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INTRODUCTION

Hurricanes Katrina and Rita caused tremendous loss of life and destruction of property when they struck coastal Louisiana in 2005. The US Army Corps of Engineers and the New Orleans District continue to investigate the shortcomings of the hurricane and storm damage reduction system. Engineers are working to learn what happened and to make appropriate and effective changes and improvements in the planning, design, construction, operation and maintenance of hurricane protections to prevent future disasters to the greatest extent possible.

Several efforts to restore, repair and improve the hurricane and storm damage reduction system in coastal Louisiana have been completed or are currently underway. The Chief of Engineering Division, New Orleans District, directed the preparation of this compilation of design guidelines to provide a comprehensive collection of best practices for those engaged in these projects.

This guide is presented in two parts. The first part, "Design Guidelines," presents methods and criteria that shall be used by engineers in the design of hurricane system components. The design methods and criteria presented in this report should not be considered final. As new information is continuously discovered and design techniques always evolve, updates will be issued. Engineers are encouraged to consult with appropriate subject matter experts for updates and improvements to the procedures and criteria presented herein.

The second part of this guide is a compilation of "Standards" used by the New Orleans District. This includes requirements for surveys and typical details for common construction elements. While exceptions and variations for specific projects are likely to arise, engineers working on projects for the District should follow the standards as presented as much as possible.

A list of acronyms and links to referenced and other common publications is provided to assist engineers in their work.

Questions, corrections or suggestions should be submitted in writing for review and action. The Engineering Division Point of Contact is Timothy M. Ruppert, P.E. at Timothy.M.Ruppert@usace.army.mil.

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 Denis J. Beer, P.E. at Denis.J.Beer@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.

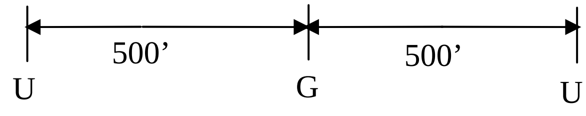


Figure 3.1 Boring spacing

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 below. 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. (Note that risk-based approaches are currently being developed for future incorporation in these criteria.)

Note: see Table 3.2 below, with increased factors of safety for MOP analyses, for interim design criteria for earthen embankments until a software program using Spencer procedure has been fully tested and can efficiently model southeast Louisiana’s unique foundation conditions that contain varying unit weights and shear strength within the same stratum.

Table 3.1 – Slope Stability Design Factors of Safety.

Analysis Condition	Required Minimum Factor of Safety	
	Spencer Method[1]	MOP[2]
End of Construction[3]	1.3	1.3
Design Hurricane[4] (SWL)	1.5	1.3
Extreme Hurricane (top of levee)	1.4[5] (1.5)[6]	1.2
Extreme Hurricane (top of wall)	1.4[5] (1.5)[6]	1.3
Low Water (hurricane condition)[7]	1.4	1.3
Low Water(non-hurricane condition)[8] S-case	1.4	1.3
Design Hurricane Utility Crossing[9]	1.5	1.5 (1.3)
Extreme Hurricane Utility Crossing[10]	1.5 (1.4)	1.4 (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. LMVD Method of Planes shall be used as a design check for verification that the HPS design satisfies historic district requirements. Analysis shall include a full search for the critical failure surface since it may vary from that found following the Spencer method.
3. Applies to flood side and protected side. 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. Normal water level conditions would be used and strength gain with time is conservatively ignored. (For limited cases over soft foundations (i.e., new levees), strength gains during construction can be considered but will require a detailed design study).
4. Applies to protected side for the SWL condition (100-yr return period is authorized as the current design hurricane loading condition). 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.
5. Applies to protected side for an extreme load condition with water to the top of barrier under a short term hurricane condition. 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.

(Note: The MOP factor of safety agrees with 20 Apr 06 design criteria for I-walls.)

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

7. Applies to flood side where low hurricane flood side water levels provide a destabilizing force. 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 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. This analysis represents a long-term water level drawdown where steady state conditions prevail. Stability is analyzed using drained strengths expressed in terms of effective stresses. (S-case type analysis for normal loading condition; non-hurricane loading.)

9. Applies to floodside and protected sides. Design Hurricane water elevation is SWL. 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 may be used for levees that have received their final levee lift.

10. Applies to floodside and protected sides. Extreme Hurricane water elevation is to the top of the levee. 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 may be used for levees that have received their final levee lift.

3.1.2.3 Interim Slope Stability Design Criteria

Given the lack of a software program which will adequately analyze slope stability factors of safety (FOS) utilizing Spencer's Method (varying both shear strength and unit weights along the levee cross section), the following criteria shall be utilized for design until such a program has been approved by the government. At that time designs will be checked to verify that the criteria stated in Table 3.1 are satisfied. Utilizing the interim design criteria for earthen embankments shown in Table 3.2 should ensure that the appropriate Spencer's Method FOS will be obtained.

Table 3.2 – Interim slope stability factors of safety.

Stability Analysis Method	**Conditions**	**Protected Side**		**Flood Side Low Water Condition** [1]
		Still Water Level (SWL)	**Water at Top of Levee**	
Method of Planes	Berm designed for a FOS= [3]	1.40	1.30	1.35
	Berm designed for a FOS= [4]	1.35	1.25	1.30
Limited Spencer's Analysis [2]	Equal Unit Weights (Centerline vs. Toe)	1.50	1.40	1.40
	Different Unit Weights (Centerline vs. Toe)	1.55	1.45	1.45

NOTES:

1. The S-Case shall also be analyzed for normal water conditions toward both the protected side and flood side.
2. Limited Spencer Analysis: The UTexas4 program may be utilized to perform a Limited Spencer's Analysis to verify the required levee sections when limited rights-of-way are available. Since the UTexas4 program presently cannot vary unit weights along a cross-section, the required factors of safety will be a function of whether the actual unit weights (centerline vs. levee toe) are the same or vary due to those actual conditions.
3. Utilizing the higher Method of Planes FOS for interim design procedures should ensure that the appropriate Spencer FOS will be obtained once the levee section is analyzed with a software program that can perform Spencer Analysis and can efficiently model MVN unique foundation conditions that contain varying unit weights and shear strength within the same stratum.
4. If the less conservative interim FOS criteria of 1.35 by MOP is applied (still higher than final criteria by the MOP) to avoid over shooting the final Spencer-based criteria, a limited Spencer's analysis should be performed to meet the FOS in Table 3.2 above.

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.

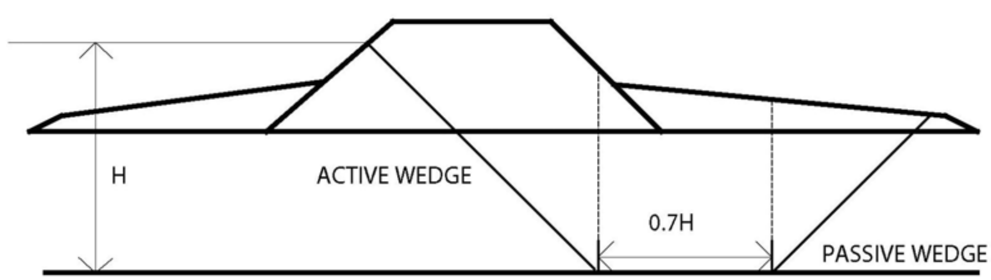


Figure 3.2 Minimum distance between active and passive wedges (embankments)

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

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

Note. Weight of riprap may vary based on the filling of the riprap voids over time.

For most designs, the central portion of the levee and flood side stability/wave berm consists of compacted clay, and the protected side stability berms consist of uncompacted clay. For berms that will support a substantial amount of rock for erosion protection or roadways, use compacted clay material.

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 1.50 with a flooded pit, and 1.30 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

It is the intent of these criteria to provide requirements that result in a safe design for seepage and uplift based on loading to the top of the barrier at any stage in the life of the project. In support of that, the following criteria are based on steady state seepage conditions in coarse grained soils. Due to their permeability it is unlikely that steady state conditions will develop in fine grained soils within the relatively short duration of a hurricane storm surge. However, open seepage entrances and non-continuity in blanket materials may allow steady state conditions to occur in coarser strata.

The following criteria are based on ETL 1110-2-569 except that factors of safety are presented instead of seepage gradients. Factors of safety are used because of the lighter weight blanket materials that may be encountered in the local region. If the criteria presented in the following table are not met, at the levee toe, seepage berms or remediation measures shall be designed in accordance with EM 1110-2-1901, DIVR 1110-1-400 (for material properties where site specific information is not available), and ETL 1110-2-569 (with additional criteria requiring specific factors of safety at the seepage berm toe). Hurricane and storm damage reduction system seepage berms, relief wells or other seepage control measures shall be designed to meet the minimum factors of safety illustrated in Table 3.5. The factors of safety for seepage are computed using effective stresses (defined by gradient) as:

$$FS_g = \frac{\gamma' \times z_t}{\gamma_w \times h_o} \quad \text{same as} \quad FS_g = \frac{I_{cr}}{I_e}$$

γ' = effective unit weight of soil (or average effective unit weight of soil)
γ_w = unit weight of water
z_t = landside (protected side) blanket thickness
h_o = excess head (above hydrostatic) at toe
I_{cr} = critical exit gradient
I_e = exit gradient

Table 3.5 – Seepage and Uplift Design Criteria

Levee/Wall Application	Required Minimum Factor of Safety at Levee or Wall Toe [1]			
	Design Water Surface Elevation (DWSE) [2]		Project Grade [3]	
	Levee Toe	Berm Toe	Levee Toe	Berm Toe
Riverine	1.6	1.15	1.3	1.0
Coastal (Top of Protection < 5 ft above DWSE)	1.6	1.15	1.3	1.0
Coastal (Top of Protection > 5 ft above DWSE)	1.6	1.15	1.2	1.0

NOTES:

1. Minimum factors of safety at the levee toe are based on steady state seepage conditions. Loading in excess of the "Project Grade" is considered sufficiently short term that steady state conditions do not fully develop and safety is adequately addressed by the steady state factors of safety.
2. Design water surface elevation (DWSE) represents the stage or water level used in deterministic analyses such as the geotechnical and structural stability analyses and seepage analysis. For the MVN 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
3. The project grade, sometimes referred to as top of protection or net levee grade, includes increases above the design water surface elevation to account for runup and/or grade elevations for other reasons minus overbuild for primary consolidation.

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 Denis J. Beer, P.E. at Denis.J.Beer@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 (previously known as the Lower Mississippi Valley Division Method of Planes) shall be used for slope stability analysis. Note that equivalent factors of safety (FOS) associated with other slope stability methods have not been determined. The system shall be designed for global stability utilizing the "Q" shear strengths for the following load cases:

Table 3.6 - Global Stability Criteria

Tension Crack Depth	FOS_{min} 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 γ_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.

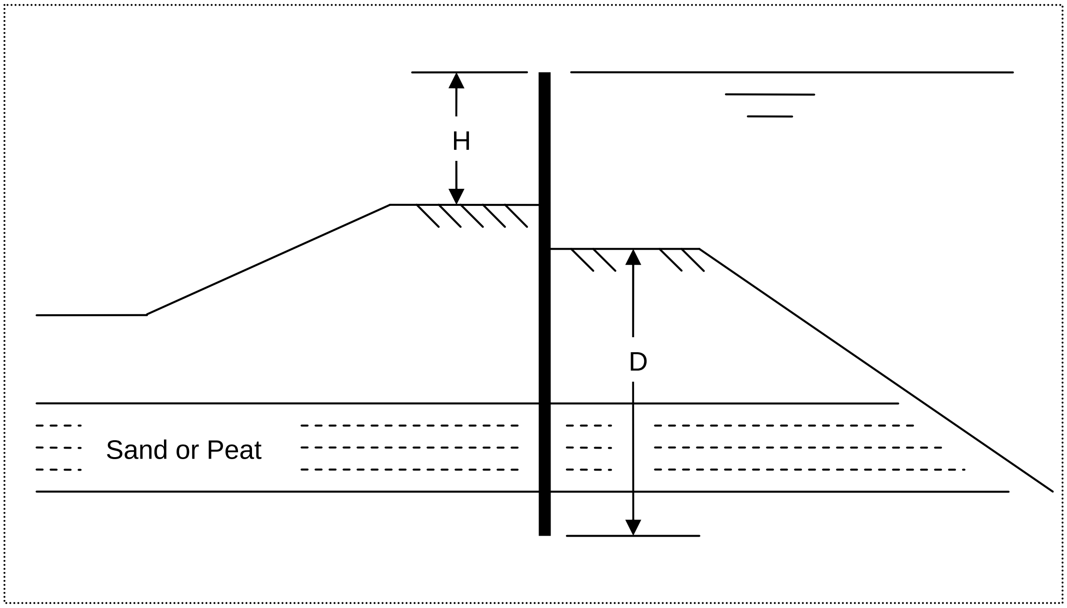


Figure 3.3 Minimum tip penetration depth

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 ft 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.

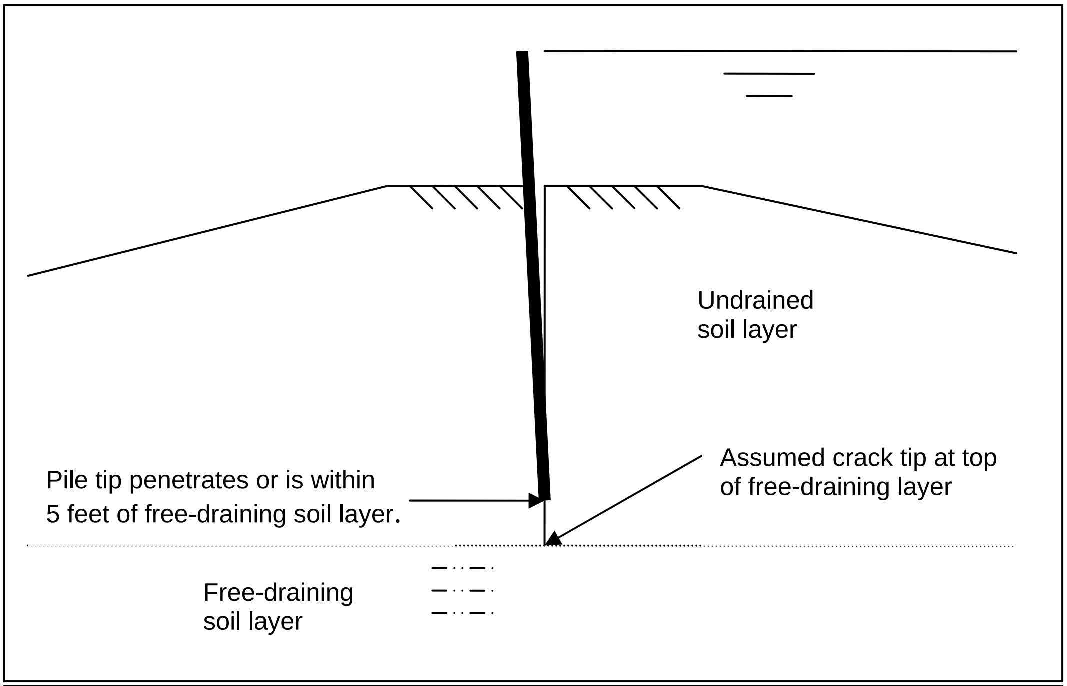


Figure 3.4 Computed crack depth near an aquifer

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:

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

γ_{sat} = saturated unit wt. soil
γ_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 = 30º and SP= 33º 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

	With Pile Load Test	W/O Pile Load Test
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

Q-Case					
Type	phi	Kc	Kt	Nc	Nq
Clay	0	1	0.7	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

S-Case					
Type	**phi**	**Kc**	**Kt**	**Nc**	**Nq**
Clay	23	1	0.7	0	10
Silt	30	1	0.5	0	22.5
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 Denis J. Beer, P.E. at Denis.J.Beer@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

<u>Stability.</u> 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.

<u>Stability Analysis Results:</u>

(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 10 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

<u>Piping (cutoff-wall tip elevation)</u>. 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

This design procedure 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 (using Spencer's method of analysis). This procedure accounts for the reinforcing effect the piles have on the foundation soils and evaluates safe allowable shear and axial forces for the piles. This design procedure is a supplement to EM 1110-2-2906, which shall govern for design aspects not specifically stated here.

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.

As an optional check, the Group 7 model is changed to only include the unbalanced force. The computed axial and shear forces are then used in the slope stability model and global stability is evaluated using those reinforcement loads rather than the unbalanced force. If the computed factor of safety is too low the design is changed.

3.4.3.1 Design Steps

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. 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.

<u>Step 1. Initial Slope Stability Analysis</u>

1.1 Determine the critical failure surface from a slope stability analysis for loading to the SWL and to the top of barrier using a software program capable of performing Spencer's method with a robust search procedure (hereinafter termed Spencer's method). 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 loads acting directly on the structure because these loads are presumed to be carried by the piles to deeper soil layers.

1.2 If the factor of safety of this critical failure surface is greater than required (see Section 3.1.2.2. for slope stability criteria), 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 applied directly to the structure. If the factor of safety of the critical failure surface 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. The lowest elevation of this failure surface is determined for use in the following design steps.

Step 2. Unbalanced Force Computation

2.1 Determine the unbalanced forces (for both loading to SWL and to top of wall) required to achieve the target factor of safety using Spencer's method and either a circular search or non-circular search whichever returns the larger unbalanced force. The unbalanced force should 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 in a non-circular search. The Y-coordinate is 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 from Step 1. The unbalanced force is arrived at through a trial and error process where the force is varied until the desired factor of safety is achieved. The failure surface found in Step 1 is "searched" with the specified line load so that the largest unbalanced force is computed. The unbalanced force, and the defining failure surface coordinates should be noted for use in subsequent steps. The critical failure surface found in this step is used in Step 7.

Comments: The critical failure surface found in this step is not necessarily the critical failure surface once the foundation piles are installed. However, this failure surface conservatively returns a larger unbalanced force for design. However, searching for the failure surface with a line load included sometimes results in erroneous results. In these cases, the failure surface found in Step 1 should be used.

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. An alternative method is to find the allowable axial load capacity through computation using a computer software program such as TZPILE to simulate a pile load test. This procedure is similar to the procedure described in paragraph 3.2 for allowable lateral capacity.

3.2 Compute allowable shear loads in the pile at the critical failure surface. Allowable 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 the Ensoft program LPILE or the Corps program COM624G can be used to compute allowable lateral shear in the pile using the following 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. The pile will fail when deflections increase greatly with minimal increase in load. 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 3.5.

c. Divide the shear load by the same factors of safety used to compute allowable axial capacity from calculated ultimate values.

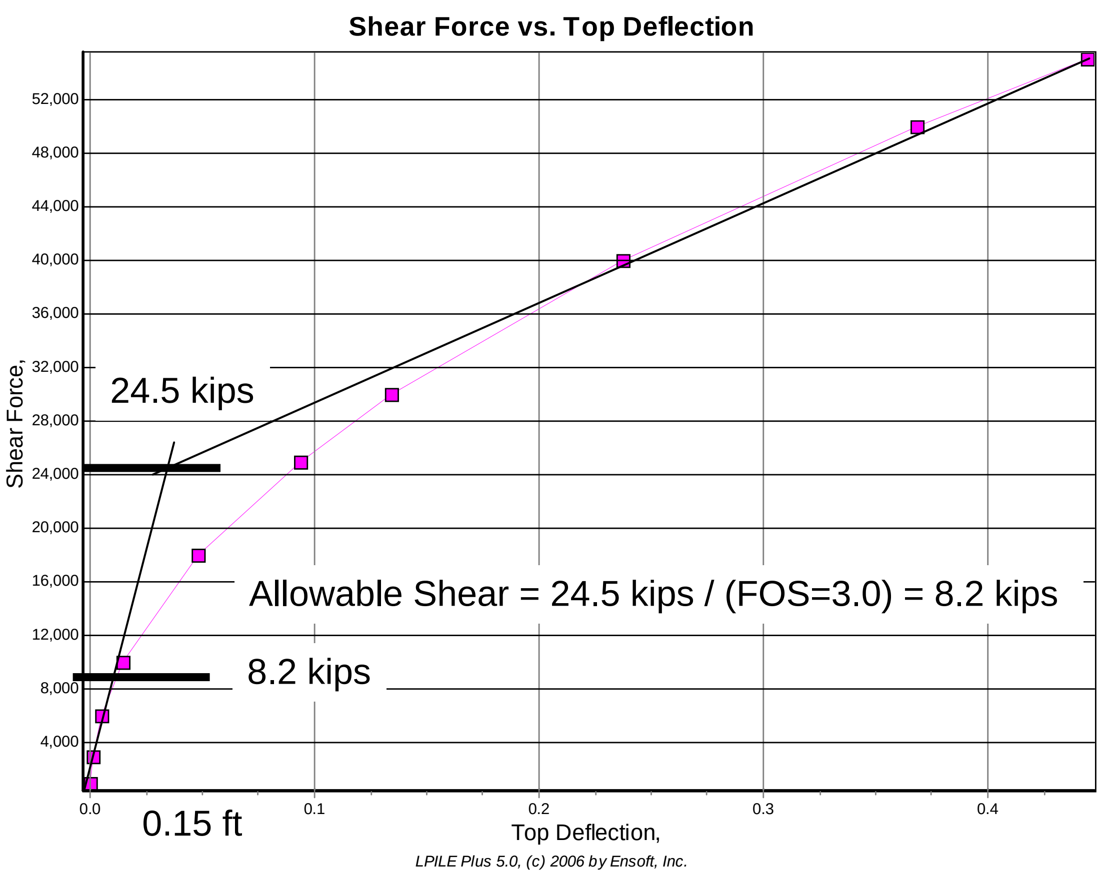


Figure 3.5 Example of computation of ultimate shear load in the pile from a load vs. deflection curve developed using LPILE. FOS varies depending on load case.

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 as being about equal to the stiffness factor, R, below the ground surface. The equivalent force, F_{cap}, is calculated as shown below and in Figure 3.6:

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

Where:
F_{ub} = unbalanced force computed in step 2.
L_u = distance from top of ground to lowest el. of critical failure surface (in)
L_p = distance from bottom of footing to lowest el. of crit. failure surface (in)

$$R = \sqrt[4]{\frac{EI}{Es}} \tag{2}$$

E = Modulus of Elasticity of Pile (lb/in^2)
I = Moment of Inertia of Pile (in^4)
Es = Modulus of Subgrade Reaction (lb/in^2) below critical failure surface. In New Orleans District this equates to the values listed as K_HB.

Comments:

a. The above procedure does not directly account for the unbalanced force that 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 later design steps.

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 to date shows that the presence of the piles influence the actual location of the critical failure surface approximating that shown in the figure. This procedure is considered to provide acceptable design forces in the piles.

