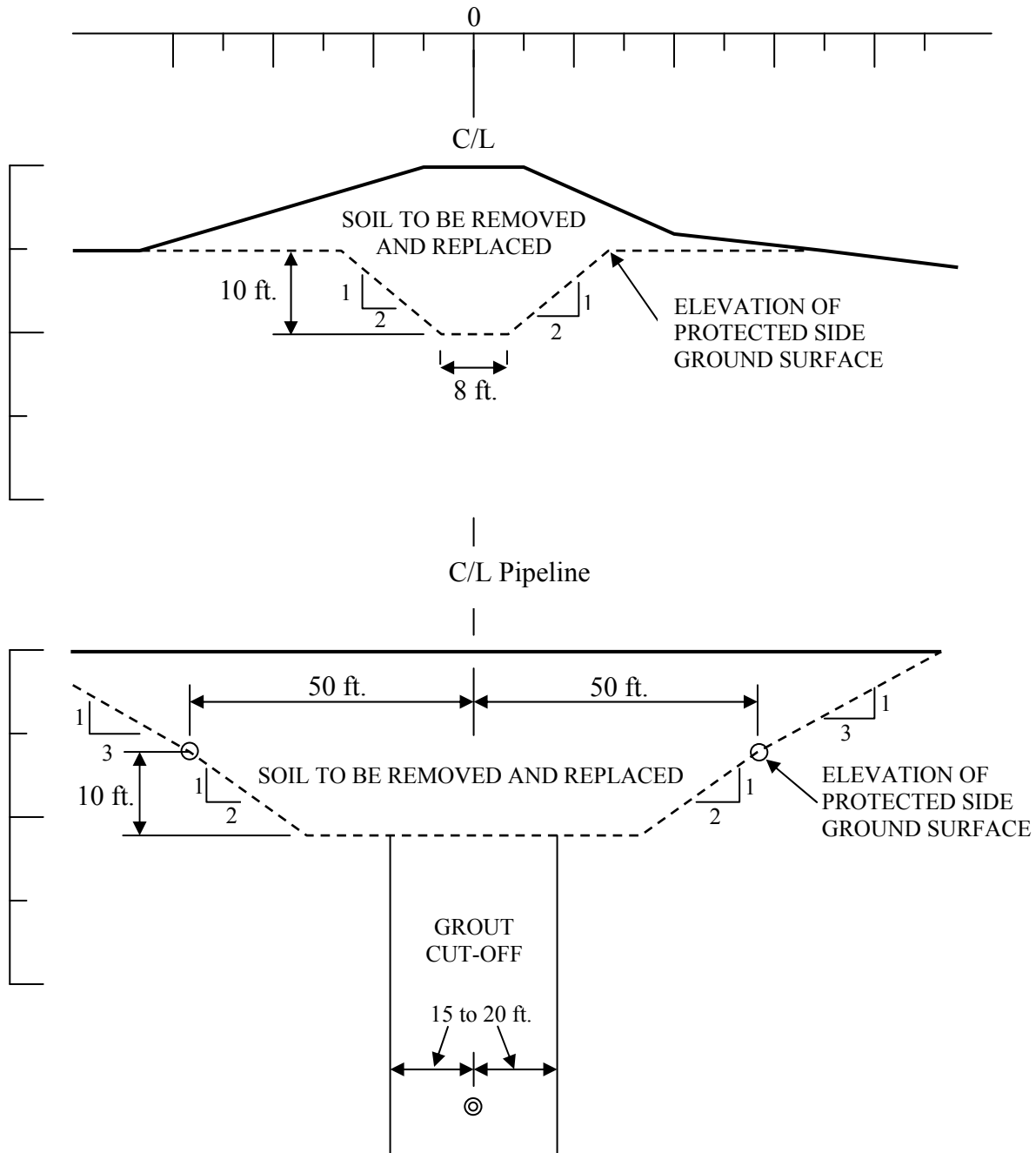


Figure 3.10 Sample detail of repair of directional drilling damage to levee

### **3.6.2 Structural Elevated Support**

This method consists of a structure supporting the pipeline using pile bents and framing that elevates the pipeline a minimum of 15 feet above the authorized design grade and section. This method must be engineered for structural integrity, capacity and clearance for site-specific conditions. Some limitations are listed below:

- The low chord of the pipeline truss must be a minimum of 15 feet above the design section.
- If the truss carries power, the minimum above the design section increases to 18 feet for voltages up to 0.75Kv.
- Piles must be at least 10 feet from theoretical levee toe.