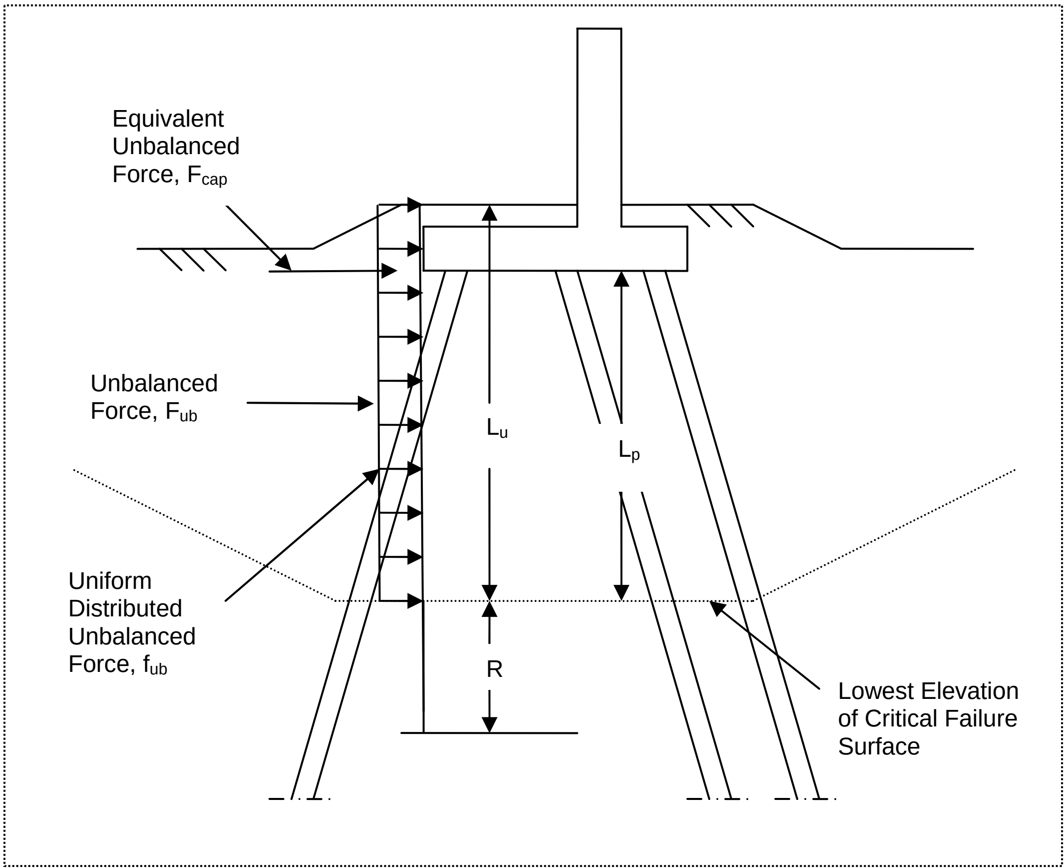


Figure 3.6 Unbalanced Forces.

4.2 In CPGA, the top of soil will be modeled at the ground surface, and the subgrade modulus, Es, is reduced with reduced global stability factors of safety to account for lack of support from the less stable soil mass. For cases where the global factor of safety without piles is less than 1.0, Es is input at an extremely low value, such as 0.00001 ksi (CPGA will not run with Es set at 0.0). For conditions where the factor of safety is between 1.0 and the target factor of safety, Es 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 Es. For example, if the FS = 1.0, Es is input as 0.00001. If the FS = 1.2, the target factor of safety is 1.5, and the estimated value of Es below the foundation is 100 psi, Es 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.

4.3 No reductions to the subgrade modulus are required for cyclic loading. Group reduction factors to be applied to subgrade modulus for the CPGA analysis should be computed as required by EM 1110-2-2906.

4.4 Sheet piling shall be included and designed to control under seepage and is not relied on for stability or to limit soil displacement between piles. Sheet pile shall be designed for seepage in accordance with Sections 3.4.2.4 through 3.4.2.6.

4.5 Storm surge loading on the soil beyond 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. 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:

$$\sum P_{all} = \frac{n \sum P_{ult}}{1.5} \tag{3}$$

Where:

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

ΣP_{ult} = summation of P_{ult} over the height L_p, as defined in paragraph 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

$$P_{ult} = \beta(9S_u b) \tag{4}$$

S_u = soil shear strength

b = pile width

β = group reduction factor pile spacing parallel to the load:

For leading (flood side) piles:

$$\beta = 0.7(s/b)^{0.26} \quad ; or \ = \ 1.0 \ for \ s/b > 4.0 \tag{5}$$

For trailing piles, the reduction factor, β, is:

$$\beta = 0.48(s/b)^{0.38} \quad ; or = 1.0 \ for \ s/b > 7.0 \tag{6}$$

Where:

s = spacing *between* piles parallel to loading

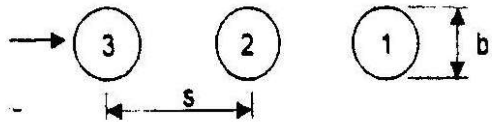


Figure 3.7 Spacing between piles

Note: These group reduction factors are for lateral soil loading on the piles, and may be different than the factors used for the CPGA analysis. 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.

b. Compute the unbalanced load on the piles (F_p) to check against ΣP_{all}:

$$F_p = wf_{ub}L_p \tag{7}$$

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

$$f_{ub} = \frac{F_{ub}}{L_u} \tag{8}$$

Where:
F_{ub} = Total unbalanced force per foot from Step 2
L_u and L_p are as defined in paragraph 4.1

c. If 50% of F_p exceeds ΣP_{all} for the flood side pile row, then compute ΣP_{all} for all of the piles. If ΣP_{all} for all piles is less than $F_{p,}$ then the pile foundation will need to be modified (decreasing pile spacing and/or increasing pile rows) until this condition is met.

4.6 For an additional flow-though 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:

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

Where:

A_pS_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_pS_u. See Figure 3.8.

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

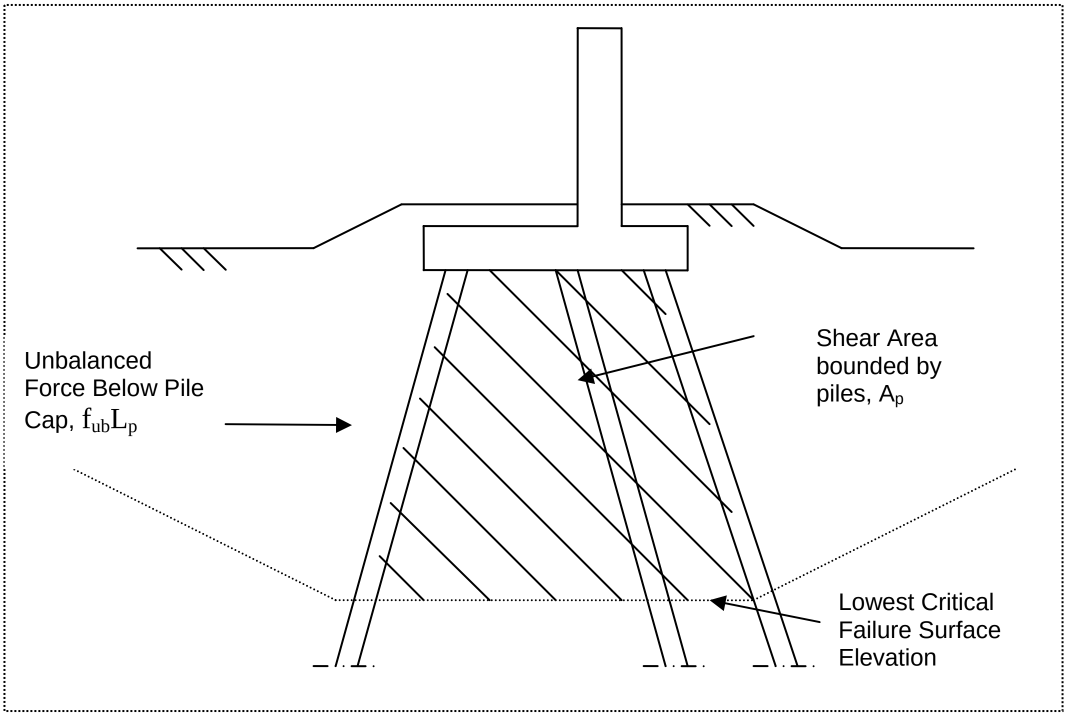


Figure 3.8 Area for soil flow-through shear check.

Note: The sheet pile seepage cut off is conservatively neglected for this computation as its contribution to flow through resistance is not well understood. If this check is not satisfied, the foundation will need to be modified until it is.

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 and the global factor of safety with the piles included. 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 or the top of wall.

5.2 The portion of the unbalanced load above the bottom of the T-wall base is applied as a force and equivalent moment at the pile cap, in addition to the other loads applied directly to the T-wall depending on load case (water pressures, soil weight, concrete weight, vessel impact, etc.).

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 includes the computed unbalanced force as distributed loads acting on the piles. Distribution of unbalanced loading onto the rows of piles is as follows. The total distributed load on the piles (F_p) was defined in paragraph 4.5.

- 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 paragraph 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 (β).

Note: ΣP_{ult} rather than ΣP_{all} is used for the distribution of the unbalanced load to the piles as it is more conservative for the flood side row of piles.

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 paragraph 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. The partial p-y curves are interpolated on the basis of the unreinforced factor of safety determined in Step 1. 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 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 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 is met.

5.7 Compare the allowable axial and shear capacities from Step 3 to the pile responses due to all T-wall loads. 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.

Step 6. Pile Group Analysis (unbalanced force)

6.1 Perform a pile group analysis with Group 7 with the distributed loads applied directly to the piles to replicate the load transfer behavior. This analysis is performed without water loads or other loads applied directly to the T-wall structure since the objective of this step is to determine the extent that the piles resist the unbalanced load. In the analysis, the piles should be treated as free-standing at elevations between the base of the T-wall and the lowest elevation of the critical failure surface from Step 5.

6.2 The response of each pile is determined from the output of the Group 7 analysis by noting the axial and shear forces carried by each pile. The axial forces used in the next step are those from the pile cap and the lateral loads are found from the shear forces where the piles cross the failure surface lowest elevation found in Step 2. These forces must be determined from the piles local coordinate system.

Step 7. Pile Reinforced Slope Stability Analysis

7.1 Run Spencer's method to determine the stability of the foundation due to the reinforcing effects of the piles. The factor of the safety for the critical slip surface from Step 2 (with the water loads only on the ground surface behind the T-wall) will be improved by the pile elements that are represented by the shear and axial forces in the piles (moments are neglected since their contribution to stability is expected to be small). The shear and axial forces found in Step 6 are divided by the pile spacing and imported to Spencer's method as reinforcement forces. This step must be made because Spencer's method analysis is two-dimensional and forces are based on a unit width, whereas Group 7 is also two-dimensional but the forces in the system are based on the force per spacing width. Additionally, close attention must be paid to the sign conventions of both the pile group and slope stability programs. If the computed FOS for this analysis is equal to or greater than the target FOS value the design check is complete and the structure is safe. If the computed FOS is less than the target factor of safety the global stability requirements cannot be met with this pile configuration and the analysis must start over with a new pile design.

3.4.3.2 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, σ_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.)

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

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).

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

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 3.9.

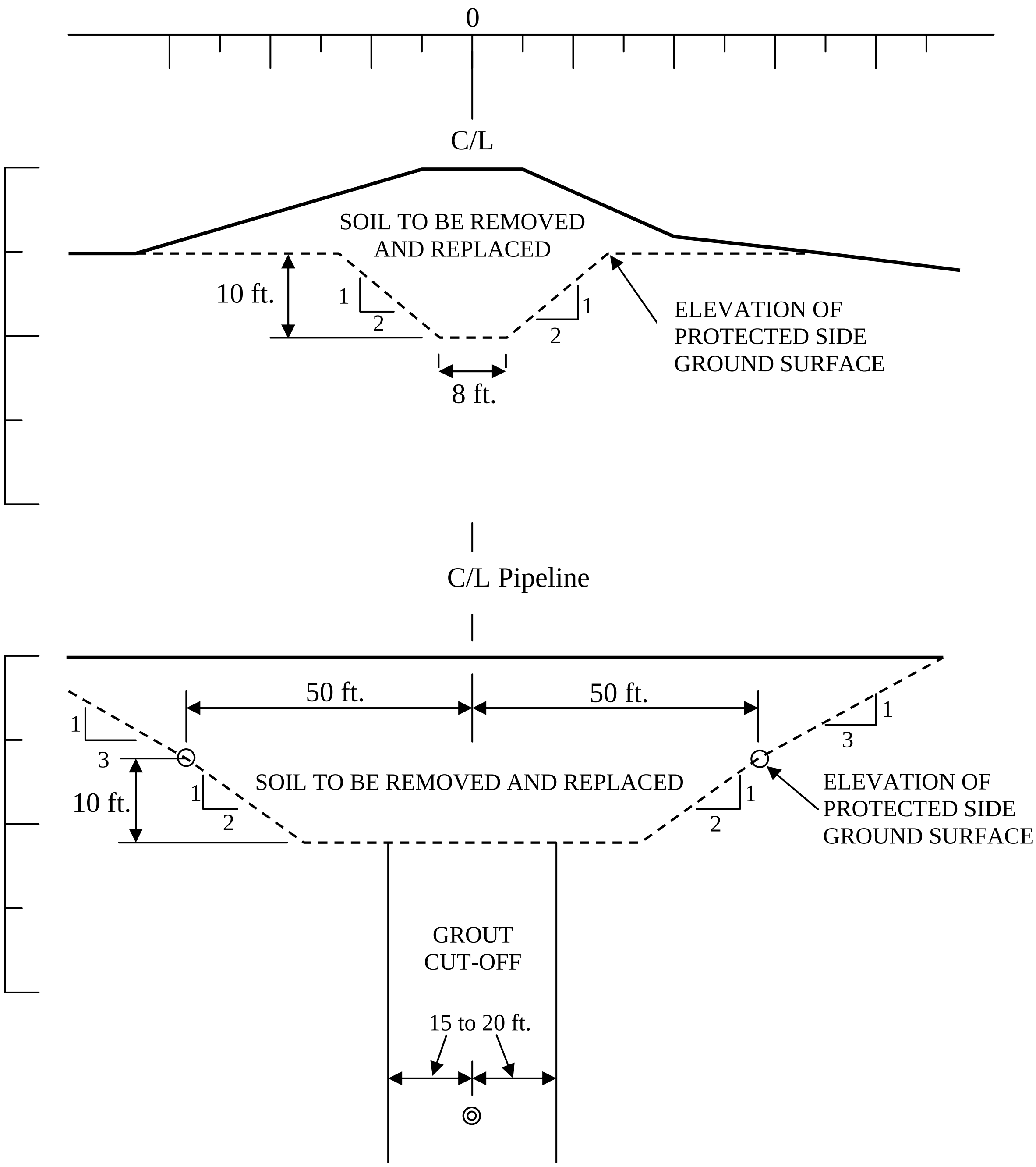


Figure 3.9 Sample detail of repair of directional drilling damage to levee

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 of 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.

E. T-WALL DESIGN EXAMPLES

The following three design examples illustrate the application of the T-Wall Design Procedure outlined in Section 3.4.3 of the Design Guidelines. These examples are provided to help users understand the step-by-step procedure. Nothing presented here shall supersede sound engineering design and judgment.

Design Example #1

A cross section of the wall section used for Example 1 is in Figure 1, based on a wall constructed in New Orleans. The water level used in this example is elevation 10.0. The soil information for this example is shown in Figure 2.

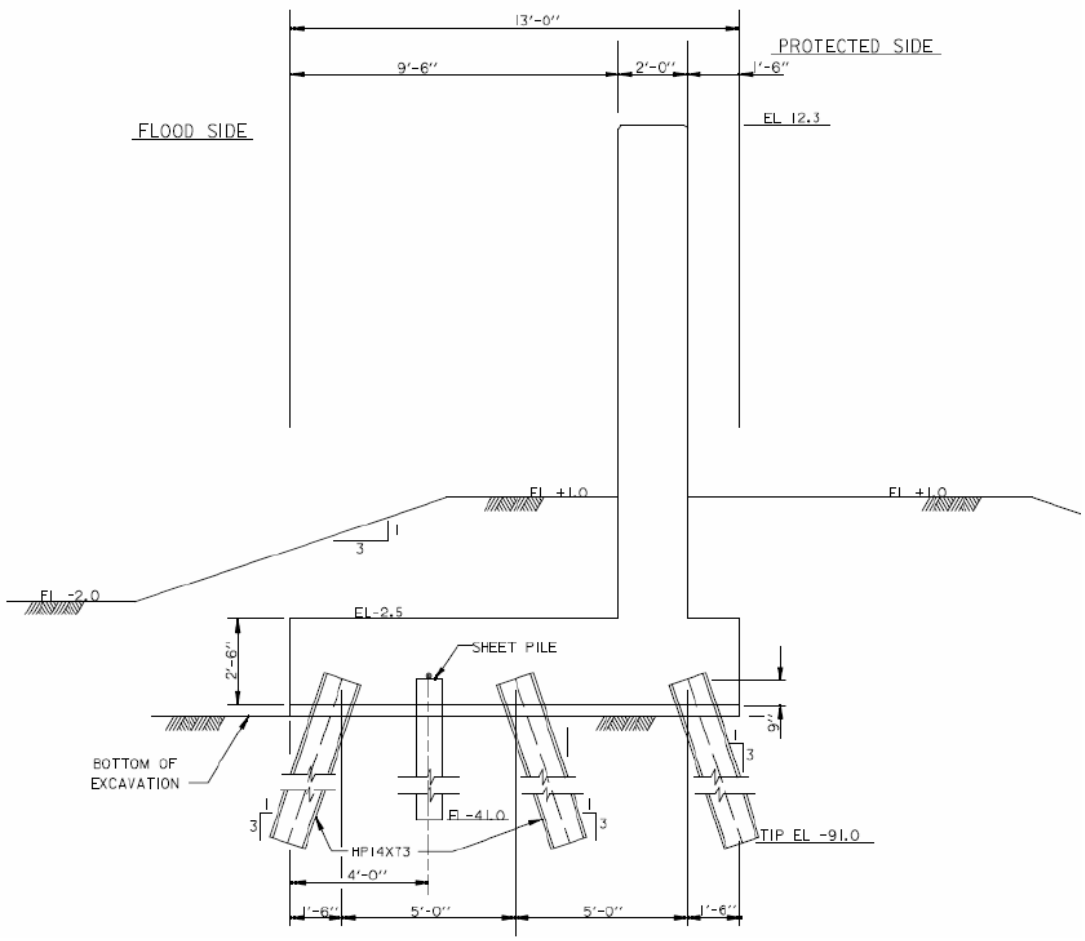


Figure 1. Wall Geometry.

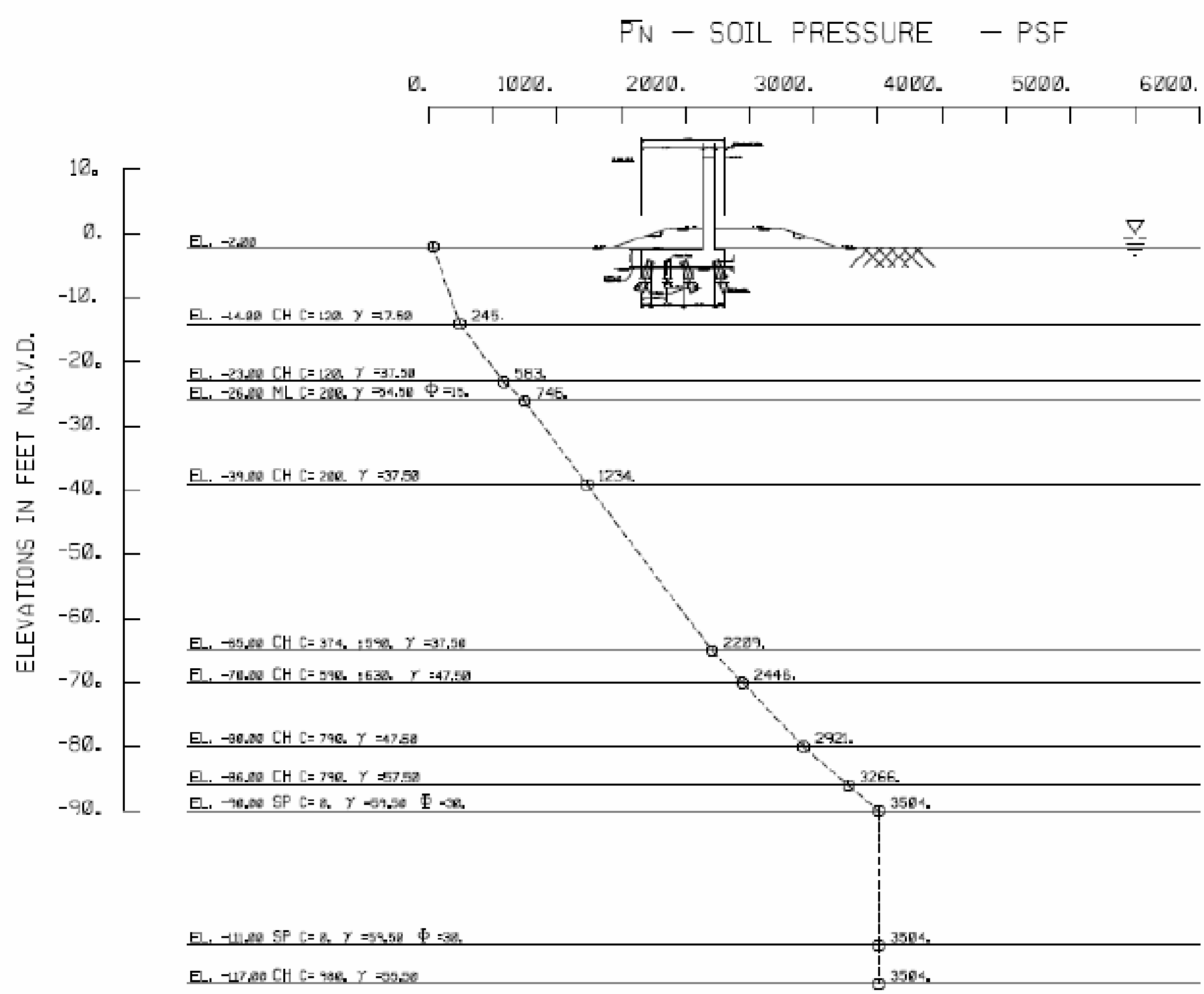


Figure 2. Soil Profile.

Step 1 Initial Slope Stability Analysis

Perform a Spencer's method slope stability analysis to determine the critical slip surface with the water load only on the ground surface and no piles. UTexas4 was used in this example for all of the slope stability analysis. For the design example, the critical failure surface is shown in Figure 3 where the factor of safety is 1.02. Because this value is less than the required value of 1.5, the T-Wall will need to carry an unbalanced load in addition to any loads on the structure.

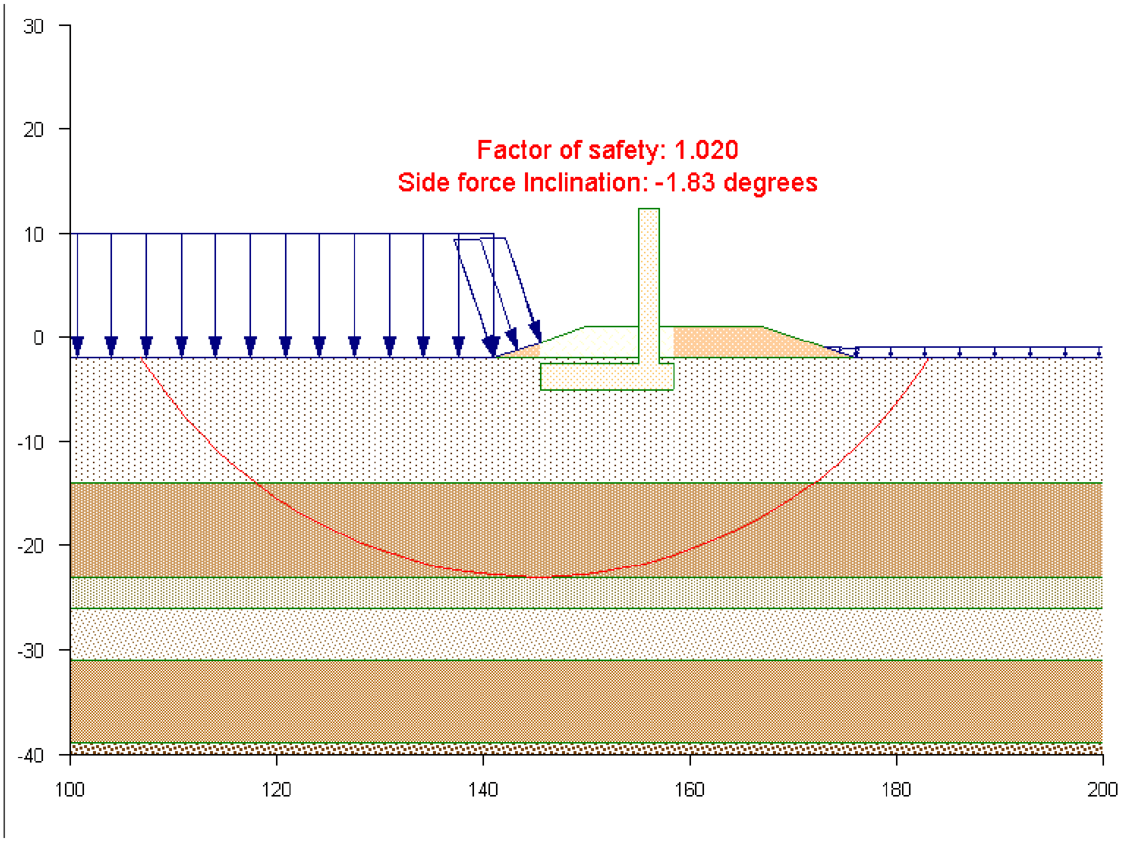


Figure 3. Spencer's analysis of the T-Wall without piles.

Step 2 Unbalanced Force Computations

Determine (unbalanced) forces required to provide the required global stability factor of safety. The critical failure surface extends down to elevation -23' in this example. The top of the soil near the heel is elevation -0.5'. It is assumed that the unbalanced load is halfway between these two elevations. Apply a line load at elevation -11.75, at the x-coordinate of the critical failure surface in Figure 3. After several iterations, a line load of 4,575 lb/ft was found that results in FS = 1.50, as shown in Figure 4.

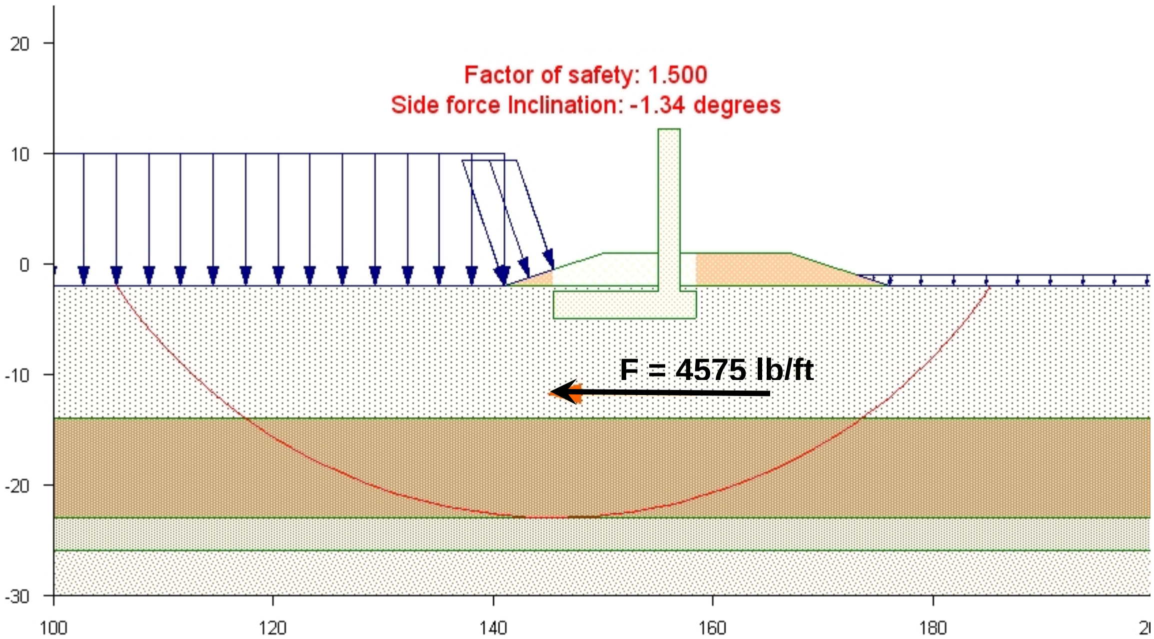


Figure 4. Spencer's analysis of the T-Wall with an unbalanced load to increase global stability.

It should be noted that a search for the critical failure surface was performed with the unbalanced load shown in Figure 4. The search ensures that if the pile foundation of the T-Wall can safely carry the unbalanced load in addition to any other loads on the structure, the global stability will meet the required factor of safety. The UTexas4 input files for Figures 3 and 4 are attached at the end of this example.

Step 3 Allowable Pile Capacity Analysis

3.1 For the preliminary analysis, allowable pile capacities determined by engineers in New Orleans District for the original design of this project are used.

Allowable Compression Load	= 74 kips
Allowable Tensile Load	= 49 kips

See Figure 5 for ultimate loads vs. depth from a compression pile load test. The compression load above was computed using a factor of safety of 2.0 at a depth of 92 feet. For this test, a casing used precludes skin friction above the critical failure surface.

The tension load is taken from calculated values shown in Figure 6. At elevation -92 feet the ultimate load is calculated to be about 81 tons. The capacity above elevation -23 is about 7 tons. Therefore, the tension capacity can be estimated as 81-7 = 74 tons. Using a safety factor of 3 (no load test), the allowable capacity is 74(2)/3) = 49 kips.

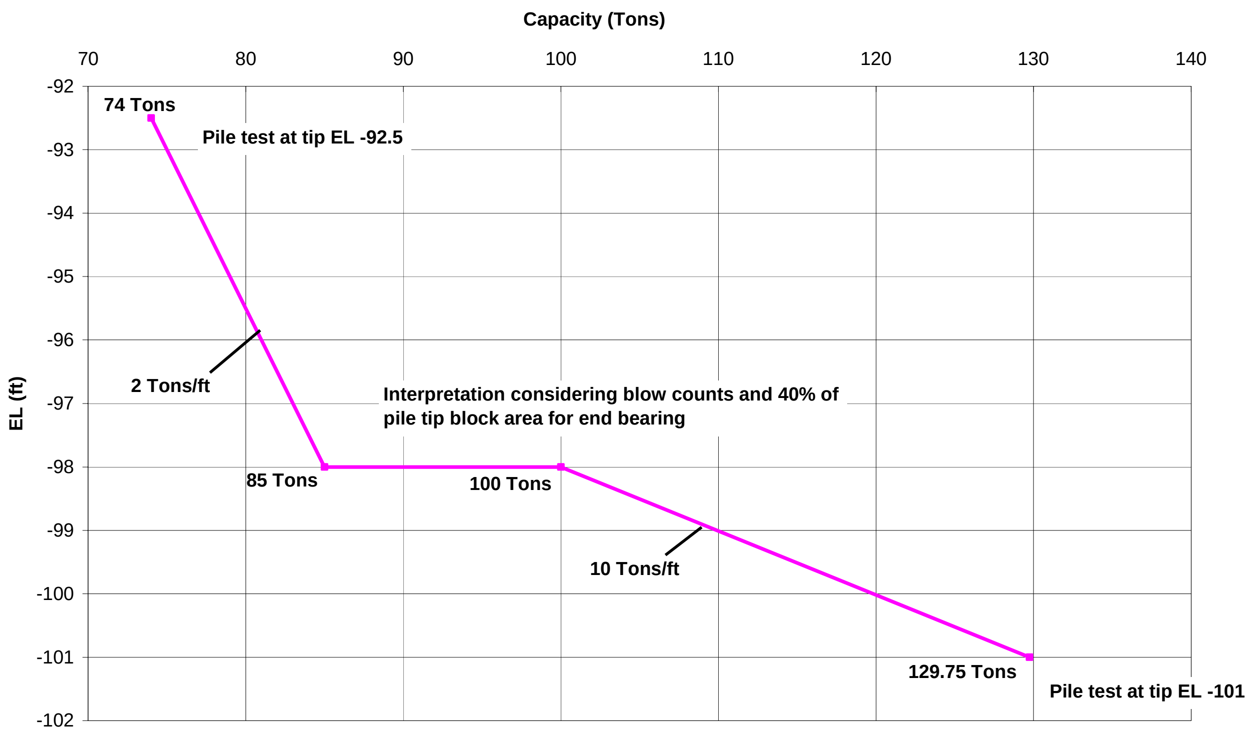


Figure 5. Pile Load Test Data

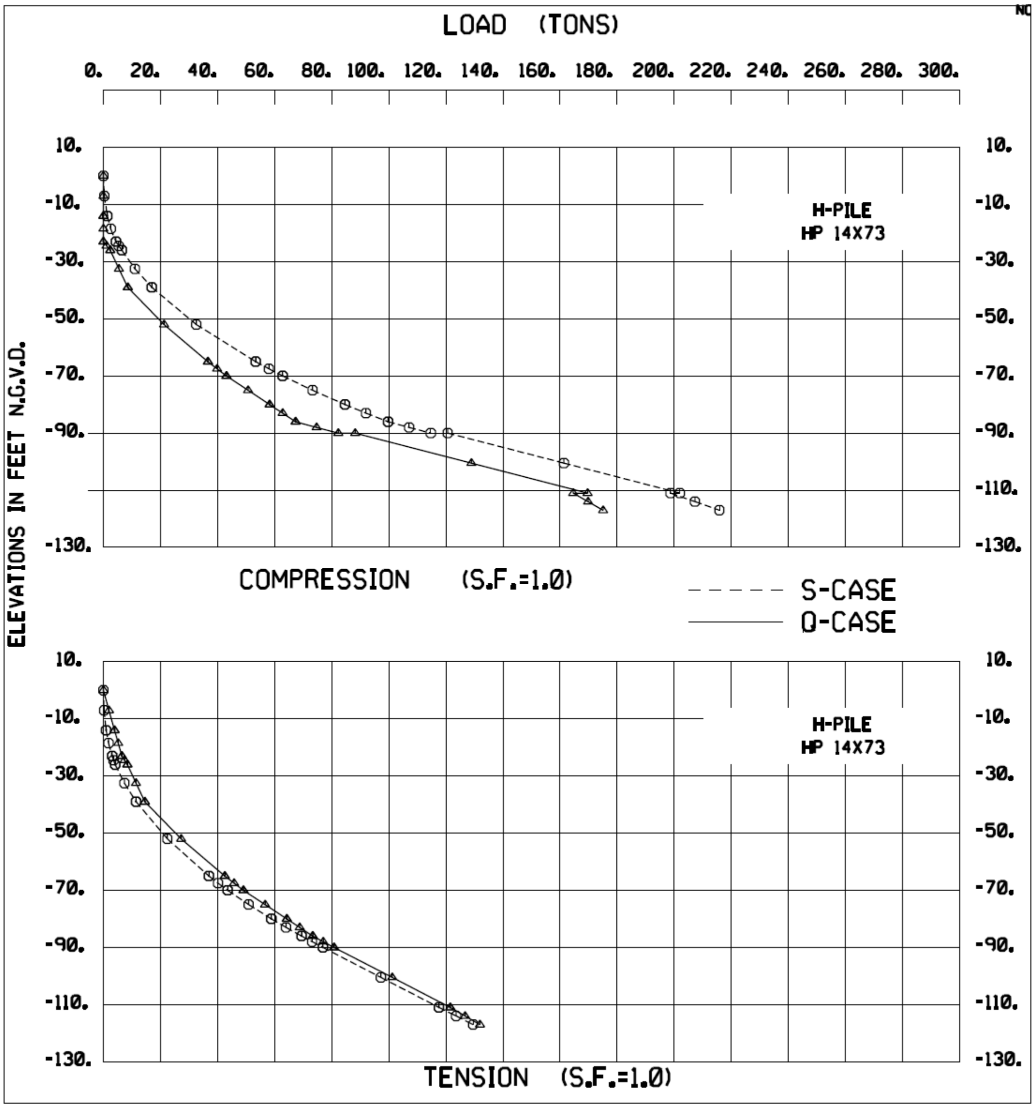


Figure 6. Ultimate Axial Capacity with Depth, Calculated

3.1 Alternate Method. If load tests are not performed, or allowable capacities computed from an ultimate strength method like APile or CAXPile, the axial pile capacities can be determined using TZPILE analyses that simulate lateral and axial pile load tests. The soil profiles used in these analyses are presented in Figure 7. The depth scale is in inches. The simulated load tests (after stripping off the top two layers) were performed at Elevation -23 which is the lowest elevation of the critical circle from Step 1.

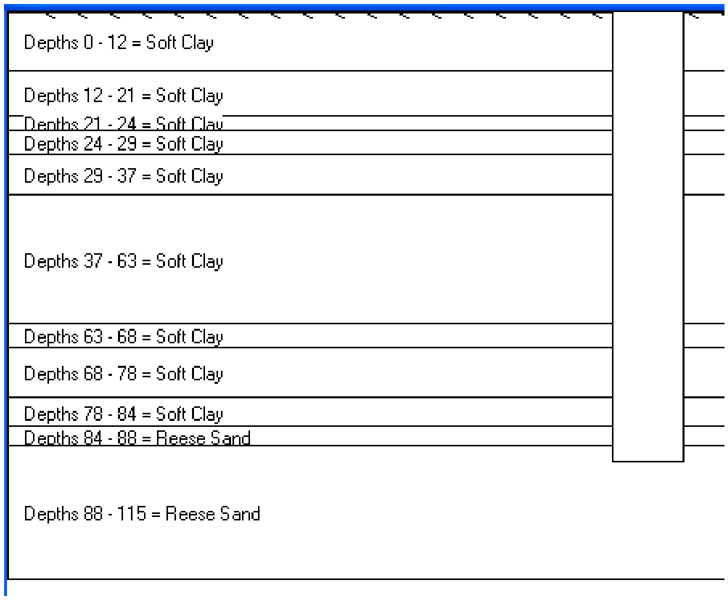


Figure 7. Soil Profiles - Stripped to critical surface of minus 23 for TZPILE and LPILE analysis

A plot of the TZPILE compression load versus settlement (at the pile head) is presented in Figure 8. The allowable compressive load is 58 kips based on and ultimate load of 174 kips and a factor of safety equal to 3.0 (assuming no pile load tests will be performed and no load case related reductions are applicable). Note that the ultimate of 174 kips (87 tons) is approximately equal to the pile capacity curves in Figure 5.

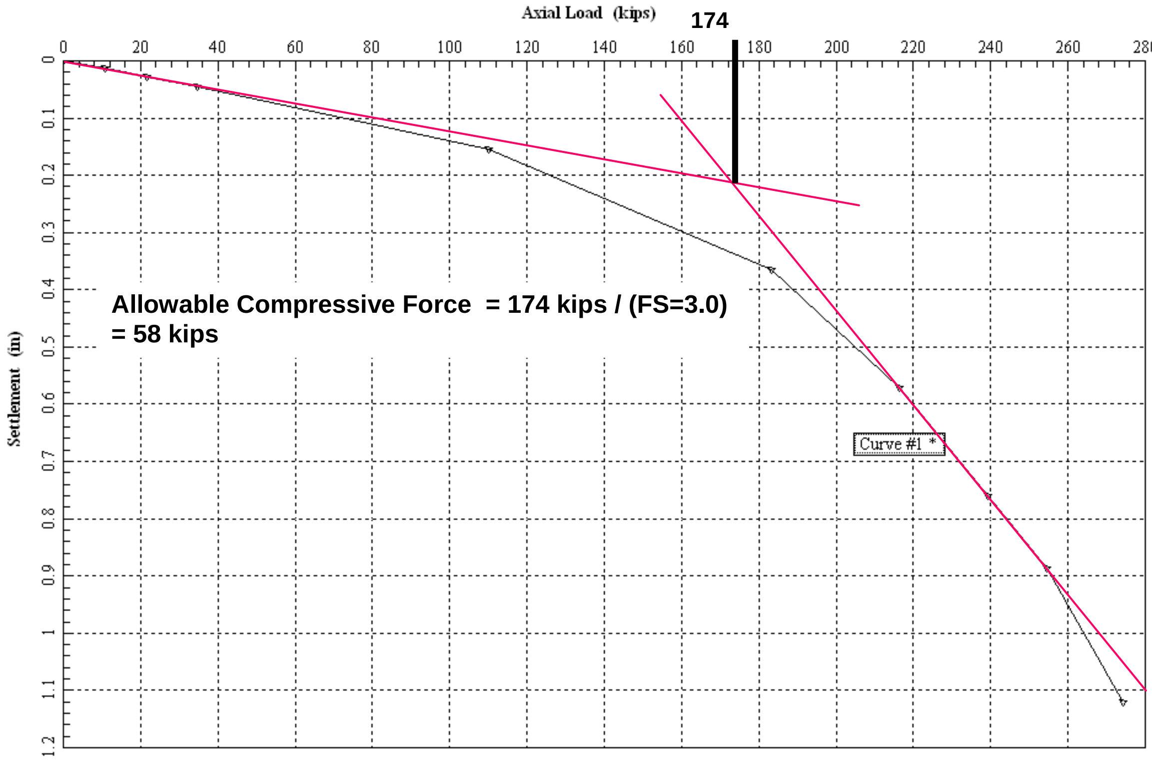


Figure 8. TZPILE Axial Pile Analysis Compression Settlement vs Axial Load Plot for determination of allowable compressive loads in piles by load simulation method.

Similarly, the allowable tensile capacity for a pile can be determined from analysis using the load simulation method. As shown in Figure 9, the ultimate tensile capacity is computed to be 84 kips. The allowable tensile capacity is determined by dividing the ultimate load by the factor of safety of 3.0 (assuming no pile load tests were performed and no load case related reductions are applicable). Thus, the allowable tensile load is 28 kips. This is less than the tension load computed above, but is presented as an example only and is not used in later design. Most likely there is a discrepancy in assumptions in stratigraphy or ultimate strength.

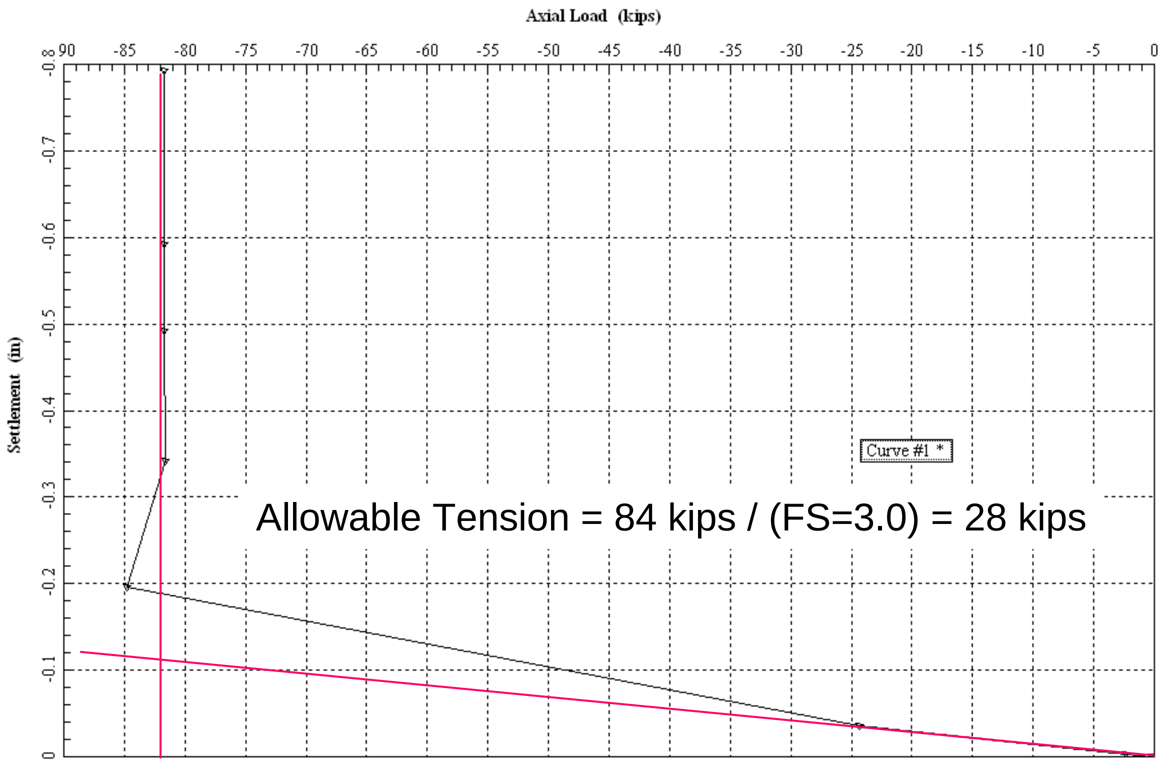


Figure 9. TZPILE Axial Pile Analysis TENSION Settlement vs Load Plot for allowable tensile loads in Piles

3.2 The allowable shear load (from LPILE) is determined from pile head deflection versus lateral load plot on Figure 10. The ultimate load was determined to be 24.5 kips. The allowable load is determined to be 8.2 kips after dividing by the factor of safety of 3.0.

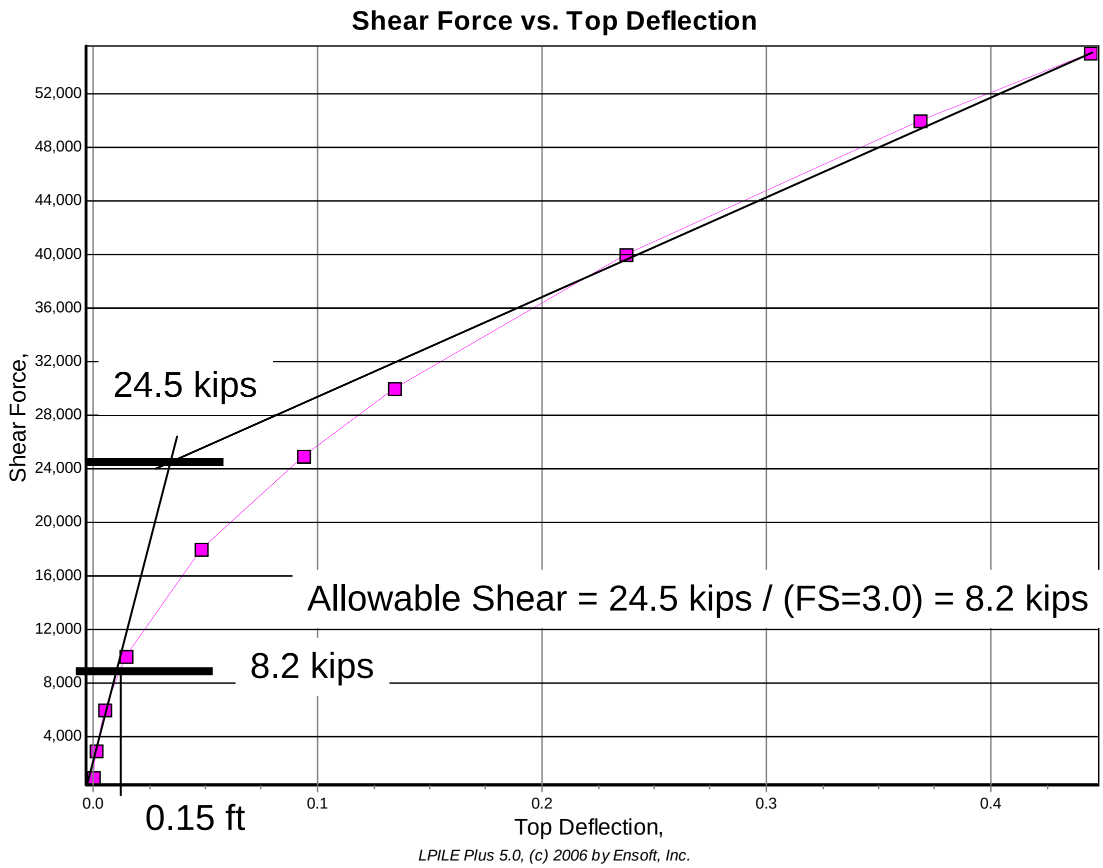


Figure 10. LPILE analysis of Pile head deflection vs shear force at critical surface to determine allowable shear force in piles.

Table 1 tabulates the allowable loads for axially loaded compressive and tensile piles,

Table 1. Allowable Axial and shear loads	
Type	Force (kips)
Axial Compressive	74
Axial Tensile	54
Shear	8.2

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. The unbalanced force is converted to an "equivalent" force applied to the bottom of the T-wall, $F_{cap,}$ as calculated as shown below (See Figure 11):

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

Where:

F_{ub} = unbalanced force computed in step 2.