### **3.6.3 T-Wall Construction**

This method focuses on passing the pipeline through T-wall construction with the existing pipeline remaining in place. This method consists of constructing a pile-founded, inverted T-wall flanked by a sheet-pile wall on either side to provide seepage reduction measures for flood protection. The T-wall is built around the in-situ pipeline.

This will require that the pipeline be supported on pile bents for a distance on either side the T-wall to be determined by the pipeline owner. The pipeline can penetrate either the T-wall or its attendant cutoff wall depending on specific site conditions and pipeline geometry, but the T-wall is not allowed to support the pipeline. Again, existing site conditions must be taken into account when using this alternative.

### **3.6.4 Direct Contact Method**

(1) The pipeline owner has the option of placing the pipeline in direct contact with the surface of the newly constructed hurricane levee. This will require the owner to relocate the pipeline when the levee is raised because of settlement or change in design grade. The owners must also determine that the pipeline can sustain the settlement and resulting stresses that are associated with it. Slope pavement or other approved methods must be installed over pipeline throughout transition area.

(2) A modification to the direct contact method is to place pile supports under the pipeline to mitigate the settlement problem. The supported pipe maintains its position as the levee settles beneath it without requiring removal and replacement as additional levee lifts are placed beneath the elevated pipeline. Erosion protection is required beneath the pipeline and around the support piles. Erosion protection will need to be removed and replaced after each levee lift. Since the pile supports are placed in the levee seepage reduction measure is required in the

form of a sheet pile. After the final levee lift is conducted and completed the pile supports are removed by cutting them off below the levee surface and the pipeline is placed in direct contact with the levee and protected with earth cover and erosion protection. Some limitations are listed below:

- Supports are allowed into the levee cross section provided a sheetpile is constructed within the levee section. The vertical supports shall not be located within 15 feet of the levee centerline. The sheetpile must not only provide seepage reduction but also be stable in the event up to 6 feet of scour or erosion could take place. Sheetpile must extend at least 30 feet on either side of pipeline
- Settlement of pile bents within levee section must be addressed.
- Slope pavement over crown and on both protected side and flood side slopes with adequate joints to handle differential settlement must be installed above pipeline and to a distance at least 10 feet past sheetpile. It is suggested that any pile be isolated from slope pavement. Settlement expectation shall be considered while designing scour protection to ensure that sheetpile or pipeline is embedded sufficiently to avoid contact with slope pavement.
- Access along the levees is required on the levee crown and/or by a road on the landside along the berm or at the levee toe. Pipeline crossings must be so designed to insure continuous access during its construction and adequate cover to provide for access over the completed crossing. The cover must be designed for HS20-44 loading over the line for the life of the crossing. (The HS20-44 loading is for tractor trailers and semi-trailers (including dump trucks) of variable axle spacing. This loading covers a gross truck weight of 20 tons and a rear axle weight of 16 tons).
- Stability analysis and settlement analysis will/may be required for pipeline crossings in some instances, particularly those involving the addition of a substantial amount of fill including road surfacing or the levee section and for levees that require future levee enlargements. The pipeline owner will need to contact the Corps for the slope stability Factor of Safety and load cases.

Other methods have been used in the past with unsuccessful results and are therefore not acceptable methods for pipelines crossing hurricane levees in this project area. In particular, the New Orleans District used the encasement method on an experimental basis in a hurricane protection levee on the west bank of Jefferson Parish. The first time a tropical event was experienced, the bentonite washed out, causing a significant seepage problem. In addition, pipelines passing through I-walls are not allowed.

### **3.7 Borrow Specifications**

Material, quality control and construction specifications for levees and embankments is fully detailed in the New Orleans District's Standard Specification 31 24 00.00 12. Parts of that specification are reproduced here for easy reference by engineers engaged in design work for the HSDRRS.

### **3.7.1 Material**

Embankments shall be constructed of earth materials naturally occurring or Contractor blended. Materials that are classified in accordance with ASTM D 2487 as CL or CH with less than 35% sand content are suitable for use as embankment fill. Materials classified as ML are suitable if blended to produce a material that classifies as CH or CL according to ASTM D 2487.