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

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

$$R = \sqrt[4]{\frac{EI}{Es}}$$

E = Modulus of Elasticity of Pile (lb/in^2)

I = Moment of Inertia of Pile (in^4)

Es = Modulus of Subgrade Reaction (lb/in^2) below critical failure surface. In New Orleans District this equates to the values listed as K_HB.

For the solution:
Piles = HP 14x73. I = 729 in^4, E = 29,000,000 psi

Soils – Importance of lateral resistance decreases rapidly with depth, therefore only first three layers are input – with the third assumed to continue to the bottom of the pile. The parameters were developed from soil borings from the New Orleans District shown in Figure 12.

Silt, $\phi = 15$, C = 200 psf, γ_{sat} = 117 pcf, K_HB ave. = k =167 psi
Clay 1, $\phi = 0$, C = 200 psf, γ_{sat} = 100 pcf, K_HB = k = 88.8 psi
Clay 2, $\phi = 0$, C = 374 psf, γ_{sat} = 100 pcf, K_HB = k = 165.06 psi

The top layer of silt under the critical failure surface is stiffer but only three feet thick. Will use a k = 100 psi.

R therefore is equal to 121 in = 10.08 feet

P_{cap} = 4,575 * (22.5/2 + 10.08) / (18 + 10.08) = 3,475 lb/ft

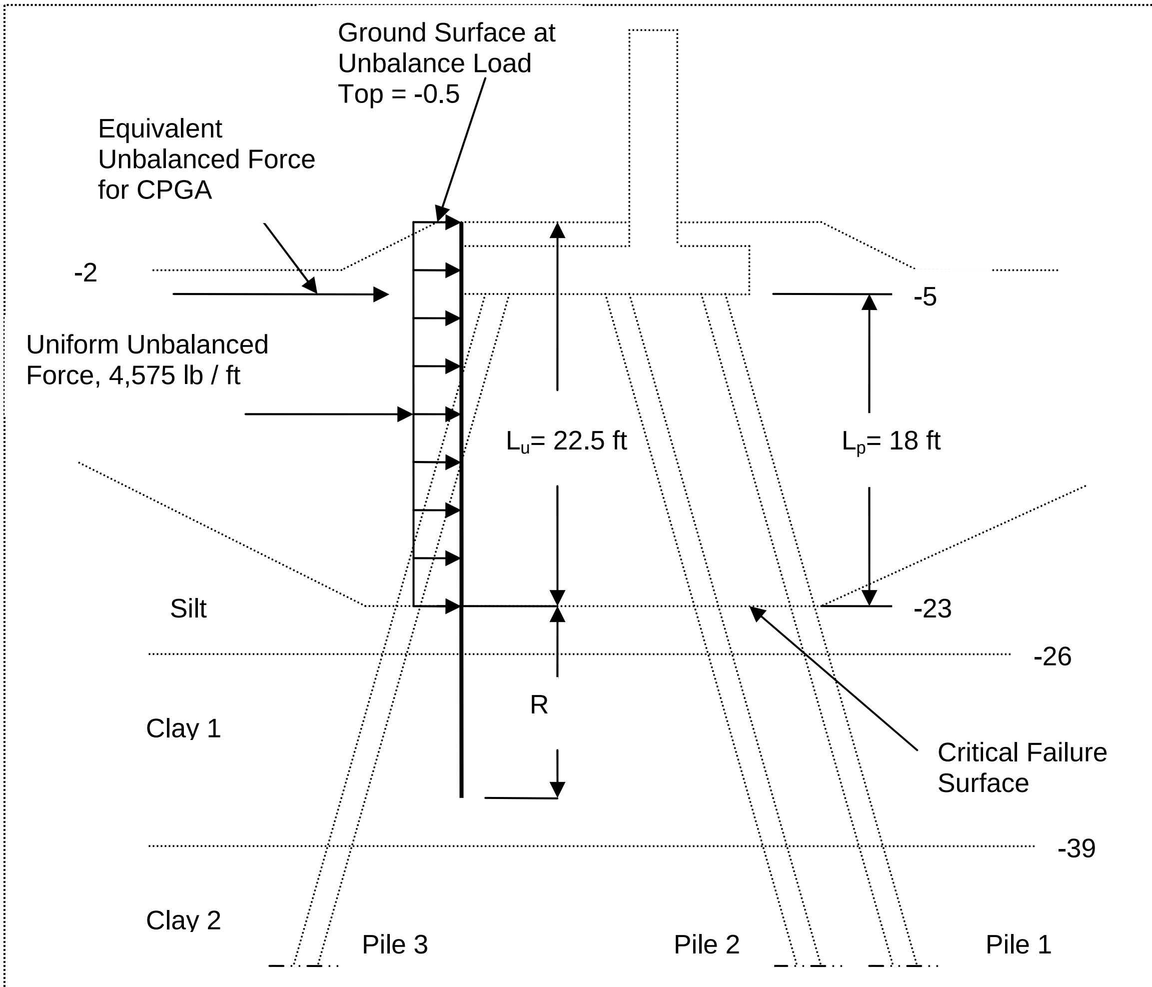


Figure 11. Equivalent Force Computation for Preliminary Design With CPGA

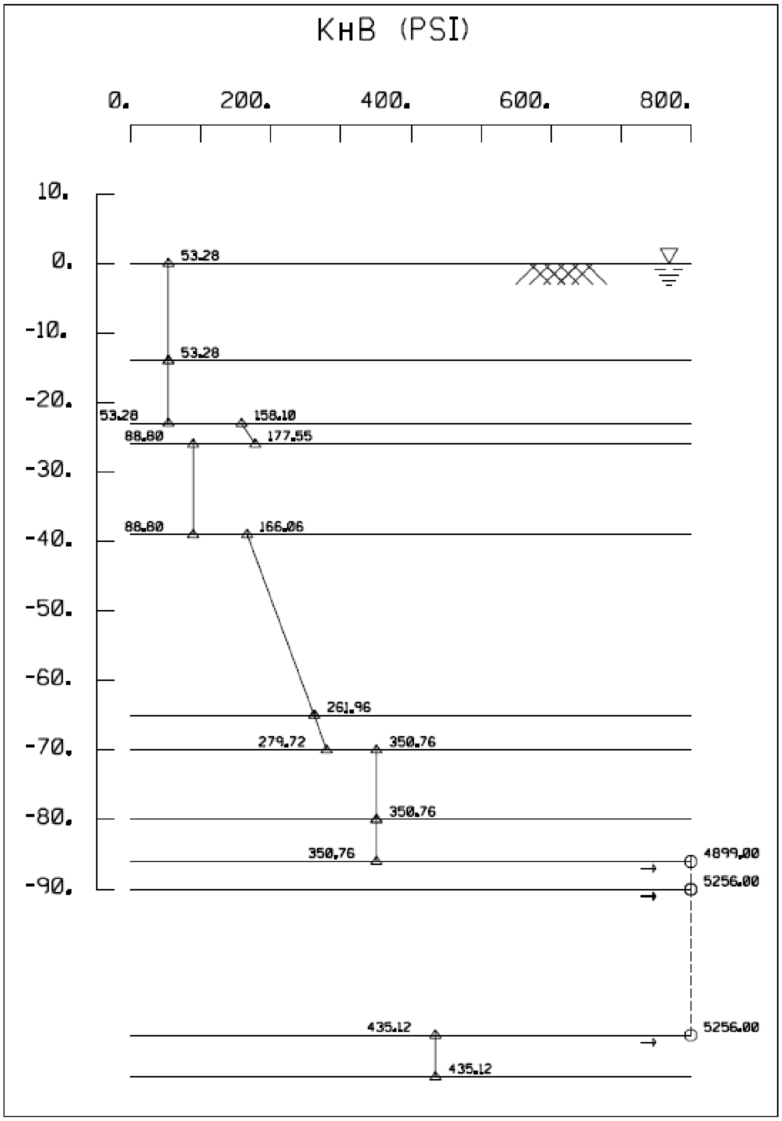


Figure 12. Soil Stiffness with Depth

4.2 This unbalanced force, $P_{cap,}$ is then analyzed with appropriate load cases in CPGA. Generally 8 to 20 load cases may be analyzed depending on expected load conditions. For this example, only the still water case is analyzed but both pervious and impervious foundation conditions are evaluated. See the spreadsheet calculations in Attachment 3 for the computation of the input for CPGA. The model is a 5 foot strip of the pile foundation.

For the CPGA analysis, the soil modulus, Es is adjusted based on the global stability factor of safety. For this example case, the factor of safety is 1.02. Es for CPGA is compute from the ratio of the computed factor of safety to the target factor of safety. From Figure 12, Es at the bottom of the wall footing is about 53.3 psi.

CPGA Es. = (1.02-1.0) / (1.5 – 1.0) * 53.3 = 2.1 psi

4.3 This is already a low value, but group factors from EM 1110-2-2906 can also be added. From page 4-35 of the EM with a spacing to pile diameter ratio of 5 ft / (14/12) = 4B, the reduction is 2.6. Es is therefore 2.1/2.6 = 0.8 psi

The CPGA output is shown in Attachment 4. A summary of results for the two load conditions analyzed are shown below:

LOAD CASE - 1 Pervious Condition

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF	
1	.2	.0	1.5	.0	-31.9	.0	.02	.03	
2	.2	.0	104.6	.0	-29.4	.0	1.41	.35	*
3	-.2	.0	-50.5	.0	30.7	.0	1.03	.18	*

LOAD CASE - 2 Impervious Condition

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF	
1	.2	.0	8.9	.0	-29.6	.0	.12	.05	
2	.1	.0	101.9	.0	-27.3	.0	1.38	.34	*
3	-.2	.0	-46.1	.0	28.7	.0	.94	.16	

Where:
F1 = Shear in pile at pile cap perpendicular to wall
F2 = Shear in Pile at Pile Cap parallel to wall
F3 = Axial Load in Pile
M1 = Maximum moment in pile perpendicular to wall
M2 = Maximum moment in pile parallel to wall
M3 = Torsion in pile
ALF= Axial load factor – computed axial load divided by allowable load
CBF= Combined Bending factor – combined computed axial and bending forces relative to allowable forces

Allowable axial pile capacities used for this analysis, 74 kips compressive and 49 kips tensile, were shown in step 3. The maximum pile forces computed in the middle piles exceed these values. This would require deeper piles or perhaps a revision of the pile layout. From Figure 4, and a factor of safety of 2 for an allowable pile capacity from pile load test data, to reach an allowable of 105 kips (ultimate of 210 kips or 105 tons), the piles only need to be increase to about 99 feet in length. This is not much difference, and the next steps will continue with the layout as shown. The tension piles have slightly exceeded the allowable capacity and could be made a few feet deeper to achieve required loads as well.

Computed deflections from the CPGA analysis are shown below:

```
         PILE CAP DISPLACEMENTS

 LOAD
 CASE        DX          DZ          R
             IN          IN         RAD

    1   -.7241E+00  -.2963E+00  -.3212E-02
    2   -.6757E+00  -.2609E+00  -.2899E-02
```

These deflections are less than the allowable vertical deflection (DZ) of 0.5 inches and allowable horizontal deflection (DX) of 0.75 inches from the Hurricane and Storm Damage Reduction Design Guidelines.

4.4 Sheet pile design. Seepage design of the sheet pile is not performed for this example.

4.5 Check for resistance against flow through. Since the pile spacing is uniform, we will analyze one row of piles parallel with the loading rather than the entire monolith.

a. Compute the resistance of the flood side row of piles.

$$\sum P_{all} = \frac{n \sum P_{ult}}{1.5}$$

Where:

n = number of piles in the row within a monolith. Or, for monoliths with uniformly spaced pile rows, n = 1. Use 1 for this example

$P_{ult} = \beta(9S_u b)$

S_u = soil shear strength

b = pile width = 14”

β = group reduction factor pile spacing parallel to the load - since the piles batter opposite to each other, there group affects are not computed.

For the soils under the slab, S_u = 120 psf

Therefore: P_{ult} = 9(120 psf)(14 in/12 in/ft) = 1,260 lb/ft

ΣP_{ult} = summation of P_{ult} over the height L_p, as defined in paragraph 4.1

For single layer soil is P_{ult} multiplied by L_p (18 ft) - That is the condition here since the shear strength is constant from the base to the critical failure surface.

ΣP_{ult} = 1,260 lb/ft (18 ft) = 22,680 lb

ΣP_{all} = 1(22,680 lb)/1.5 = 15,120 lb

b. Compute the load acting on the piles below the pile cap.

$$F_{up} = w f_{ub} L_p$$

Where:

w = Monolith width. Since we are looking at one row of piles in this example, w = the pile spacing perpendicular to the unbalanced force (s_t) = 5 ft.

$$f_{ub} = \frac{F_{ub}}{L_u}$$

F_{ub} = Total unbalanced force per foot from Step 2 = 4,575 lb/ft
L_u = 22.5 ft
L_p = 18 ft

f_{ub} = 4,575 lb/ft / 22.5 ft = 203 lb/ft/ft

F_p = 5 ft(203lb/ft/ft)(18ft) = 18,270 lb

c. Check the capacity of the piles 50% of F_p = 18,270 lb(0.50) = 9,135 lb

The capacity ΣP_{all} = 15,120 lb > 9,135 lb so OK for flow-through with this check.

4.6 Second flow through 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:

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

Where:
A_pS_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. – See Figure 13. S_u =120 psf
A_pS_u = (18(10+22)/2)(120 psf) = 34,560 lb
FS = Target factor of safety used in Steps 1 and 2. – 1.5
s_t= the spacing of the piles transverse (perpendicular) to the unbalanced force 5 ft
b = pile width – 14 inches

$f_{pb}L_p$ = (203 lb/ft)(18 ft) = 3,654 lb

$$\frac{A_pS_u}{FS}\left[\frac{2}{(s_t - b)}\right] = \frac{34{,}560}{1.5}\left[\frac{2}{5 - \left(\frac{14}{12}\right)}\right] = 12{,}021 \text{ lb}$$

Therefore, capacity against flow through is OK

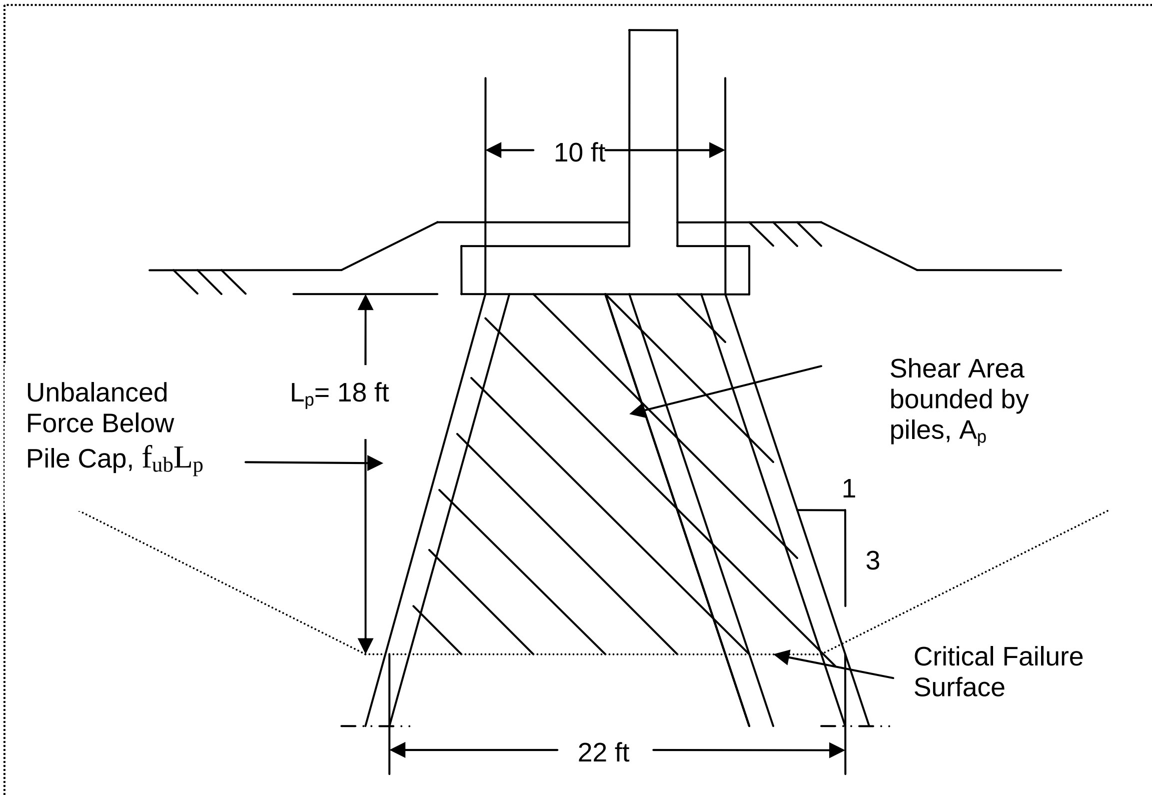


Figure 13. Shear Area for Flow Through Check

Step 5 Pile Group Analysis

5.1 A Group 7 analysis is performed using all loads applied to the T-wall structure. Critical load cases from step 4 would be used. In this example, only one load case with two foundation conditions is shown.

5.2 The loads applied in the Group 7 model include the distributed loads representing the unbalanced force that acts directly on the piles and also the water loads and self-weight of the wall that acts directly on the structure. In Group 7 these loads are resultant horizontal and vertical forces and the moments per width of spacing that act on the T-wall base (pile cap). They also include the unbalance force from the base of the cap to the top of soil, converted to a force and moment at the base of the structure. These forces are calculated using a worksheet or Excel spreadsheet and are shown at then end of the spreadsheets shown in Attachment 3. For this analysis the resultant forces per 5-ft of pile spacing were:

Impervious Foundation Condition

Vertical force	=	61,325 lb
Horizontal force	=	37,231 lb
Moment	=	1,540,666 in-lbs

Pervious Foundation Condition

Vertical force	=	52,731 lb
Horizontal force	=	37,231 lb
Moment	=	1,031,916 in-lbs

5.3 The unbalanced load below the bottom of the footing is applied directly as distributed loads on the pile. Check if ($n\Sigma P_{ult}$) of the flood side pile row is greater than 50% F_p, (from 4.5)
.

$(n\Sigma P_{ult}) = 1\ (22{,}680) = 22{,}680$ lb

50% F_p = 9,135 lb

Therefore distribute 50% of F_p onto the flood side (left) row of piles.

$0.5 f_{ub} s_t = 0.5$ (203 lb/ft/ft)(5 ft) = 507.5 lb/ft = 42 lb/in

The remainder is divided among the remaining piles.

Middle pile = 21 lb/in
Right pile = 21 lb/in

5.4 The group 7 model is illustrated in Figure 14.

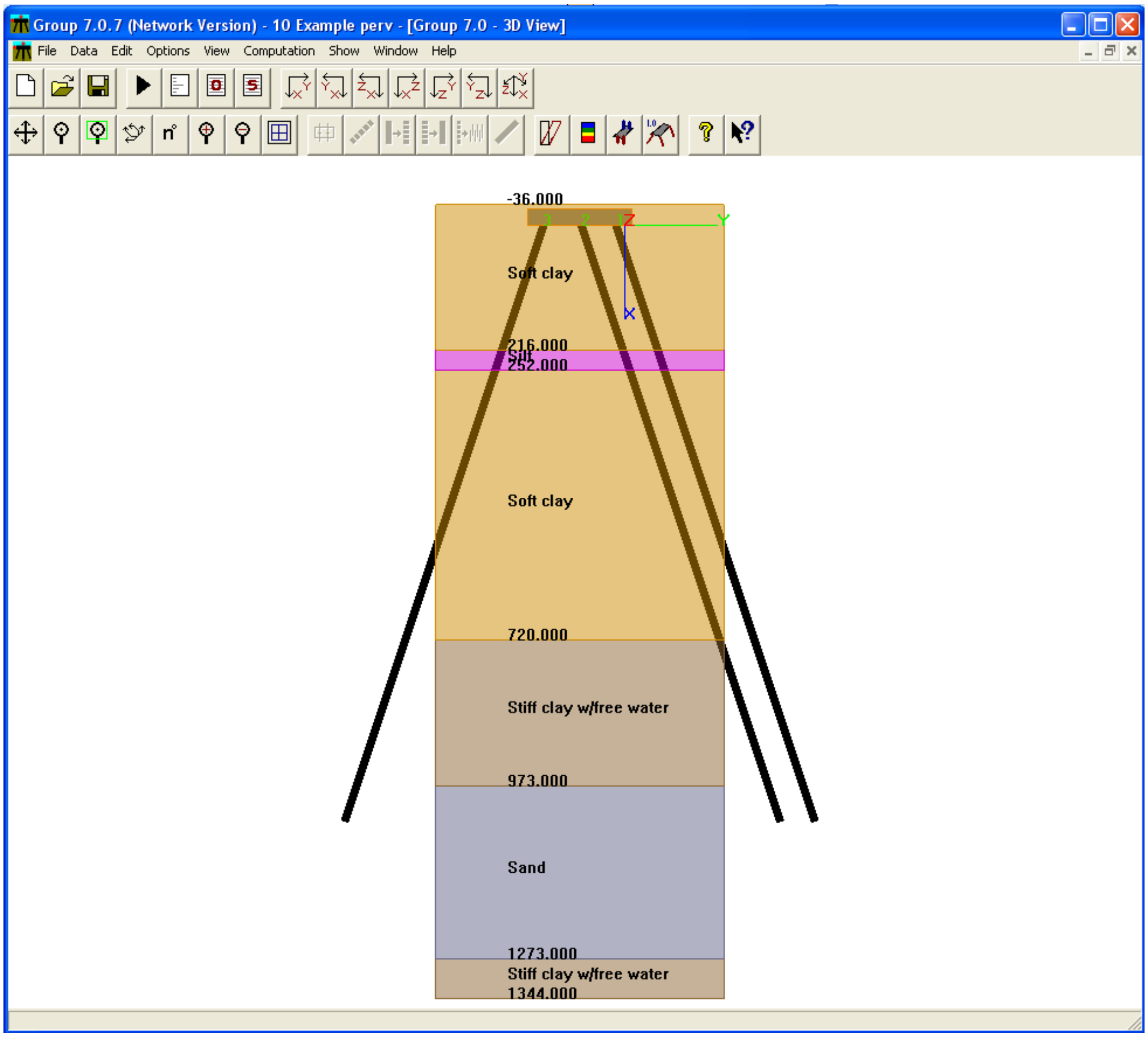


Figure 14. Group 7 Model with Soil Stratigraphy.

5.5 Additionally, in this analysis partial p-y springs can be used be cause the unreinforced factor of safety of 1.020 is between 1.0 and 1.5. The percentage of the full springs is determined as follows:

Partial spring percentage = (1.020 – 1.000)/ (1.5- 1.0) x 100% = 4%

Thus the strengths of in the top two layers, extending to Elevation -23 ft, were reduced to 4% of the undrained shear strength of 120 psf or 4.8 psf (0.0333 psi). The reduced undrained shear strength was used to scale the p-y curves above elevation -23 ft only. The results of the Group 7 analysis are listed in Table 1 where the pile responses for the full loading conditions on T-wall systems are listed. An example of the Group 7 output for the pervious condition are shown in Attachment 5

Table 2. Axial and shear Pile loads per 5-ft of width computed by Group 7 for full loading conditions that include distributed load in 50-25-25 split applied directly to piles and resultant horizontal, vertical and moments due to water loads and self weight applied directly to the structure			
Impervious Case	Left Pile	Center Pile	Right Pile
Axial Force (kips)	-35.3 (T)	88.5 (C)	11.6 (C)
Shear Force (kips)	4.49	2.4	2.7
Max. Moment (k-in)	-227	-199	-225
Pervious Case	Left Pile	Center Pile	Right Pile
Axial Force (kips)	-41.3 (T)	93.3 (C)	4.0 (C)
Shear Force (kips)	4.58	2. 5	2.7
Max. Moment (k-in)	-243	-219	-249

Figure 15 shows moment in the piles vs. depth and Figure 16 shows shear vs depth. There is no lateral soil stiffness from 0 to 216 inches.

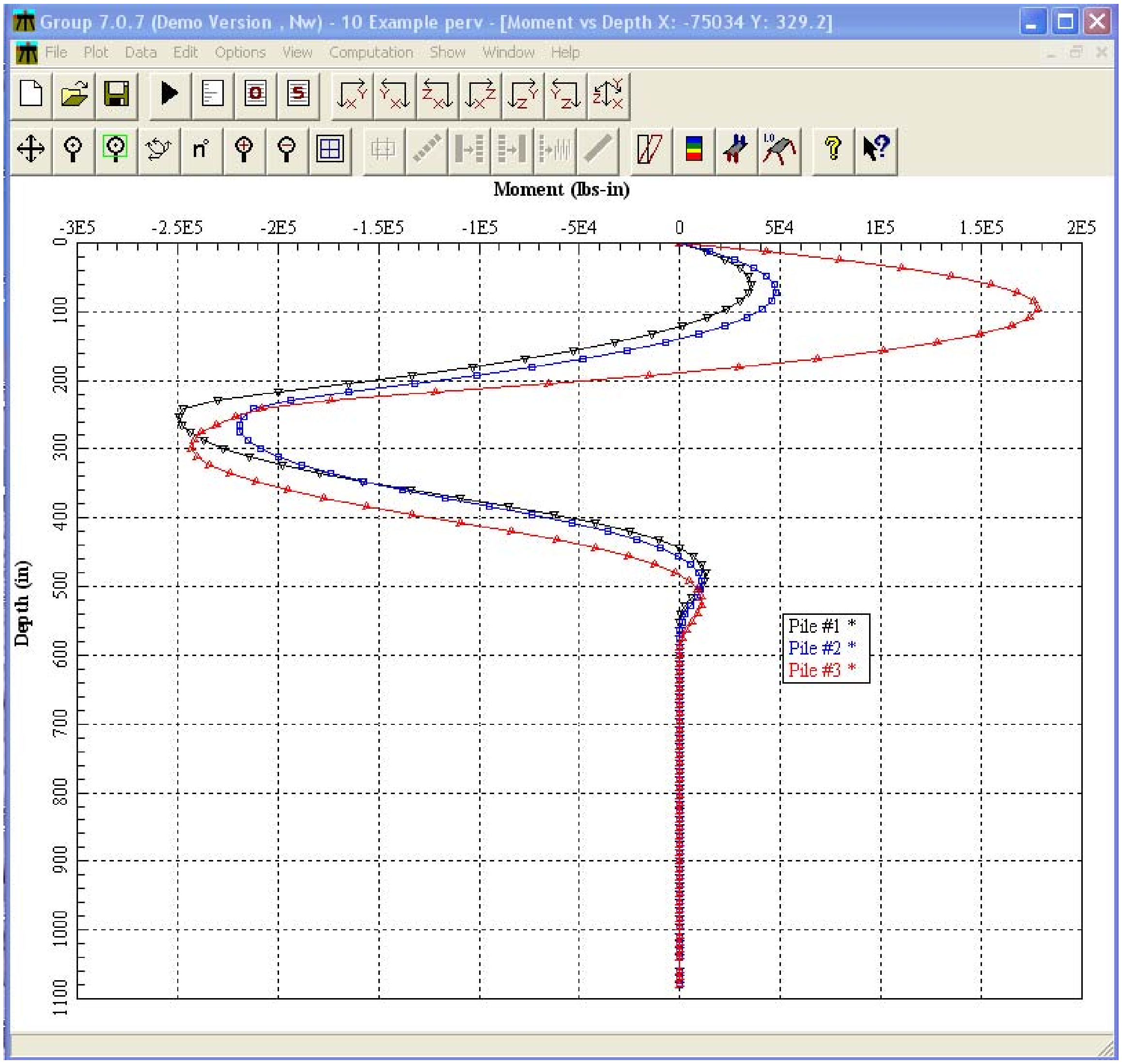


Figure 15. Moment vs depth.

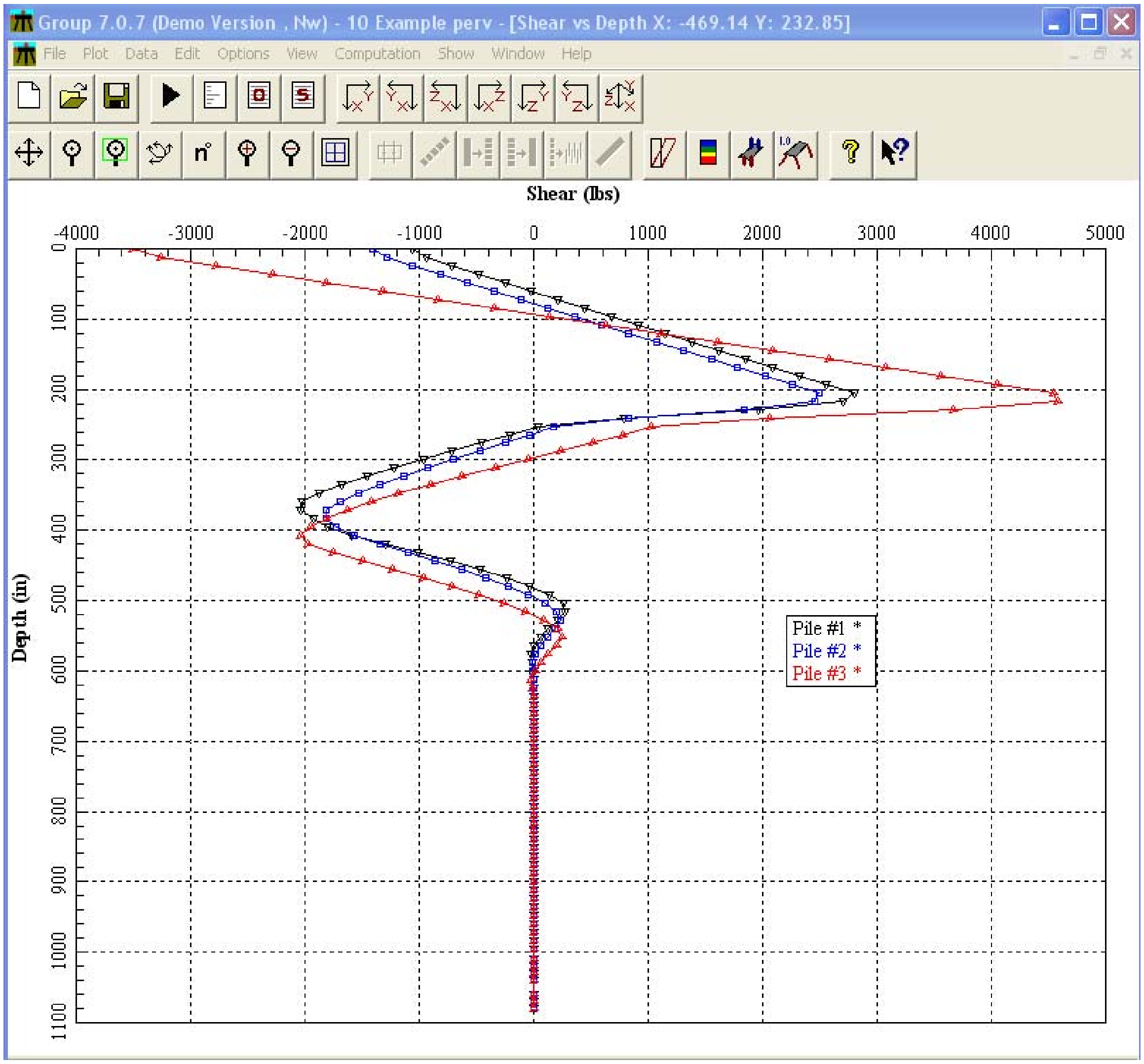


Figure 16. Shear vs depth

5.7 The axial forces and shear in Table 2 are then compared with allowable loads listed in Table 1. The results of the comparison show that:

a. the axial compressive forces in the center pile, 92.5 kips, exceeds the allowable compressive load of 74 kips.
b. the axial tensile force from the left (flood side) pile of -41.0 kips is less than the allowable tensile load of 54 kips.
c. The shear forces in each of the three piles are lower than the allowable shear of 8.2 kips.

Because the axial capacities of the center pile is exceeded, the pile layout must be repeated using a different pile layout. Axial forces and moment in the pile would be compared to allowable values computed according to EM 1110-2-2906. Moment and axial forces in the piles would also be checked for structural strength according to criteria in the Hurricane and Storm Damage Reduction System Design Guidelines and EM1110-2-2906.

Displacements from the Group 7 analysis are as follows:
Deflections

LOAD CASE	DX IN	DZ IN
Pervious	0.520	-0.20
Impervious	0.485	-0.18

These deflections are less than the allowable vertical deflection (DZ) of 0.5 inches and allowable horizontal deflection (DX) of 0.75 inches from the Hurricane and Storm Damage Reduction Design Guidelines.

Deflection of the piles vs. depth is shown in Figure 17.

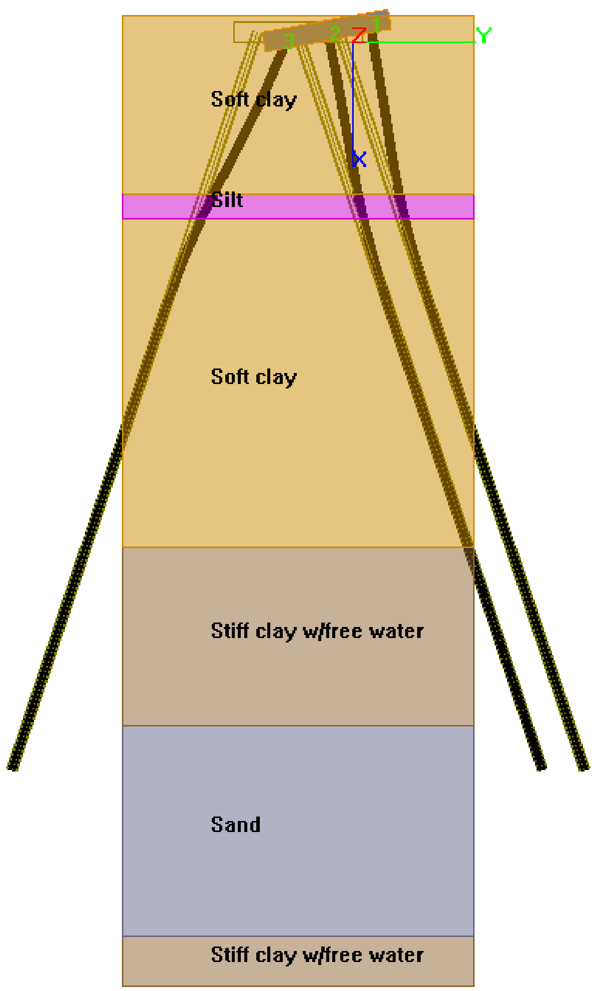


Figure 17 Deformed shape of pile cap

Deflection of the piles vs. depth is shown in Figure 18.

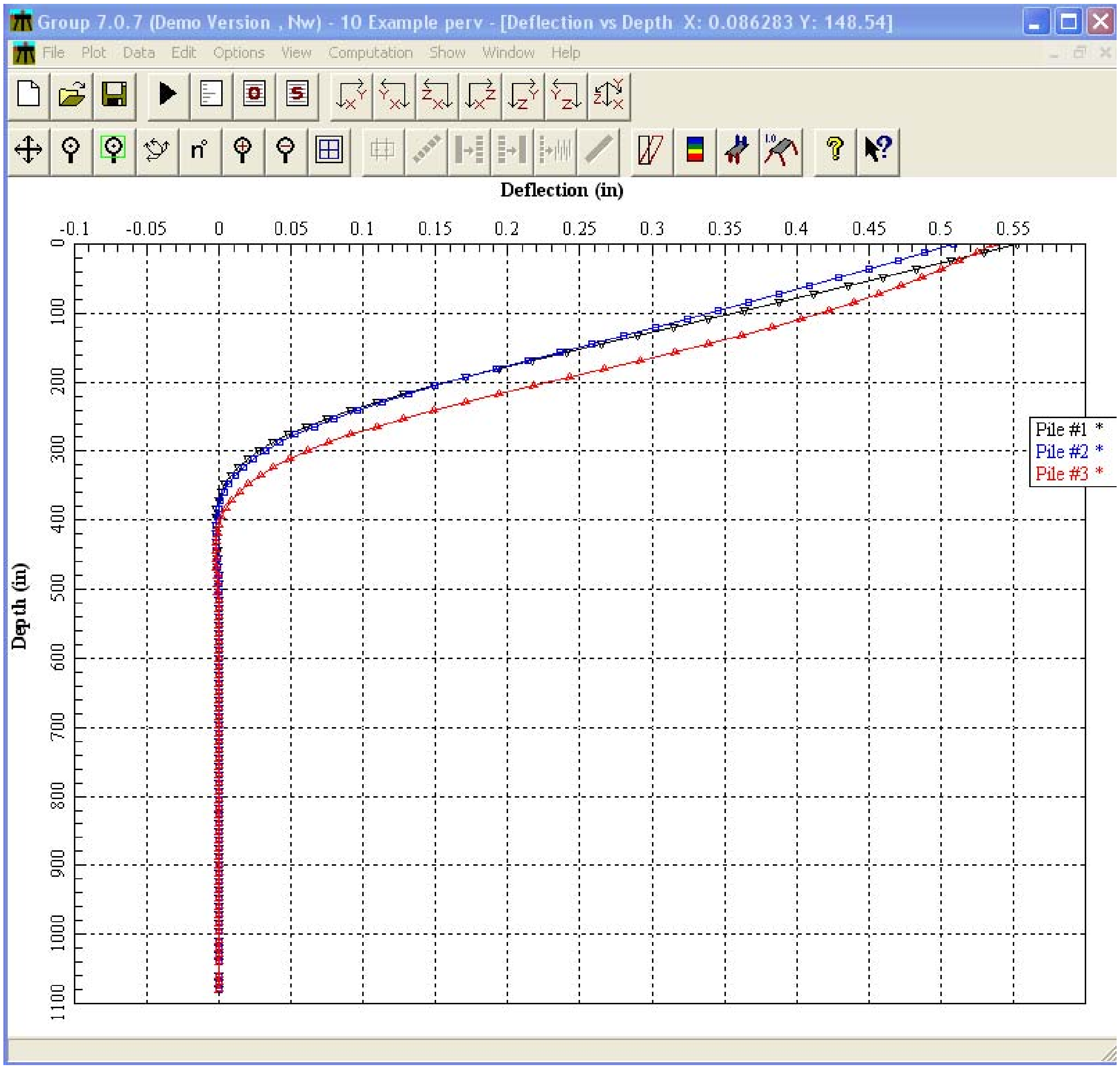


Figure 18 Deflection vs Depth

Step 6 Pile Group Analysis (unbalanced force)

6.1 Perform a Group 7 analysis with the distributed loads applied directly to the piles. The distributed loads are statically equivalent to the unbalanced force of 4,575 lb/ft. No loads are applied to the cap except unbalance forces. The p-y springs are set to 0 to the lowest critical failure surface elevation by setting the ultimate shear stress of these soils at a very low value. The distributed loads were computed in the previous step and are shown in the Excel spreadsheet computations shown in Attachment 3. Results of the Group analysis are shown below:

Table 3. Axial and shear Pile loads per 5-ft of width computed by Group 7

	Left Pile	Center Pile	Right Pile
Axial Force (kips)	-21.9 (T)	46.5 (C)	-24.5 (T)
Shear Force (kips)	4.24	2.32	2.48

Step 7 Pile Reinforced Slope Stability Analysis

7.1 The UT4 pile reinforcement analysis using the circle from Step 2 is performed to determine if the target Factor of Safety of 1.5 is achieved. The piles are treated as reinforcements in the UT4 and the shear and axial forces from Step 6 are used to determine these forces. The forces in Table 3 must be converted to unit width conditions by dividing by the 5-ft pile spacing to be used as the axial and shear forces in the pile reinforcements in UT4. The results of the analysis are shown in Figure 18. The factor of safety is 1.521 which exceeds that target factor of safety of 1.5. Therefore, the global stability of the foundation is verified in this Step. The input file is listed in Attachment 6.

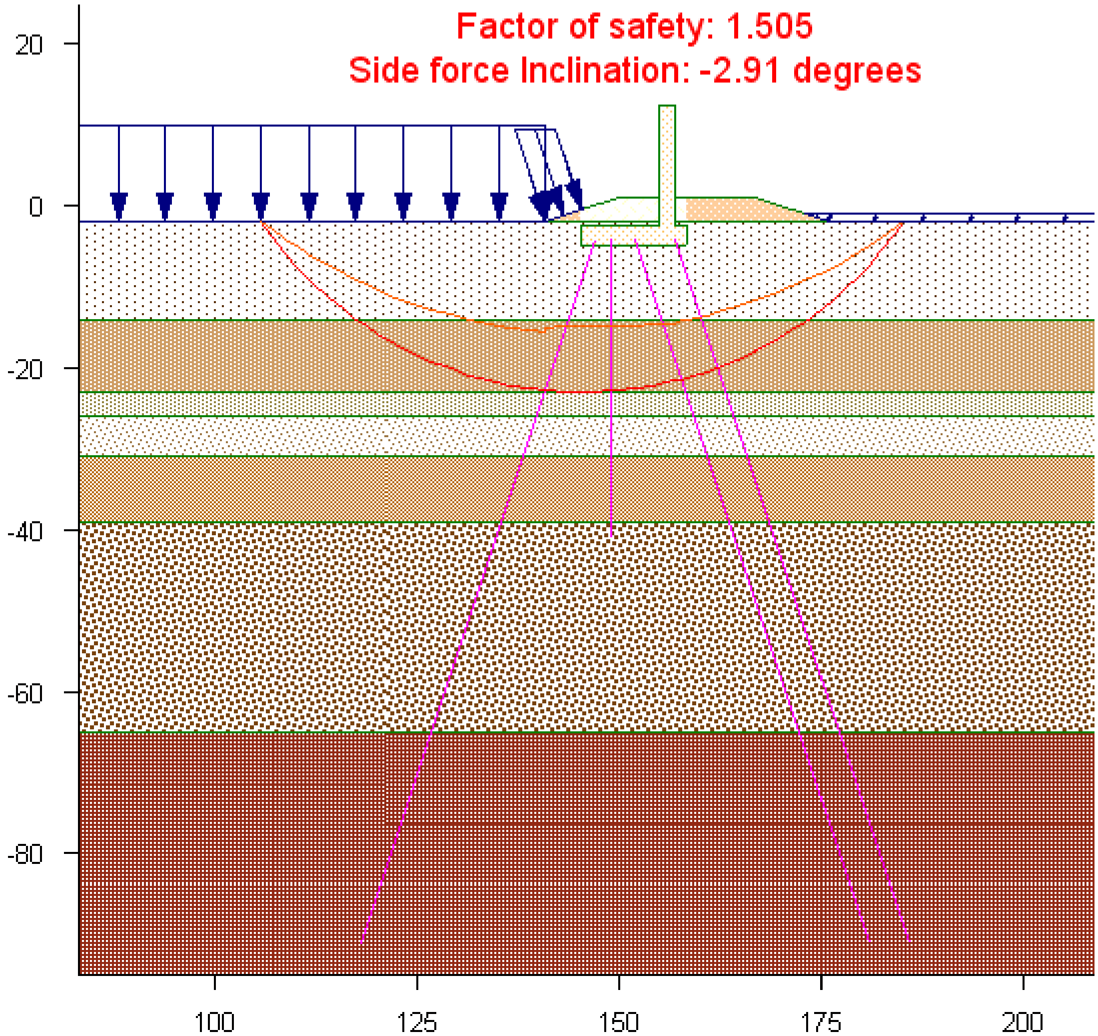


Figure 19. Factor of safety computed using pile forces from Group 7 analysis And critical circle from fixed grid analysis

7.2 Pile axial and shear forces determined in the pile group analysis are input in the slope stability analysis as longitudinal and transverse reinforcement forces. Sign convention for longitudinal forces in UTexas4 is that tensile forces are positive and compressive forces are negative. Sign convention for pile founded T-Walls with piles that extend below the critical failure surface and resist sliding of the soil mass is that transverse forces in UTexas4 are positive in the clockwise direction and negative in the counter-clockwise direction. This results in positive transverse forces in cases where the left side of the T-Wall is the flood side and negative transverse forces in cases where the right side of the T-Wall is the flood side. Positive longitudinal and transverse reinforcement forces for pile founded T-Walls are shown in Figure 20.

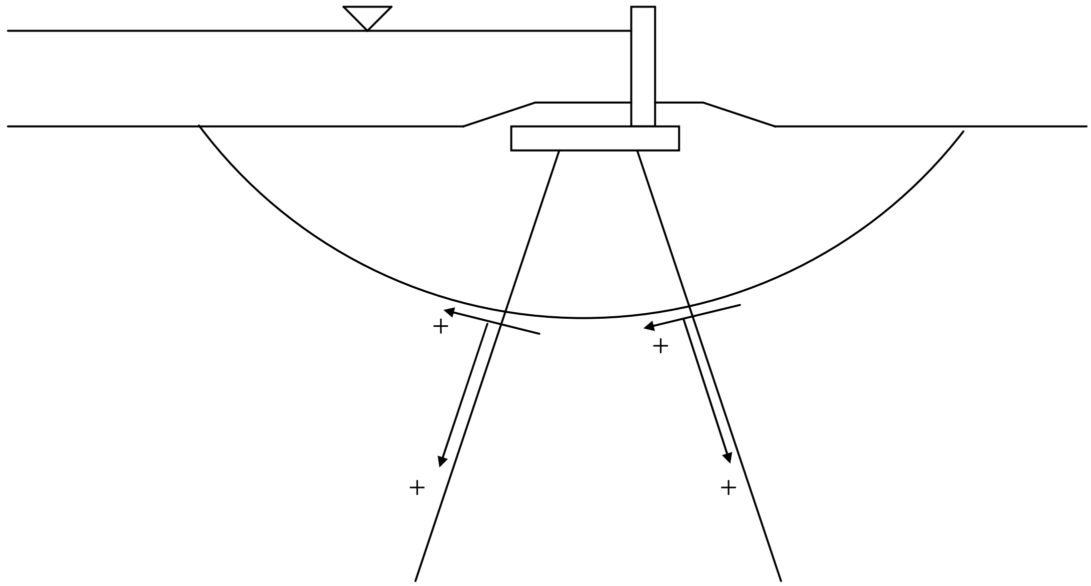


Figure 20. Positive directions for longitudinal and transverse reinforcement loads in pile.