All fill materials shall be free from masses of organic matter, sticks, branches, roots, and other debris including hazardous and regulated solid wastes. As earth from the designated excavation areas may contain excessive amounts of wood, isolated pieces of wood will not be considered objectionable in the embankment provided their length does not exceed 1 foot, their cross-sectional area is less than 4 square inches, and they are distributed throughout the fill. Not more than 1 percent (by volume) of objectionable material shall be contained in the earth material placed in each cubic yard of the levee section. Pockets and/or zones of wood shall not be placed in the embankment.

Materials placed in the section must be at or above the Plasticity Index of 10. As a precaution, Contractors are required to notify the Contracting Officer whenever the in-place Plasticity Index of the material is 15 or less.

Materials placed in the section must be at or below organic content of 9 percent by weight, as determined by ASTM D 2974, Method C.

### **3.7.2 Quality Control**

A. Soil Classification Tests. Determination of soil classification shall be in accordance with ASTM D 2487. Atterberg Limits Test required for soil classification shall be performed in accordance with ASTM D 4318. One Atterberg test shall be obtained from the sample material used for each control compaction curve and one shall be obtained from the sample material used for each in-place density test. If the Nuclear Method is used, the material to be tested shall come from within a radius of (12 inches) of the center of the in-place density test site. The soil classification obtained from in-place density tests will serve as the basis for determining the applicable control compaction curves.

B. Control Compaction Curves - Compacted Fills. Control compaction curves shall be established in accordance with ASTM D 698 (Standard Proctor Density Tests). Two control compaction curves will be required for each type of material from each source. Where construction operations result in blending of several types of material prior to or during fill placement within the embankment design sections, two control compaction curves will be required for each resulting blend of material and will be utilized in lieu of those required for the "unblended materials". The average of the two tests shall be the controlling optimum

moisture content and maximum density. To ensure contract compliance, the Contractor shall submit the compaction curves to the Contracting Officer's representative so they can be faxed to Chief of Geotechnical Branch for review.

C. In-Place Density Tests. In-place density tests for compacted fill material shall be made in accordance with ASTM D 2922 or ASTM D 1556. The location of the test shall be representative of the area being tested or as directed by the Contracting Officer. For each in-place density test, the Contractor shall determine the percent of ASTM D 698 maximum dry density and the percent (plus or minus) optimum water content using the control compaction curves for the same type of material. To ensure contract compliance, the Contractor shall submit the results of the in-place density test to the Contracting Officer's representative so they can be faxed to Chief of Geotechnical Branch for review.

D. Moisture Content Tests. Moisture content tests at each density test location. Determination of moisture content shall be performed in accordance with ASTM D 2216 or ASTM D 3017. For moisture control in the field, microwave testing in accordance with ASTM D 4643 is allowed in the Contractor's field laboratory.

E. Organic Content Tests. Organic content tests at each density test location. Determination of organic content shall be performed in accordance with ASTM D 2974, Method C.

F. Government Testing. As a control, the Government will perform assurance and check tests for maximum density for all materials in accordance with ASTM D 698. If values for maximum density as determined by the Contractor and as determined by the Government do not agree, the Government will determine the values to be used. The Government will also perform check and assurance testing of the other control testing.

### **3.7.3 Construction**

Compacted fill shall not be placed in water. The materials for compacted fill shall be placed or spread in layers, the first or bottom layer and the last two layers not more than 6 inches in thickness and all layers between the first and the last two layers not more than 12 inches in thickness prior to compaction except the first layer on top of a geotextile shall be 15 inches, plus or minus 3 inches tolerance, as specified in Section 31 05 19.03 12 GEOTEXTILE SEPARATOR UNDER LEVEE CROWNS, ROADS, OR RAMPS.

Layers shall be started full out to the slope stakes and shall be carried substantially horizontal and parallel to the levee centerline with sufficient crown or slope to provide satisfactory drainage during construction. Areas on which geotextile is to be placed shall be dressed out and leveled to the grade indicated on the drawings. When placing fill on the geotextile, mechanical equipment shall not be allowed to come in contact with the geotextile in any way.