Attachment 1 – Spencer's method analysis without piles that results in Figure 3.

```
HEADING
    T-Wall Deep Seated Analysis
    Analysis without piles

PROFILE LINES
         1     1 Layer 3 (CH) - Floodside
                  .00     -2.00
               141.00     -2.00
               155.00     -2.00

         2     1 Layer 3 (CH) - Landside
               157.00     -2.00
               375.00     -2.00

         3     2 Compacted Fill - FS
               141.00     -2.00
               145.50      -.50

         4     2 Compacted Fill - LS
               158.50      1.00
               167.00      1.00
               176.00     -2.00

         5     3 T-Wall
               145.50     -5.00
               145.50     -2.50
               155.00     -2.50
               155.00     -2.00
               155.00     12.30
               157.00     12.30
               157.00      1.00
               157.00     -2.00
               157.00     -2.50
               158.50     -2.50
               158.50     -5.00

         6     1 Layer 3 (CH) - Under Wall
               145.50     -5.00
               158.50     -5.00

         7     4 Layer 4 (CH)
                  .00    -14.00
               375.00    -14.00

         8     5 Layer 5 (ML)
                  .00    -23.00
               375.00    -23.00

         9     6 Layer 6 (CH)
                  .00    -26.00
               375.00    -26.00

        10     7 Layer 7 (CH)
```

```
              .00     -31.00
           375.00     -31.00

      11     8 Layer 8 (CH)
              .00     -39.00
           375.00     -39.00

      12     9 Layer 9 (CH)
              .00     -65.00
           375.00     -65.00

      13    10 Compacted Fill - Above T Wall Base  FS
           145.50       -.50
           150.00       1.00
           155.00       1.00

      14    10 Compacted Fill - Above T Wall Base  LS
           157.00       1.00
           158.50       1.00


MATERIAL PROPERTIES
     1 Layer 3 (CH)
          80.00 Unit Weight
          Conventional Shear
              120.00        .00
          No Pore Pressure
     2 Compacted Fill
          110.00 Unit Weight
          Conventional Shear
              500.00        .00
          No Pore Pressure
     3 T Wall
          .00 Unit Weight
          Very Strong
     4 Layer 4 (CH)
          100.00 Unit Weight
          Conventional Shear
              120.00        .00
          No Pore Pressure
     5 Layer 5 (ML)
          117.00 Unit Weight
          Conventional Shear
              200.00      15.00
          Piezometric Line
          1
     6 Layer 6 (CH)
          100.00 Unit Weight
          Conventional Shear
              200.00        .00
          No Pore Pressure
     7 Layer 7 (CH)
          100.00 Unit Weight
          Linear Increase
              217.00       8.10
          No Pore Pressure
     8 Layer 8 (CH)
```

```
          100.00 Unit Weight
          Linear Increase
              374.00      8.30
          No Pore Pressure
     9 Layer 9 (CH)
          100.00 Unit Weight
          Linear Increase
              590.00      8.00
          No Pore Pressure
     10 Compacted Fill - Above T-Wall Base
          .00 Unit Weight
          Conventional Shear
                 .00        .00
          No Pore Pressure

PIEZOMETRIC LINES
         1      62.40 Water Level
                 .00     10.00
              145.50     10.00
              145.51     -1.00
              157.00     -1.00
              375.00     -1.00

         2      62.40 Piezometeric levels in ML
                 .00     10.00
              149.50     10.00
              156.00     10.00
              158.50      1.00
              167.00      1.00
              173.00     -1.00
              375.00     -1.00


DISTRIBUTED LOADS
      1
ANALYSIS/COMPUTATION
     Circular Search 1
         146     22      1.00    -100.00       .00
     Tangent
          -23
SINgle-stage Computations
RIGht Face of Slope
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

Attachment 2 – Spencer's method analysis with unbalanced load that results in Figure 4.

```
HEADING
    T-Wall Deep Seated Analysis
    Analysis without piles

PROFILE LINES
         1     1 Layer 3 (CH) - Floodside
                  .00      -2.00
               141.00      -2.00
               155.00      -2.00

         2     1 Layer 3 (CH) - Landside
               157.00      -2.00
               375.00      -2.00

         3     2 Compacted Fill - FS
               141.00      -2.00
               145.50       -.50

         4     2 Compacted Fill - LS
               158.50       1.00
               167.00       1.00
               176.00      -2.00

         5     3 T-Wall
               145.50      -5.00
               145.50      -2.50
               155.00      -2.50
               155.00      -2.00
               155.00      12.30
               157.00      12.30
               157.00       1.00
               157.00      -2.00
               157.00      -2.50
               158.50      -2.50
               158.50      -5.00

         6     1 Layer 3 (CH) - Under Wall
               145.50      -5.00
               158.50      -5.00

         7     4 Layer 4 (CH)
                  .00     -14.00
               375.00     -14.00

         8     5 Layer 5 (ML)
                  .00     -23.00
               375.00     -23.00

         9     6 Layer 6 (CH)
                  .00     -26.00
               375.00     -26.00

        10     7 Layer 7 (CH)
```

```
              .00     -31.00
           375.00     -31.00

     11      8 Layer 8 (CH)
              .00     -39.00
           375.00     -39.00

     12      9 Layer 9 (CH)
              .00     -65.00
           375.00     -65.00

     13     10 Compacted Fill - Above T Wall Base   FS
           145.50       -.50
           150.00       1.00
           155.00       1.00

     14     10 Compacted Fill - Above T Wall Base   LS
           157.00       1.00
           158.50       1.00


MATERIAL PROPERTIES
     1 Layer 3 (CH)
          80.00 Unit Weight
          Conventional Shear
              120.00         .00
          No Pore Pressure
     2 Compacted Fill
          110.00 Unit Weight
          Conventional Shear
              500.00         .00
          No Pore Pressure
     3 T Wall
          .00 Unit Weight
          Very Strong
     4 Layer 4 (CH)
          100.00 Unit Weight
          Conventional Shear
              120.00         .00
          No Pore Pressure
     5 Layer 5 (ML)
          117.00 Unit Weight
          Conventional Shear
              200.00       15.00
          Piezometric Line
          1
     6 Layer 6 (CH)
          100.00 Unit Weight
          Conventional Shear
              200.00         .00
          No Pore Pressure
     7 Layer 7 (CH)
          100.00 Unit Weight
          Linear Increase
              217.00        8.10
          No Pore Pressure
     8 Layer 8 (CH)
```

```
          100.00 Unit Weight
          Linear Increase
              374.00      8.30
          No Pore Pressure
     9 Layer 9 (CH)
          100.00 Unit Weight
          Linear Increase
              590.00      8.00
          No Pore Pressure
     10 Compacted Fill - Above T-Wall Base
          .00 Unit Weight
          Conventional Shear
                 .00        .00
          No Pore Pressure

PIEZOMETRIC LINES
         1     62.40 Water Level
                 .00     10.00
              145.50     10.00
              145.51     -1.00
              157.00     -1.00
              375.00     -1.00

         2     62.40 Piezometeric levels in ML
                 .00     10.00
              149.50     10.00
              156.00     10.00
              158.50      1.00
              167.00      1.00
              173.00     -1.00
              375.00     -1.00


DISTRIBUTED LOADS
      1
LINE LOADS
              1    145      -11.75 -4575.00       .00 1

ANALYSIS/COMPUTATION
     Circular Search 1
         145     22       0.50   -100.00       .00
     Tangent
          -23
SINgle-stage Computations
RIGht Face of Slope
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

UPDATED 23 OCT 07

Attachment 3 Structural Loads for CPGA and Group Analyses

US Army Corps of Engineers Saint Paul Distict	PROJECT TITLE: T-Wall Design Example	COMPUTED BY: KDH	DATE: 07/27/07	SHEET:
	SUBJECT TITLE: Water at El. 10', Pervious	CHECKED BY:	DATE:	

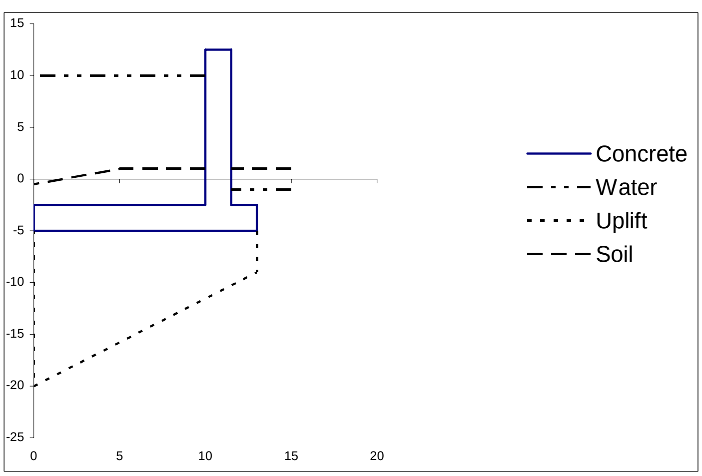
Input for CPGA pile analysis **Pervious Foundation Assumption**

Upstream Water Elevation	*10* ft		Back Fill Soil Elevation	*1*	ft
Downstream Water Elevation	*-1* ft		Front Fill Soil Elevation	*1*	ft
Wall Top Elevation	*12.5* ft		Gamma Water	*0.0625*	kcf
Structure Bottom Elevation	*-5* ft		Gamma Concrete	*0.15*	kcf
Base Width	*13* ft		Gamma Sat. Backfill	*0.110*	kcf
Toe Width	*1.5* ft		Distance to Backfill Break	*5.0*	ft
Wall Thickness	*1.5* ft		Slope of Back Fill	*0.30*	
Base Thickness	*2.5* ft		Soil Elevation at Heel	-0.50	ft

Vertical Forces								
Component	Height	x1	x2	Gamma	Force	Arm	Moment	
Stem Concrete	15	10	11.5	0.15	3.38	10.75	36.3	
Heel Concrete	2.5	0	11.5	0.15	4.31	5.75	24.8	
Toe Concrete	2.5	11.5	13	0.15	0.56	12.25	6.9	
Heel Water	9	0	10	0.0625	5.63	5	28.1	
Toe Water	1.5	11.5	13	0.0625	0.14	12.25	1.7	
Heel Soil	3.5	0	10	0.110	3.85	5	19.3	
-Triangle	1.50	0	5.0	-0.048	-0.18	1.67	-0.3	
Toe Soil	3.5	11.5	13	0.110	0.58	12.25	7.1	
Rect Uplift	-4	0	13	0.0625	-3.25	6.5	-21.1	
Tri Uplift	-11	0	13	0.0625	-4.47	4.3	-19.4	
Sum Vertical Forces					10.5		83.4	ft-k

Horizontal Forces								
Component	H1	H2	Gamma	Lat. Coeff.	Force	Arm	Moment	
Driving Water	10	-5	0.0625	1	7.03	5.00	35.16	
Resisting Water	-1	-5	0.0625	1	-0.50	1.33	-0.67	
Lateraral soil forces assumed equal and negligible								
Sum Horizontal Forces					6.53	5.28	34.49	ft-k

Total Structural Forces	Net Vert. Force	Arm	Moment	
About Heel	10.55	11.17	117.84	ft-k



Net Vertical Arm		
From Toe	1.83	ft

Moment About Toe
-19.3 ft-k

Model Width
5 ft

UPDATED 23 OCT 07

US Army Corps of Engineers Saint Paul Distict	PROJECT TITLE: T-Wall Design Example	COMPUTED BY: KDH	DATE: 07/27/07	SHEET:
	SUBJECT TITLE: Water at El. 10', Pervious	CHECKED BY:	DATE:	

Calculation of Unbalanced Force

Unbalanced Force. F_{ub}	*4,575*	lb/ft	From UTexas Analysis
Elevation of Critical Surface	*-23*	ft	From UTexas Analysis
Length - Ground to Crit. Surface, Lu	22.5	ft	(assume failure surface is normal to pile)
Length - Base to Crit. Surface, Lp	18	ft	
Pile Moment of Inertia. I	*729*	in^4	HP14x73
Pile Modulus of Elasticity E	*29,000,000*	lb/in^2	
Soil Modulus of Subgrade Reaction, k	*100*	lb/in^2	
Soil Stiffness Parameter, R	121	in	$(EI / k)^{1/4}$
Equivalent Unbalanced Force, Pcap	3,474	lb/ft	F_{ub} * (L_u/2 +R) / (L_p +R)

CPGA Input

PX	-50.03	kips
PY		
PZ	52.73	kips
MX	0	
MY	-96.29	kip-ft
MZ	0	

Group Input

3 Pile Rows Parallel to Wall Face

Unbalanced Loading on Piles for Group Analysis

Total	85	lb/in	F_{ub} * Model Width /L_u
50%	42	lb/in	For Pile on Protected Sied
25%	21	lb/in	

Note: Applied to length of pile from bottom of cap to top of critical surface. 18

Unbalanced Loads on Wall for Group Analysis of Just Unbalanced Forces

Distance From Base to Ground Surface, Ds 4.50 ft

PX	0	lb	
PY	4,575	lb	F_{ub} * Model Width / L_u * Ds
PZ	0	lb	
MX	0		
MY	0		
MZ	-123,525	lb-in	-PZ * Ds/2

Total Loads for Group Analysis

PX	52,731	lb	
PY	37,231	lb	PYub + Sum Horizontal * Model Width
PZ	0	lb	
MX	0		
MY	0		
MZ	1,031,916	lb-in	

US Army Corps of Engineers Saint Paul Distict	PROJECT TITLE: T-Wall Design Example	COMPUTED BY: KDH	DATE: 07/27/07	SHEET:
	SUBJECT TITLE: Water at El. 10', Impervious	CHECKED BY:	DATE:	

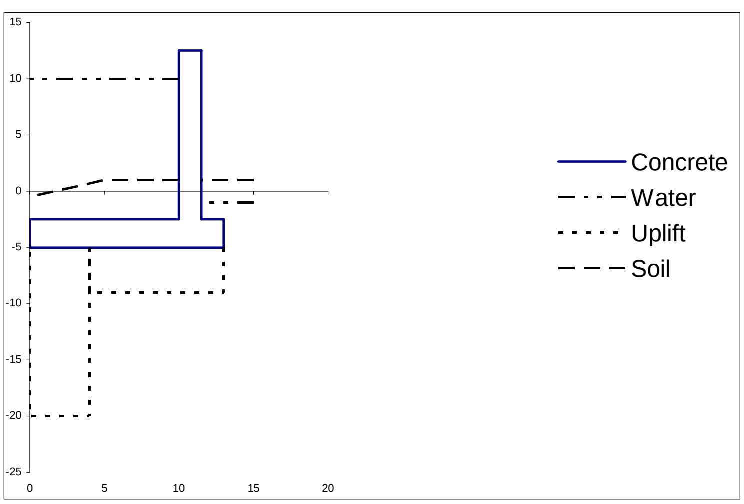
Input for CPGA pile analysis **Impervious Foundation Assumption**

Upstream Water Elevation	*10*	ft	Back Fill Soil Elevation	*1*	ft
Downstream Water Elevation	*-1*	ft	Front Fill Soil Elevation	*1*	ft
Wall Top Elevation	*12.5*	ft	Gamma Water	*0.0625*	kcf
Structure Bottom Elevation	*-5*	ft	Gamma Concrete	*0.15*	kcf
Base Width	*13*	ft	Gamma Soil	*0.110*	kcf
Toe Width	*1.5*	ft	Distance to Backfill Break	*5.0*	ft
Wall Thickness	*1.5*	ft	Slope of Back Fill	*0.30*	
Base Thickness	*2.5*	ft	Soil Elevation at Heel	-0.50	ft

Vertical Forces Component	Height	x1	x2	Gamma	Force	Arm	Moment	
Stem Concrete	15	10	11.5	0.15	3.38	10.75	36.3	
Heel Concrete	2.5	0	11.5	0.15	4.31	5.75	24.8	
Toe Concrete	2.5	11.5	13	0.15	0.56	12.25	6.9	
Heel Water	9	0	10	0.0625	5.63	5	28.1	
Toe Water	1.5	11.5	13	0.0625	0.14	12.25	1.7	
Heel Soil	3.5	0	10	0.110	3.85	5	19.3	
-Triangle	1.50	0	5.0	-0.048	-0.18	1.67	-0.3	
Toe Soil	3.5	11.5	13	0.110	0.58	12.25	7.1	
Prot. Side Uplift	-4	*4*	13	0.0625	-2.25	8.5	-19.1	
Flood Side Uplift	-15	0	4	0.0625	-3.75	2	-7.5	
Sum Vertical Forces					12.3	kip	97.2	ft-k

Horizontal Forces Component	H1	H2	Gamma	Lat. Coeff.	Force	Arm	Moment	
Driving Water	10	-5	0.0625	1	7.03	5.00	35.16	
Resisting Water	-1	-5	0.0625	1	-0.50	1.33	-0.67	
Lateraral soil forces assumed equal and negligible								
Sum Horizontal Forces					6.53	kip	34.49	ft-k

Total Structural Forces	Net Vert. Force	Arm	Moment	
About Heel	12.27	10.74	131.71	ft-k



Net Vertical Arm		
From Toe	2.26	ft

Moment About Toe
-27.7 ft-k

Model Width
5 ft

US Army Corps of Engineers Saint Paul Distict	PROJECT TITLE: T-Wall Design Example	COMPUTED BY: KDH	DATE: 07/27/07	SHEET:	
	SUBJECT TITLE: Water at El. 10', Impervious	CHECKED BY:	DATE:		

Calculation of Unbalanced Force

Unbalanced Force. F_{ub}	*4,575*	lb/ft	From UTexas Analysis
Elevation of Critical Surface	*-23*	ft	From UTexas Analysis
Length - Ground to Crit. Surface, Lu	23	ft	(assume failure surface is normal to pile)
Length - Base to Crit. Surface, Lp	18	ft	
Pile Moment of Inertia. I	*729*	in^4	HP14x73
Pile Modulus of Elasticity E	*29,000,000*	lb/in^2	
Soil Modulus of Subgrade Reaction, k	*100*	lb/in^2	
Soil Stiffness Parameter, R	121	in	$(EI / k)^{1/4}$
Equivalent Unbalanced Force, Pcap	3,474	lb/ft	$F_{ub} * (L_u/2 + R) / (L_p + R)$

CPGA Input

PX	-50.03	kips
PY		
PZ	61.33	kips
MX	0	
MY	-138.68	kip-ft
MZ	0	

Group Input

3 Pile Rows Parallel to Wall Face

Unbalanced Loading on Piles for Group Analysis

Total	85	lb/in	F_{ub} * Model Width /L_u
50%	42	lb/in	For Pile on Protected Sied
25%	21	lb/in	

Note: Applied to length of pile from bottom of cap to top of critical surface. 18 ft

Unbalanced Loads on Wall for Group Analysis of Just Unbalanced Forces

Distance From Base to Ground Surface, Ds 4.50 ft

PX	0	lb	
PY	4,575	lb	F_{ub} * Model Width / L_u * Ds
PZ	0	lb	
MX	0		
MY	0		
MZ	-123,525	lb-in	-PZ * Ds/2

Total Loads for Group Analysis

PX	61,325	lb	
PY	37,231	lb	PYub + Sum Horizontal * Model Width
PZ	0	lb	
MX	0		
MY	0		
MZ	1,540,666	lb-in	

Attachment 4 - Preliminary Analysis with CPGA

```
10 Geomatrix T-wall, Example
15 2.5 ft slab, hp 14 x 73 piles, pinned head, 3:1 batter
20 PROP 29000 261 729 21.4 1.0 0 all
30 SOIL ES 0.0008 "TIP" 87 0 all
40 PIN all
50 ALLOW H 74.0 49.0 315.8  315.8  520.6  1573.1 all
70 BATTER 3.0 1 2 3
80 ANGLE 180 1 2
180 PILE 1  1.500 0.00 0.00
201 PILE 2  6.500 0.00 0.00
202 PILE 3  11.50 0.00 0.00
230 LOAD 1 -50.03  0.0  52.73  0.00  -96.29
240 LOAD 2 -50.03  0.0  61.33  0.00  -138.68
334 FOUT 1 2 3 4 5 6 7 MVN10EXT.OUT
335 PFO ALL
```

```
*********************************
 * CASE PROGRAM   #  X0080       *  CPGA - CASE PILE GROUP ANALYSIS PROGRAM
 * VERSION NUMBER # 1993/03/29   *  RUN DATE 27-JUL-2007   RUN TIME 16.23.07
 *********************************


 GEOMATRIX T-WALL, EXAMPLE


 THERE ARE    3 PILES AND
              2 LOAD CASES IN THIS RUN.

 ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                    X          Y          Z
                                  -----      -----      -----
 WITH DIAGONAL COORDINATES = (      1.50 ,      .00 ,      .00 )
                             (     11.50 ,      .00 ,      .00 )


 ******************************************************************************

          PILE PROPERTIES AS INPUT


      E            I1            I2            A            C33           B66
     KSI          IN**4         IN**4        IN**2
  .29000E+05   .26100E+03   .72900E+03   .21400E+02   .10000E+01   .00000E+00

 THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

     ALL


 ******************************************************************************

          SOIL DESCRIPTIONS AS INPUT
```

```
   ES      ESOIL      LENGTH        L            LU
         K/IN**2                   FT            FT
         .80000E-03    T        .87000E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

    ALL


********************************************************************************

         PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM         X          Y          Z      BATTER    ANGLE    LENGTH  FIXITY
           FT         FT         FT                          FT

   1      1.50        .00        .00      3.00    180.00    91.71     P
   2      6.50        .00        .00      3.00    180.00    91.71     P
   3     11.50        .00        .00      3.00       .00    91.71     P
                                                           ------
                                                           275.12


********************************************************************************

                         APPLIED LOADS

LOAD      PX         PY         PZ          MX          MY          MZ
CASE       K          K          K         FT-K        FT-K        FT-K

  1     -50.0         .0       52.7          .0       -96.3          .0
  2     -50.0         .0       61.3          .0      -138.7          .0


********************************************************************************

         ORIGINAL PILE GROUP STIFFNESS MATRIX

  .16980E+03   .98653E-05  -.16911E+03   .00000E+00  -.71028E+04   .47353E-03
  .98653E-05   .52928E+00  -.29569E-04   .00000E+00   .14193E-02   .41284E+02
 -.16911E+03  -.29569E-04   .15227E+04   .00000E+00  -.11877E+06  -.14193E-02
  .00000E+00   .00000E+00   .00000E+00   .00000E+00   .00000E+00   .00000E+00
 -.71028E+04   .14193E-02  -.11877E+06   .00000E+00   .12919E+08   .94738E-01
  .47353E-03   .41284E+02  -.14193E-02   .00000E+00   .94738E-01   .44904E+04


S(4,4)=0.  PROBLEM WILL BE TREATED AS TWO DIMENSIONAL IN THE X-Z PLANE.

LOAD CASE    1.  NUMBER OF FAILURES =    2.  NUMBER OF PILES IN TENSION =    1.

LOAD CASE    2.  NUMBER OF FAILURES =    1.  NUMBER OF PILES IN TENSION =    1.

********************************************************************************
```

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DZ IN	R RAD
1	-.7241E+00	-.2963E+00	-.3212E-02
2	-.6757E+00	-.2609E+00	-.2899E-02

****

ELASTIC CENTER INFORMATION

ELASTIC CENTER IN PLANE X-Z	X FT	Z FT
	7.74	-11.20

LOAD CASE	MOMENT IN X-Z PLANE
1	.21918E+06
2	.44689E+06

****

PILE FORCES IN LOCAL GEOMETRY

M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO
(F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF	
1	.2	.0	1.5	.0	-31.9	.0	.02	.03	
2	.2	.0	104.6	.0	-29.4	.0	1.41	.35	*
3	-.2	.0	-50.5	.0	30.7	.0	1.03	.18	*

LOAD CASE - 2

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF	
1	.2	.0	8.9	.0	-29.6	.0	.12	.05	
2	.1	.0	101.9	.0	-27.3	.0	1.38	.34	*
3	-.2	.0	-46.1	.0	28.7	.0	.94	.16	

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX	PY	PZ	MX	MY	MZ
	K	K	K	IN-K	IN-K	IN-K
1	-.7	.0	1.4	.0	.0	.0
2	-33.2	.0	99.2	.0	.0	.0
3	-16.1	.0	-47.9	.0	.0	.0

LOAD CASE - 2

PILE	PX	PY	PZ	MX	MY	MZ
	K	K	K	IN-K	IN-K	IN-K
1	-3.0	.0	8.4	.0	.0	.0
2	-32.4	.0	96.6	.0	.0	.0
3	-14.7	.0	-43.6	.0	.0	.0

Attachment 5. Group 7 Output for the Pervious Condition.

```
===============================================================================

                GROUP for Windows, Version 7.0.7

                 Analysis of A Group of Piles
              Subjected to Axial and Lateral Loading

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===============================================================================


This program is licensed to:

k
c

Path to file locations:      C:\KDH\New Orleans\T-walls\Group\
Name of input data file:     10 Example perv.gpd
Name of output file:         10 Example perv.gpo
Name of plot output file:    10 Example perv.gpp
Name of runtime file:        10 Example perv.gpr
Name of output summary file: 10 Example perv.gpt


------------------------------------------------------------------------------
-
                          Time and Date of Analysis
------------------------------------------------------------------------------
-

               Date:  July 27, 2007     Time:  17:44: 4
 PILE GROUP ANALYSIS PROGRAM-GROUP
 PC VERSION 6.0 (C) COPYRIGHT ENSOFT,INC. 2000

 THE PROGRAM WAS COMPILED USING MICROSOFT FORTRAN
 POWERSTATION 4.0 (C) COPYRIGHT MICROSOFT CORPORATION, 1996.



      T-wall Example:  F.S. 10.0, P.S. -1.0, Pervious 50% Unbal. Force on
left pile



                *****     INPUT INFORMATION     *****




     * TABLE C *  LOAD AND CONTROL PARAMETERS


     UNITS--
```

```
    V LOAD,LBS      H LOAD,LBS     MOMENT,LBS-IN

    0.5273E+05      0.3723E+05      0.1032E+07

 GROUP NO. 1


             DISTRIBUTED LOAD CURVE          2 POINTS

                        X,IN       LOAD,LBS/IN
                        0.00         0.210E+02
                      216.00         0.210E+02

 GROUP NO. 2


             DISTRIBUTED LOAD CURVE          2 POINTS

                        X,IN       LOAD,LBS/IN
                        0.00         0.210E+02
                      216.00         0.210E+02

 GROUP NO. 3


             DISTRIBUTED LOAD CURVE          2 POINTS

                        X,IN       LOAD,LBS/IN
                        0.00         0.420E+02
                      216.00         0.420E+02


* THE LOADING IS STATIC *


   KPYOP =  0     (CODE TO GENERATE P-Y CURVES)

   ( KPYOP = 1 IF P-Y YES; = 0 IF P-Y NO; = -1 IF P-Y ONLY )


* CONTROL PARAMETERS *
   TOLERANCE ON CONVERGENCE OF FOUNDATION REACTION      =  0.100E-04 IN
   TOLERANCE ON DETERMINATION OF DEFLECTIONS            =  0.100E-04 IN
   MAX NO OF ITERATIONS ALLOWED FOR FOUNDATION ANALYSIS =     100
   MAXIMUM NO. OF ITERATIONS ALLOWED FOR PILE ANALYSIS  =     100



* TABLE D *   ARRANGEMENT OF PILE GROUPS

 GROUP  CONNECT  NO OF PILE PILE NO  L-S CURVE  P-Y CURVE
   1      PIN         1        1         1           0
   2      PIN         1        1         1           0
   3      PIN         1        1         1           0
```

GROUP	VERT,IN	HOR,IN	SLOPE,IN/IN	GROUND,IN	SPRING,LBS-IN
1	0.0000E+00	-0.1500E+02	0.3218E+00	-0.3600E+02	0.0000E+00
2	0.0000E+00	-0.7500E+02	0.3218E+00	-0.3600E+02	0.0000E+00
3	0.0000E+00	-0.1410E+03	-0.3218E+00	-0.3600E+02	0.0000E+00
4	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00

* TABLE E *   PILE GEOMETRY AND PROPERTIES
PILE TYPE = 1 - DRIVEN PILE
= 2 - DRILLED SHAFT

PILE	SEC	INC	LENGTH, IN	E ,LBS/IN**2	PILE TYPE
1	1	91	0.1092E+04	0.2900E+08	1

PILE	FROM,IN	TO,IN	DIAM,IN	AREA,IN**2	I,IN**4
1	0.0000E+00	0.1092E+04	0.1400E+02	0.2140E+02	0.7290E+03

* THE PILE ABOVE IS OF LINEARLY ELASTIC MATERIAL *

* TABLE F *   AXIAL LOAD VS SETTLEMENT

(THE LOAD-SETTLEMENT CURVE OF SINGLE PILE IS GENERATED INTERNALLY)

NUM OF CURVES 1

CURVE 1   NUM OF POINTS = 19

POINT	AXIAL LOAD,LBS	SETTLEMENT, IN
1	-0.1727E+06	-0.2221E+01
2	-0.1647E+06	-0.1208E+01
3	-0.1607E+06	-0.7010E+00
4	-0.1369E+06	-0.2609E+00
5	-0.1280E+06	-0.1948E+00
6	-0.4099E+05	-0.5077E-01
7	-0.1984E+05	-0.2476E-01
8	-0.3931E+04	-0.4928E-02
9	-0.3931E+03	-0.4928E-03
10	0.0000E+00	0.0000E+00
11	0.7478E+03	0.9072E-03
12	0.4682E+04	0.5805E-02
13	0.2246E+05	0.2777E-01
14	0.4482E+05	0.5521E-01
15	0.1311E+06	0.2001E+00
16	0.1406E+06	0.2675E+00
17	0.1691E+06	0.7159E+00
18	0.1763E+06	0.1228E+01
19	0.1881E+06	0.2248E+01

* TABLE H *   SOIL DATA FOR AUTO P-Y CURVES

```
SOILS INFORMATION

     AT THE GROUND SURFACE          =     -36.00 IN

    6 LAYER(S) OF SOIL

    LAYER  1
    THE SOIL IS A SOFT CLAY
    X AT THE TOP OF THE LAYER          =     -36.00 IN
    X AT THE BOTTOM OF THE LAYER       =     216.00 IN
    MODULUS OF SUBGRADE REACTION       =  0.100E+00 LBS/IN**3

    LAYER  2
    THE SOIL IS A SILT
    X AT THE TOP OF THE LAYER          =     216.00 IN
    X AT THE BOTTOM OF THE LAYER       =     252.00 IN
    MODULUS OF SUBGRADE REACTION       =  0.300E+02 LBS/IN**3

    LAYER  3
    THE SOIL IS A SOFT CLAY
    X AT THE TOP OF THE LAYER          =     252.00 IN
    X AT THE BOTTOM OF THE LAYER       =     720.00 IN
    MODULUS OF SUBGRADE REACTION       =  0.300E+02 LBS/IN**3

    LAYER  4
    THE SOIL IS A STIFF CLAY BELOW THE WATER TABLE
    X AT THE TOP OF THE LAYER          =     720.00 IN
    X AT THE BOTTOM OF THE LAYER       =     973.00 IN
    MODULUS OF SUBGRADE REACTION       =  0.100E+03 LBS/IN**3

    LAYER  5
    THE SOIL IS A SAND
    X AT THE TOP OF THE LAYER          =     973.00 IN
    X AT THE BOTTOM OF THE LAYER       =    1273.00 IN
    MODULUS OF SUBGRADE REACTION       =  0.600E+02 LBS/IN**3

    LAYER  6
    THE SOIL IS A STIFF CLAY BELOW THE WATER TABLE
    X AT THE TOP OF THE LAYER          =    1273.00 IN
    X AT THE BOTTOM OF THE LAYER       =    1344.00 IN
    MODULUS OF SUBGRADE REACTION       =  0.100E+03 LBS/IN**3


    DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                      16 POINTS

                X,IN   WEIGHT,LBS/IN**3
            -36.0000     0.1010E-01
            108.0000     0.1010E-01
            108.0000     0.2170E-01
            216.0000     0.2170E-01
            216.0000     0.3150E-01
            252.0000     0.3150E-01
            252.0000     0.2170E-01
            720.0000     0.2170E-01
            720.0000     0.2750E-01
```

900.0000	0.2750E-01
900.0000	0.3330E-01
972.0000	0.3330E-01
972.0000	0.3440E-01
1273.0000	0.3440E-01
1273.0000	0.3210E-01
1344.0000	0.3210E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
16 POINTS

X IN	C LBS/IN**2	PHI,DEGREES	E50	FMAX LBS/IN**2	TIPMAX LBS/IN**2
-36.00	0.3333E-01	0.000	0.2500E-01	0.1000E+00	0.0000E+00
216.00	0.3333E-01	0.000	0.2500E-01	0.1000E+00	0.0000E+00
216.00	0.1390E+01	15.000	0.2500E-01	0.2400E+01	0.0000E+00
252.00	0.1390E+01	15.000	0.2500E-01	0.2700E+01	0.0000E+00
252.00	0.1390E+01	0.000	0.2500E-01	0.1390E+01	0.0000E+00
408.00	0.1390E+01	0.000	0.2500E-01	0.1390E+01	0.0000E+00
408.00	0.2590E+01	0.000	0.2000E-01	0.2590E+01	0.0000E+00
720.00	0.4100E+01	0.000	0.1000E-01	0.4100E+01	0.0000E+00
720.00	0.4100E+01	0.000	0.1000E-01	0.4100E+01	0.0000E+00
780.00	0.4300E+01	0.000	0.1000E-01	0.4300E+01	0.0000E+00
780.00	0.5500E+01	0.000	0.1000E-01	0.5500E+01	0.0000E+00
973.00	0.5500E+01	0.000	0.1000E-01	0.5500E+01	0.0000E+00
973.00	0.0000E+00	30.000	0.0000E+00	0.1300E+02	0.0000E+00
1273.00	0.0000E+00	30.000	0.0000E+00	0.1400E+02	0.0000E+00
1273.00	0.6800E+01	0.000	0.1000E-01	0.6800E+01	0.0000E+00
1344.00	0.6800E+01	0.000	0.1000E-01	0.6800E+01	0.0000E+00

REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS

GROUP NO	P-FACTOR	Y-FACTOR
1	1.00	1.00
2	0.83	1.00
3	0.87	1.00

T-wall Example: F.S. 10.0, P.S. -1.0, Pervious 50% Unbal. Force on left pile

***** COMPUTATION RESULTS *****

VERT. LOAD, LBS	HORI. LOAD, LBS	MOMENT,IN-LBS
0.5273E+05	0.3723E+05	0.1032E+07

DISPLACEMENT OF GROUPED PILE FOUNDATION

```
           VERTICAL,IN    HORIZONTAL,IN    ROTATION,RAD

          -0.2048E+00     0.5260E+00       0.2313E-02


      NUMBER OF ITERATIONS =   4


   * TABLE I *   COMPUTATION ON INDIVIDUAL PILE

     * PILE GROUP *  1

 PILE TOP DISPLACEMENTS AND REACTIONS

       THE GLOBAL STRUCTURE COORDINATE SYSTEM
       --------------------------------------

 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

-0.170E+00  0.526E+00 -.192E-02 0.421E+04 0.307E+02 0.000E+00   0.187E+03

       THE LOCAL MEMBER COORDINATE SYSTEM
       ------------------------------------

 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

 0.496E-02  0.553E+00 -.192E-02 0.400E+04-0.130E+04 0.000E+00   0.187E+03

          LATERALLY LOADED PILE
```

X	DEFLECTION	MOMENT	SHEAR	SOIL REACTION	TOTAL STRESS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS	LBS/IN	LBS/IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
0.00	0.553E+00	0.000E+00	-0.106E+04	0.180E+01	0.187E+03	0.211E+11
12.00	0.530E+00	0.126E+05	-0.944E+03	0.178E+01	0.308E+03	0.211E+11
24.00	0.507E+00	0.225E+05	-0.714E+03	0.175E+01	0.403E+03	0.211E+11
36.00	0.483E+00	0.296E+05	-0.482E+03	0.172E+01	0.471E+03	0.211E+11
48.00	0.460E+00	0.339E+05	-0.251E+03	0.169E+01	0.512E+03	0.211E+11
60.00	0.436E+00	0.354E+05	-0.191E+02	0.166E+01	0.527E+03	0.211E+11
72.00	0.412E+00	0.341E+05	0.213E+03	0.163E+01	0.515E+03	0.211E+11
84.00	0.388E+00	0.301E+05	0.446E+03	0.160E+01	0.476E+03	0.211E+11
96.00	0.364E+00	0.233E+05	0.679E+03	0.157E+01	0.410E+03	0.211E+11

108.00	0.339E+00	0.136E+05	0.912E+03	0.153E+01	0.318E+03	0.211E+11
120.00	0.315E+00	0.116E+04	0.115E+04	0.149E+01	0.198E+03	0.211E+11
132.00	0.290E+00	-0.141E+05	0.138E+04	0.145E+01	0.322E+03	0.211E+11
144.00	0.265E+00	-0.322E+05	0.162E+04	0.141E+01	0.496E+03	0.211E+11
156.00	0.241E+00	-0.530E+05	0.185E+04	0.137E+01	0.696E+03	0.211E+11
168.00	0.217E+00	-0.768E+05	0.209E+04	0.132E+01	0.924E+03	0.211E+11
180.00	0.194E+00	-0.103E+06	0.232E+04	0.127E+01	0.118E+04	0.211E+11
192.00	0.171E+00	-0.133E+06	0.256E+04	0.122E+01	0.146E+04	0.211E+11
204.00	0.149E+00	-0.165E+06	0.280E+04	0.116E+01	0.177E+04	0.211E+11
216.00	0.128E+00	-0.200E+06	0.270E+04	0.572E+02	0.211E+04	0.211E+11
228.00	0.109E+00	-0.230E+06	0.196E+04	0.878E+02	0.239E+04	0.211E+11
240.00	0.912E-01	-0.247E+06	0.791E+03	0.106E+03	0.256E+04	0.211E+11
252.00	0.751E-01	-0.249E+06	0.356E+02	0.196E+02	0.258E+04	0.211E+11
264.00	0.607E-01	-0.248E+06	-0.205E+03	0.206E+02	0.257E+04	0.211E+11
276.00	0.479E-01	-0.244E+06	-0.456E+03	0.212E+02	0.253E+04	0.211E+11
288.00	0.369E-01	-0.237E+06	-0.712E+03	0.214E+02	0.246E+04	0.211E+11
300.00	0.275E-01	-0.227E+06	-0.967E+03	0.212E+02	0.237E+04	0.211E+11
312.00	0.196E-01	-0.214E+06	-0.122E+04	0.206E+02	0.224E+04	0.211E+11
324.00	0.131E-01	-0.198E+06	-0.146E+04	0.194E+02	0.209E+04	0.211E+11
336.00	0.805E-02	-0.179E+06	-0.168E+04	0.177E+02	0.191E+04	0.211E+11
348.00	0.418E-02	-0.158E+06	-0.188E+04	0.147E+02	0.170E+04	0.211E+11
360.00	0.139E-02	-0.134E+06	-0.202E+04	0.102E+02	0.148E+04	0.211E+11
372.00	-0.487E-03	-0.109E+06	-0.204E+04	-0.728E+01	0.123E+04	0.211E+11
384.00	-0.162E-02	-0.852E+05	-0.193E+04	-0.108E+02	0.100E+04	0.211E+11
396.00	-0.217E-02	-0.627E+05	-0.180E+04	-0.119E+02	0.789E+03	0.211E+11
408.00	-0.230E-02	-0.420E+05	-0.159E+04	-0.234E+02	0.590E+03	0.211E+11
420.00	-0.214E-02	-0.247E+05	-0.130E+04	-0.244E+02	0.424E+03	0.211E+11
432.00	-0.181E-02	-0.108E+05	-0.101E+04	-0.238E+02	0.291E+03	0.211E+11
444.00	-0.141E-02	-0.421E+03	-0.733E+03	-0.225E+02	0.191E+03	0.211E+11
456.00	-0.101E-02	0.676E+04	-0.474E+03	-0.207E+02	0.252E+03	0.211E+11
468.00	-0.647E-03	0.110E+05	-0.240E+03	-0.183E+02	0.292E+03	0.211E+11
480.00	-0.363E-03	0.125E+05	-0.368E+02	-0.155E+02	0.307E+03	0.211E+11
492.00	-0.165E-03	0.118E+05	0.130E+03	-0.123E+02	0.301E+03	0.211E+11
504.00	-0.465E-04	0.940E+04	0.254E+03	-0.832E+01	0.277E+03	0.211E+11
516.00	0.748E-05	0.575E+04	0.277E+03	0.452E+01	0.242E+03	0.211E+11
528.00	0.223E-04	0.276E+04	0.209E+03	0.681E+01	0.213E+03	0.211E+11
540.00	0.184E-04	0.743E+03	0.128E+03	0.658E+01	0.194E+03	0.211E+11
552.00	0.943E-05	-0.323E+03	0.563E+02	0.543E+01	0.190E+03	0.211E+11
564.00	0.264E-05	-0.608E+03	0.160E+01	0.368E+01	0.193E+03	0.211E+11
576.00	0.200E-08	-0.362E+03	-0.232E+02	0.448E+00	0.190E+03	0.211E+11
588.00	-0.177E-06	-0.514E+02	-0.161E+02	-0.163E+01	0.187E+03	0.211E+11
600.00	-0.493E-08	0.245E+02	-0.217E+01	-0.692E+00	0.187E+03	0.211E+11
612.00	0.875E-10	0.750E+00	0.102E+01	0.159E+00	0.187E+03	0.211E+11
624.00	0.841E-14	-0.128E-01	0.312E-01	0.538E-02	0.187E+03	0.211E+11
636.00	-0.136E-15	-0.127E-05	-0.535E-03	-0.891E-04	0.187E+03	0.211E+11
648.00	-0.135E-19	0.199E-07	-0.531E-07	-0.913E-08	0.187E+03	0.211E+11
660.00	0.200E-21	0.205E-11	0.830E-09	0.138E-09	0.187E+03	0.211E+11
672.00	0.206E-25	-0.293E-13	0.853E-13	0.146E-13	0.187E+03	0.211E+11
684.00	-0.279E-27	-0.310E-17	-0.122E-14	-0.204E-15	0.187E+03	0.211E+11
696.00	-0.296E-31	0.410E-19	-0.129E-18	-0.221E-19	0.187E+03	0.211E+11
708.00	0.370E-33	0.233E-23	0.171E-20	0.285E-21	0.187E+03	0.211E+11
720.00	0.144E-31	0.149E-23	0.632E-25	0.109E-26	0.187E+03	0.211E+11
732.00	0.183E-31	0.815E-24	0.482E-25	0.140E-26	0.187E+03	0.211E+11
744.00	0.166E-31	0.338E-24	0.320E-25	0.130E-26	0.187E+03	0.211E+11
756.00	0.126E-31	0.470E-25	0.182E-25	0.100E-26	0.187E+03	0.211E+11
768.00	0.833E-32	-0.995E-25	0.819E-26	0.670E-27	0.187E+03	0.211E+11
780.00	0.471E-32	-0.150E-24	0.186E-26	0.385E-27	0.187E+03	0.211E+11

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     792.00  0.211E-32 -0.144E-24 -0.150E-26  0.175E-27  0.187E+03  0.211E+11
     804.00  0.493E-33 -0.114E-24 -0.279E-26  0.414E-28  0.187E+03  0.211E+11
     816.00 -0.351E-33 -0.772E-25 -0.286E-26 -0.299E-28  0.187E+03  0.211E+11
     828.00 -0.670E-33 -0.450E-25 -0.234E-26 -0.579E-28  0.187E+03  0.211E+11
     840.00 -0.682E-33 -0.211E-25 -0.163E-26 -0.597E-28  0.187E+03  0.211E+11
     852.00 -0.551E-33 -0.584E-26 -0.979E-27 -0.489E-28  0.187E+03  0.211E+11
     864.00 -0.379E-33  0.239E-26 -0.481E-27 -0.341E-28  0.187E+03  0.211E+11
     876.00 -0.224E-33  0.570E-26 -0.154E-27 -0.204E-28  0.187E+03  0.211E+11
     888.00 -0.108E-33  0.608E-26  0.290E-28 -0.998E-29  0.187E+03  0.211E+11
     900.00 -0.332E-34  0.501E-26  0.107E-27 -0.311E-29  0.187E+03  0.211E+11
     912.00  0.749E-35  0.350E-26  0.122E-27  0.710E-30  0.187E+03  0.211E+11
     924.00  0.244E-34  0.208E-26  0.104E-27  0.234E-29  0.187E+03  0.211E+11
     936.00  0.270E-34  0.101E-26  0.738E-28  0.263E-29  0.187E+03  0.211E+11
     948.00  0.228E-34  0.314E-27  0.446E-28  0.224E-29  0.187E+03  0.211E+11
     960.00  0.164E-34 -0.599E-28  0.213E-28  0.164E-29  0.187E+03  0.211E+11
     972.00  0.105E-34 -0.197E-27  0.512E-29  0.106E-29  0.187E+03  0.211E+11
     984.00  0.590E-35 -0.183E-27 -0.169E-29  0.753E-31  0.187E+03  0.211E+11
     996.00  0.255E-35 -0.157E-27 -0.234E-29  0.343E-31  0.187E+03  0.211E+11
    1008.00  0.260E-36 -0.126E-27 -0.257E-29  0.368E-32  0.187E+03  0.211E+11
    1020.00 -0.117E-35 -0.953E-28 -0.249E-29 -0.174E-31  0.187E+03  0.211E+11
    1032.00 -0.194E-35 -0.666E-28 -0.220E-29 -0.304E-31  0.187E+03  0.211E+11
    1044.00 -0.226E-35 -0.424E-28 -0.180E-29 -0.370E-31  0.187E+03  0.211E+11
    1056.00 -0.230E-35 -0.234E-28 -0.134E-29 -0.392E-31  0.187E+03  0.211E+11
    1068.00 -0.217E-35 -0.102E-28 -0.875E-30 -0.387E-31  0.187E+03  0.211E+11
    1080.00 -0.198E-35 -0.245E-29 -0.423E-30 -0.366E-31  0.187E+03  0.211E+11
    1092.00 -0.177E-35  0.000E+00 -0.155E-44 -0.340E-31  0.187E+03  0.211E+11

     NUMBER OF ITERATIONS IN LLP =    14


       * PILE GROUP *  2


 PILE TOP DISPLACEMENTS AND REACTIONS


       THE GLOBAL STRUCTURE COORDINATE SYSTEM
       --------------------------------------

 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

-0.314E-01  0.526E+00 -.164E-02 0.890E+05 0.279E+05 0.000E+00   0.436E+04


       THE LOCAL MEMBER COORDINATE SYSTEM
       ------------------------------------

 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

 0.137E+00  0.509E+00 -.164E-02 0.933E+05-0.165E+04 0.000E+00   0.436E+04


          LATERALLY LOADED PILE
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X	DEFLECTION	MOMENT	SHEAR	SOIL REACTION	TOTAL STRESS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS	LBS/IN	LBS/IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
0.00	0.509E+00	0.000E+00	-0.141E+04	0.146E+01	0.436E+04	0.211E+11
12.00	0.489E+00	0.151E+05	-0.129E+04	0.144E+01	0.450E+04	0.211E+11
24.00	0.470E+00	0.273E+05	-0.106E+04	0.142E+01	0.462E+04	0.211E+11
36.00	0.450E+00	0.367E+05	-0.820E+03	0.140E+01	0.471E+04	0.211E+11
48.00	0.429E+00	0.432E+05	-0.585E+03	0.138E+01	0.477E+04	0.211E+11
60.00	0.409E+00	0.469E+05	-0.349E+03	0.136E+01	0.481E+04	0.211E+11
72.00	0.388E+00	0.478E+05	-0.114E+03	0.134E+01	0.482E+04	0.211E+11
84.00	0.367E+00	0.457E+05	0.123E+03	0.131E+01	0.480E+04	0.211E+11
96.00	0.346E+00	0.408E+05	0.359E+03	0.129E+01	0.475E+04	0.211E+11
108.00	0.324E+00	0.331E+05	0.596E+03	0.126E+01	0.468E+04	0.211E+11
120.00	0.302E+00	0.225E+05	0.833E+03	0.123E+01	0.457E+04	0.211E+11
132.00	0.280E+00	0.900E+04	0.107E+04	0.120E+01	0.444E+04	0.211E+11
144.00	0.258E+00	-0.734E+04	0.131E+04	0.117E+01	0.443E+04	0.211E+11
156.00	0.236E+00	-0.265E+05	0.155E+04	0.113E+01	0.461E+04	0.211E+11
168.00	0.214E+00	-0.486E+05	0.178E+04	0.110E+01	0.482E+04	0.211E+11
180.00	0.192E+00	-0.734E+05	0.202E+04	0.106E+01	0.506E+04	0.211E+11
192.00	0.171E+00	-0.101E+06	0.226E+04	0.102E+01	0.533E+04	0.211E+11
204.00	0.150E+00	-0.132E+06	0.250E+04	0.974E+00	0.562E+04	0.211E+11
216.00	0.131E+00	-0.165E+06	0.246E+04	0.487E+02	0.594E+04	0.211E+11
228.00	0.113E+00	-0.194E+06	0.184E+04	0.757E+02	0.622E+04	0.211E+11
240.00	0.955E-01	-0.212E+06	0.826E+03	0.929E+02	0.640E+04	0.211E+11
252.00	0.799E-01	-0.217E+06	0.168E+03	0.167E+02	0.644E+04	0.211E+11
264.00	0.657E-01	-0.219E+06	-0.379E+02	0.176E+02	0.646E+04	0.211E+11
276.00	0.531E-01	-0.219E+06	-0.253E+03	0.183E+02	0.646E+04	0.211E+11
288.00	0.419E-01	-0.215E+06	-0.475E+03	0.186E+02	0.642E+04	0.211E+11
300.00	0.322E-01	-0.209E+06	-0.699E+03	0.187E+02	0.637E+04	0.211E+11
312.00	0.239E-01	-0.200E+06	-0.921E+03	0.184E+02	0.628E+04	0.211E+11
324.00	0.170E-01	-0.188E+06	-0.114E+04	0.177E+02	0.617E+04	0.211E+11
336.00	0.114E-01	-0.174E+06	-0.134E+04	0.166E+02	0.603E+04	0.211E+11
348.00	0.697E-02	-0.157E+06	-0.153E+04	0.146E+02	0.587E+04	0.211E+11
360.00	0.361E-02	-0.138E+06	-0.169E+04	0.117E+02	0.568E+04	0.211E+11
372.00	0.118E-02	-0.117E+06	-0.181E+04	0.803E+01	0.548E+04	0.211E+11
384.00	-0.453E-03	-0.951E+05	-0.182E+04	-0.594E+01	0.527E+04	0.211E+11
396.00	-0.144E-02	-0.738E+05	-0.173E+04	-0.865E+01	0.507E+04	0.211E+11
408.00	-0.192E-02	-0.537E+05	-0.157E+04	-0.184E+02	0.487E+04	0.211E+11
420.00	-0.203E-02	-0.362E+05	-0.134E+04	-0.200E+02	0.471E+04	0.211E+11
432.00	-0.190E-02	-0.216E+05	-0.110E+04	-0.201E+02	0.456E+04	0.211E+11
444.00	-0.162E-02	-0.983E+04	-0.859E+03	-0.196E+02	0.445E+04	0.211E+11
456.00	-0.127E-02	-0.910E+03	-0.629E+03	-0.186E+02	0.437E+04	0.211E+11
468.00	-0.918E-03	0.533E+04	-0.414E+03	-0.172E+02	0.441E+04	0.211E+11
480.00	-0.602E-03	0.909E+04	-0.219E+03	-0.153E+02	0.444E+04	0.211E+11
492.00	-0.347E-03	0.106E+05	-0.480E+02	-0.131E+02	0.446E+04	0.211E+11
504.00	-0.165E-03	0.103E+05	0.941E+02	-0.105E+02	0.446E+04	0.211E+11
516.00	-0.533E-04	0.841E+04	0.202E+03	-0.745E+01	0.444E+04	0.211E+11
528.00	0.143E-05	0.545E+04	0.234E+03	0.208E+01	0.441E+04	0.211E+11
540.00	0.191E-04	0.279E+04	0.189E+03	0.554E+01	0.438E+04	0.211E+11
552.00	0.177E-04	0.924E+03	0.122E+03	0.557E+01	0.437E+04	0.211E+11
564.00	0.100E-04	-0.140E+03	0.601E+02	0.475E+01	0.436E+04	0.211E+11
576.00	0.332E-05	-0.519E+03	0.111E+02	0.341E+01	0.436E+04	0.211E+11

588.00	0.151E-06	-0.408E+03	-0.181E+02	0.145E+01	0.436E+04	0.211E+11
600.00	-0.238E-06	-0.865E+02	-0.180E+02	-0.147E+01	0.436E+04	0.211E+11
612.00	-0.375E-07	0.239E+02	-0.383E+01	-0.893E+00	0.436E+04	0.211E+11
624.00	0.792E-10	0.552E+01	0.995E+00	0.890E-01	0.436E+04	0.211E+11
636.00	0.183E-11	-0.111E-01	0.230E+00	0.385E-01	0.436E+04	0.211E+11
648.00	-0.109E-15	-0.268E-03	-0.462E-03	-0.733E-04	0.436E+04	0.211E+11
660.00	-0.269E-17	0.152E-07	-0.112E-04	-0.186E-05	0.436E+04	0.211E+11
672.00	0.141E-21	0.395E-09	0.632E-09	0.999E-10	0.436E+04	0.211E+11
684.00	0.376E-23	-0.195E-13	0.165E-10	0.274E-11	0.436E+04	0.211E+11
696.00	-0.171E-27	-0.553E-15	-0.814E-15	-0.128E-15	0.436E+04	0.211E+11
708.00	-0.499E-29	0.127E-19	-0.230E-16	-0.384E-17	0.436E+04	0.211E+11
720.00	0.743E-28	0.824E-20	0.342E-21	0.562E-23	0.436E+04	0.211E+11
732.00	0.975E-28	0.455E-20	0.263E-21	0.749E-23	0.436E+04	0.211E+11
744.00	0.898E-28	0.193E-20	0.176E-21	0.700E-23	0.436E+04	0.211E+11
756.00	0.689E-28	0.321E-21	0.101E-21	0.546E-23	0.436E+04	0.211E+11
768.00	0.458E-28	-0.502E-21	0.463E-22	0.368E-23	0.436E+04	0.211E+11
780.00	0.262E-28	-0.794E-21	0.114E-22	0.214E-23	0.436E+04	0.211E+11
792.00	0.119E-28	-0.778E-21	-0.737E-23	0.988E-24	0.436E+04	0.211E+11
804.00	0.300E-29	-0.620E-21	-0.148E-22	0.252E-24	0.436E+04	0.211E+11
816.00	-0.172E-29	-0.424E-21	-0.154E-22	-0.147E-24	0.436E+04	0.211E+11
828.00	-0.355E-29	-0.250E-21	-0.127E-22	-0.307E-24	0.436E+04	0.211E+11
840.00	-0.368E-29	-0.119E-21	-0.895E-23	-0.322E-24	0.436E+04	0.211E+11
852.00	-0.300E-29	-0.347E-22	-0.542E-23	-0.266E-24	0.436E+04	0.211E+11
864.00	-0.208E-29	0.112E-22	-0.270E-23	-0.187E-24	0.436E+04	0.211E+11
876.00	-0.124E-29	0.301E-22	-0.892E-24	-0.113E-24	0.436E+04	0.211E+11
888.00	-0.606E-30	0.328E-22	0.123E-24	-0.560E-25	0.436E+04	0.211E+11
900.00	-0.193E-30	0.273E-22	0.568E-24	-0.181E-25	0.436E+04	0.211E+11
912.00	0.331E-31	0.192E-22	0.657E-24	0.314E-26	0.436E+04	0.211E+11
924.00	0.129E-30	0.115E-22	0.564E-24	0.124E-25	0.436E+04	0.211E+11
936.00	0.146E-30	0.566E-23	0.405E-24	0.142E-25	0.436E+04	0.211E+11
948.00	0.124E-30	0.182E-23	0.247E-24	0.122E-25	0.436E+04	0.211E+11
960.00	0.906E-31	-0.260E-24	0.119E-24	0.902E-26	0.436E+04	0.211E+11
972.00	0.585E-31	-0.104E-23	0.295E-25	0.589E-26	0.436E+04	0.211E+11
984.00	0.335E-31	-0.973E-24	-0.842E-26	0.427E-27	0.436E+04	0.211E+11
996.00	0.151E-31	-0.843E-24	-0.122E-25	0.203E-27	0.436E+04	0.211E+11
1008.00	0.244E-32	-0.683E-24	-0.136E-25	0.346E-28	0.436E+04	0.211E+11
1020.00	-0.554E-32	-0.518E-24	-0.133E-25	-0.826E-28	0.436E+04	0.211E+11
1032.00	-0.999E-32	-0.365E-24	-0.119E-25	-0.156E-27	0.436E+04	0.211E+11
1044.00	-0.120E-31	-0.233E-24	-0.979E-26	-0.195E-27	0.436E+04	0.211E+11
1056.00	-0.123E-31	-0.130E-24	-0.736E-26	-0.210E-27	0.436E+04	0.211E+11
1068.00	-0.118E-31	-0.568E-25	-0.483E-26	-0.210E-27	0.436E+04	0.211E+11
1080.00	-0.109E-31	-0.138E-25	-0.236E-26	-0.202E-27	0.436E+04	0.211E+11
1092.00	-0.993E-32	0.000E+00	-0.143E-40	-0.191E-27	0.436E+04	0.211E+11

NUMBER OF ITERATIONS IN LLP = 13

* PILE GROUP * 3

PILE TOP DISPLACEMENTS AND REACTIONS

THE GLOBAL STRUCTURE COORDINATE SYSTEM

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 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

 0.121E+00  0.526E+00 -.997E-03-0.405E+05 0.927E+04 0.000E+00   0.193E+04


       THE LOCAL MEMBER COORDINATE SYSTEM
       ------------------------------------

 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

-0.513E-01  0.537E+00 -.997E-03-0.413E+05-0.400E+04 0.000E+00   0.193E+04


          LATERALLY LOADED PILE
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X	DEFLECTION	MOMENT	SHEAR	SOIL REACTION	TOTAL STRESS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS	LBS/IN	LBS/IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
0.00	0.537E+00	0.000E+00	-0.351E+04	0.154E+01	0.193E+04	0.211E+11
12.00	0.525E+00	0.426E+05	-0.326E+04	0.153E+01	0.234E+04	0.211E+11
24.00	0.513E+00	0.793E+05	-0.278E+04	0.152E+01	0.269E+04	0.211E+11
36.00	0.500E+00	0.110E+06	-0.229E+04	0.151E+01	0.299E+04	0.211E+11
48.00	0.487E+00	0.135E+06	-0.181E+04	0.149E+01	0.323E+04	0.211E+11
60.00	0.472E+00	0.155E+06	-0.132E+04	0.148E+01	0.342E+04	0.211E+11
72.00	0.457E+00	0.168E+06	-0.834E+03	0.146E+01	0.355E+04	0.211E+11
84.00	0.440E+00	0.176E+06	-0.347E+03	0.144E+01	0.362E+04	0.211E+11
96.00	0.422E+00	0.178E+06	0.140E+03	0.143E+01	0.364E+04	0.211E+11
108.00	0.403E+00	0.174E+06	0.627E+03	0.140E+01	0.360E+04	0.211E+11
120.00	0.383E+00	0.165E+06	0.111E+04	0.138E+01	0.351E+04	0.211E+11
132.00	0.362E+00	0.149E+06	0.160E+04	0.135E+01	0.336E+04	0.211E+11
144.00	0.339E+00	0.128E+06	0.209E+04	0.132E+01	0.316E+04	0.211E+11
156.00	0.316E+00	0.101E+06	0.258E+04	0.129E+01	0.290E+04	0.211E+11
168.00	0.292E+00	0.681E+05	0.307E+04	0.126E+01	0.258E+04	0.211E+11
180.00	0.267E+00	0.294E+05	0.356E+04	0.122E+01	0.221E+04	0.211E+11
192.00	0.243E+00	-0.152E+05	0.405E+04	0.119E+01	0.208E+04	0.211E+11
204.00	0.218E+00	-0.656E+05	0.454E+04	0.114E+01	0.256E+04	0.211E+11
216.00	0.194E+00	-0.122E+06	0.458E+04	0.749E+02	0.310E+04	0.211E+11
228.00	0.171E+00	-0.174E+06	0.367E+04	0.119E+03	0.360E+04	0.211E+11
240.00	0.149E+00	-0.208E+06	0.206E+04	0.150E+03	0.393E+04	0.211E+11
252.00	0.128E+00	-0.221E+06	0.103E+04	0.203E+02	0.406E+04	0.211E+11
264.00	0.109E+00	-0.231E+06	0.781E+03	0.217E+02	0.415E+04	0.211E+11
276.00	0.915E-01	-0.238E+06	0.515E+03	0.227E+02	0.422E+04	0.211E+11
288.00	0.755E-01	-0.242E+06	0.237E+03	0.235E+02	0.426E+04	0.211E+11
300.00	0.612E-01	-0.243E+06	-0.479E+02	0.240E+02	0.426E+04	0.211E+11
312.00	0.486E-01	-0.240E+06	-0.336E+03	0.241E+02	0.424E+04	0.211E+11
324.00	0.376E-01	-0.234E+06	-0.624E+03	0.239E+02	0.418E+04	0.211E+11
336.00	0.282E-01	-0.224E+06	-0.907E+03	0.233E+02	0.408E+04	0.211E+11
348.00	0.203E-01	-0.211E+06	-0.118E+04	0.216E+02	0.396E+04	0.211E+11
360.00	0.139E-01	-0.195E+06	-0.142E+04	0.190E+02	0.381E+04	0.211E+11

372.00	0.875E-02	-0.177E+06	-0.163E+04	0.163E+02	0.363E+04	0.211E+11
384.00	0.484E-02	-0.156E+06	-0.181E+04	0.134E+02	0.343E+04	0.211E+11
396.00	0.200E-02	-0.133E+06	-0.195E+04	0.995E+01	0.321E+04	0.211E+11
408.00	0.559E-04	-0.109E+06	-0.204E+04	0.539E+01	0.298E+04	0.211E+11
420.00	-0.114E-02	-0.840E+05	-0.197E+04	-0.172E+02	0.274E+04	0.211E+11
432.00	-0.177E-02	-0.615E+05	-0.175E+04	-0.204E+02	0.252E+04	0.211E+11
444.00	-0.198E-02	-0.420E+05	-0.149E+04	-0.218E+02	0.233E+04	0.211E+11
456.00	-0.190E-02	-0.257E+05	-0.123E+04	-0.221E+02	0.218E+04	0.211E+11
468.00	-0.164E-02	-0.125E+05	-0.968E+03	-0.216E+02	0.205E+04	0.211E+11
480.00	-0.130E-02	-0.246E+04	-0.715E+03	-0.206E+02	0.195E+04	0.211E+11
492.00	-0.949E-03	0.462E+04	-0.477E+03	-0.190E+02	0.198E+04	0.211E+11
504.00	-0.625E-03	0.896E+04	-0.261E+03	-0.170E+02	0.202E+04	0.211E+11
516.00	-0.362E-03	0.109E+05	-0.714E+02	-0.146E+02	0.204E+04	0.211E+11
528.00	-0.173E-03	0.107E+05	0.862E+02	-0.117E+02	0.203E+04	0.211E+11
540.00	-0.561E-04	0.877E+04	0.206E+03	-0.829E+01	0.202E+04	0.211E+11
552.00	0.741E-06	0.570E+04	0.246E+03	0.168E+01	0.199E+04	0.211E+11
564.00	0.187E-04	0.287E+04	0.200E+03	0.602E+01	0.196E+04	0.211E+11
576.00	0.172E-04	0.910E+03	0.127E+03	0.603E+01	0.194E+04	0.211E+11
588.00	0.941E-05	-0.181E+03	0.604E+02	0.510E+01	0.193E+04	0.211E+11
600.00	0.288E-05	-0.539E+03	0.847E+01	0.357E+01	0.194E+04	0.211E+11
612.00	0.284E-07	-0.384E+03	-0.198E+02	0.115E+01	0.193E+04	0.211E+11
624.00	-0.207E-06	-0.632E+02	-0.171E+02	-0.160E+01	0.193E+04	0.211E+11
636.00	-0.122E-07	0.268E+02	-0.271E+01	-0.799E+00	0.193E+04	0.211E+11
648.00	0.123E-09	0.183E+01	0.112E+01	0.161E+00	0.193E+04	0.211E+11
660.00	0.251E-13	-0.180E-01	0.763E-01	0.130E-01	0.193E+04	0.211E+11
672.00	-0.176E-15	-0.374E-05	-0.750E-03	-0.125E-03	0.193E+04	0.211E+11
684.00	-0.361E-19	0.258E-07	-0.156E-06	-0.264E-07	0.193E+04	0.211E+11
696.00	0.239E-21	0.538E-11	0.108E-08	0.179E-09	0.193E+04	0.211E+11
708.00	0.488E-25	-0.186E-13	0.225E-12	0.375E-13	0.193E+04	0.211E+11
720.00	-0.113E-21	-0.119E-13	-0.501E-15	-0.853E-17	0.193E+04	0.211E+11
732.00	-0.145E-21	-0.652E-14	-0.383E-15	-0.111E-16	0.193E+04	0.211E+11
744.00	-0.132E-21	-0.272E-14	-0.255E-15	-0.103E-16	0.193E+04	0.211E+11
756.00	-0.100E-21	-0.400E-15	-0.146E-15	-0.796E-17	0.193E+04	0.211E+11
768.00	-0.664E-22	0.774E-15	-0.660E-16	-0.534E-17	0.193E+04	0.211E+11
780.00	-0.377E-22	0.118E-14	-0.154E-16	-0.308E-17	0.193E+04	0.211E+11
792.00	-0.170E-22	0.114E-14	0.115E-16	-0.141E-17	0.193E+04	0.211E+11
804.00	-0.408E-23	0.903E-15	0.220E-16	-0.343E-18	0.193E+04	0.211E+11
816.00	0.269E-23	0.615E-15	0.226E-16	0.229E-18	0.193E+04	0.211E+11
828.00	0.526E-23	0.359E-15	0.185E-16	0.455E-18	0.193E+04	0.211E+11
840.00	0.539E-23	0.170E-15	0.130E-16	0.473E-18	0.193E+04	0.211E+11
852.00	0.437E-23	0.478E-16	0.782E-17	0.388E-18	0.193E+04	0.211E+11
864.00	0.302E-23	-0.180E-16	0.386E-17	0.272E-18	0.193E+04	0.211E+11
876.00	0.179E-23	-0.448E-16	0.125E-17	0.163E-18	0.193E+04	0.211E+11
888.00	0.867E-24	-0.480E-16	-0.207E-18	0.801E-19	0.193E+04	0.211E+11
900.00	0.271E-24	-0.397E-16	-0.840E-18	0.254E-19	0.193E+04	0.211E+11
912.00	-0.544E-25	-0.278E-16	-0.961E-18	-0.516E-20	0.193E+04	0.211E+11
924.00	-0.190E-24	-0.166E-16	-0.821E-18	-0.183E-19	0.193E+04	0.211E+11
936.00	-0.213E-24	-0.810E-17	-0.587E-18	-0.207E-19	0.193E+04	0.211E+11
948.00	-0.181E-24	-0.255E-17	-0.356E-18	-0.178E-19	0.193E+04	0.211E+11
960.00	-0.131E-24	0.433E-18	-0.171E-18	-0.130E-19	0.193E+04	0.211E+11
972.00	-0.840E-25	0.154E-17	-0.417E-19	-0.847E-20	0.193E+04	0.211E+11
984.00	-0.477E-25	0.143E-17	0.127E-19	-0.608E-21	0.193E+04	0.211E+11
996.00	-0.211E-25	0.123E-17	0.181E-19	-0.284E-21	0.193E+04	0.211E+11
1008.00	-0.284E-26	0.996E-18	0.200E-19	-0.403E-22	0.193E+04	0.211E+11
1020.00	0.859E-26	0.753E-18	0.195E-19	0.128E-21	0.193E+04	0.211E+11
1032.00	0.149E-25	0.528E-18	0.173E-19	0.233E-21	0.193E+04	0.211E+11
1044.00	0.176E-25	0.336E-18	0.142E-19	0.288E-21	0.193E+04	0.211E+11

1056.00	0.180E-25	0.187E-18	0.106E-19	0.307E-21	0.193E+04	0.211E+11
1068.00	0.171E-25	0.811E-19	0.696E-20	0.305E-21	0.193E+04	0.211E+11
1080.00	0.157E-25	0.196E-19	0.339E-20	0.291E-21	0.193E+04	0.211E+11
1092.00	0.142E-25	0.211E-33	-0.227E-34	0.273E-21	0.193E+04	0.211E+11

NUMBER OF ITERATIONS IN LLP = 14

Attachment 6 – Spencer's method analysis with piles as reinforcement (Figure 20).

```
HEADING
    T-Wall Deep Seated Analysis
    Analysis without piles

PROFILE LINES
         1     1 Layer 3 (CH) - Floodside
                  .00      -2.00
               141.00      -2.00
               155.00      -2.00

         2     1 Layer 3 (CH) - Landside
               157.00      -2.00
               375.00      -2.00

         3     2 Compacted Fill - FS
               141.00      -2.00
               145.50       -.50

         4     2 Compacted Fill - LS
               158.50       1.00
               167.00       1.00
               176.00      -2.00

         5     3 T-Wall
               145.50      -5.00
               145.50      -2.50
               155.00      -2.50
               155.00      -2.00
               155.00      12.30
               157.00      12.30
               157.00       1.00
               157.00      -2.00
               157.00      -2.50
               158.50      -2.50
               158.50      -5.00

         6     1 Layer 3 (CH) - Under Wall
               145.50      -5.00
               158.50      -5.00

         7     4 Layer 4 (CH)
                  .00     -14.00
               375.00     -14.00

         8     5 Layer 5 (ML)
                  .00     -23.00
               375.00     -23.00

         9     6 Layer 6 (CH)
                  .00     -26.00
               375.00     -26.00

        10     7 Layer 7 (CH)
                  .00     -31.00
```

```
                375.00    -31.00

          11     8 Layer 8 (CH)
                   .00    -39.00
                375.00    -39.00

          12     9 Layer 9 (CH)
                   .00    -65.00
                375.00    -65.00

          13    10 Compacted Fill - Above T Wall Base   FS
                145.50      -.50
                150.00      1.00
                155.00      1.00

          14    10 Compacted Fill - Above T Wall Base   LS
                157.00      1.00
                158.50      1.00


MATERIAL PROPERTIES
     1 Layer 3 (CH)
          80.00 Unit Weight
          Conventional Shear
              120.00       .00
          No Pore Pressure
     2 Compacted Fill
          110.00 Unit Weight
          Conventional Shear
              500.00       .00
          No Pore Pressure
     3 T Wall
          .00 Unit Weight
          Very Strong
     4 Layer 4 (CH)
          100.00 Unit Weight
          Conventional Shear
              120.00       .00
          No Pore Pressure
     5 Layer 5 (ML)
          117.00 Unit Weight
          Conventional Shear
              200.00     15.00
          Piezometric Line
          1
     6 Layer 6 (CH)
          100.00 Unit Weight
          Conventional Shear
              200.00       .00
          No Pore Pressure
     7 Layer 7 (CH)
          100.00 Unit Weight
          Linear Increase
              217.00      8.10
          No Pore Pressure
     8 Layer 8 (CH)
          100.00 Unit Weight
```

```
          Linear Increase
              374.00      8.30
          No Pore Pressure
     9 Layer 9 (CH)
          100.00 Unit Weight
          Linear Increase
              590.00      8.00
          No Pore Pressure
     10 Compacted Fill - Above T-Wall Base
          .00 Unit Weight
          Conventional Shear
                 .00       .00
          No Pore Pressure

PIEZOMETRIC LINES
         1      62.40 Water Level
                 .00     10.00
              145.50     10.00
              145.51     -1.00
              157.00     -1.00
              375.00     -1.00

         2      62.40 Piezometeric levels in ML
                 .00     10.00
              149.50     10.00
              156.00     10.00
              158.50      1.00
              167.00      1.00
              173.00     -1.00
              375.00     -1.00


DISTRIBUTED LOADS
      1
REINFORCEMENT LINES
              1        .00          2
118.083     -91.0 4380  848.
147.000     -4.25   4380      848

              2        .00          2
152.000     -4.25 -9300   464
180.917     -91.0 -9300   464

              3        .00          2
157.000     -4.25  4900   496
185.917     -91.0  4900   496

              4        .00          1
149.000     -4.25     0.   0.
149.000     -41.0     0.   0.

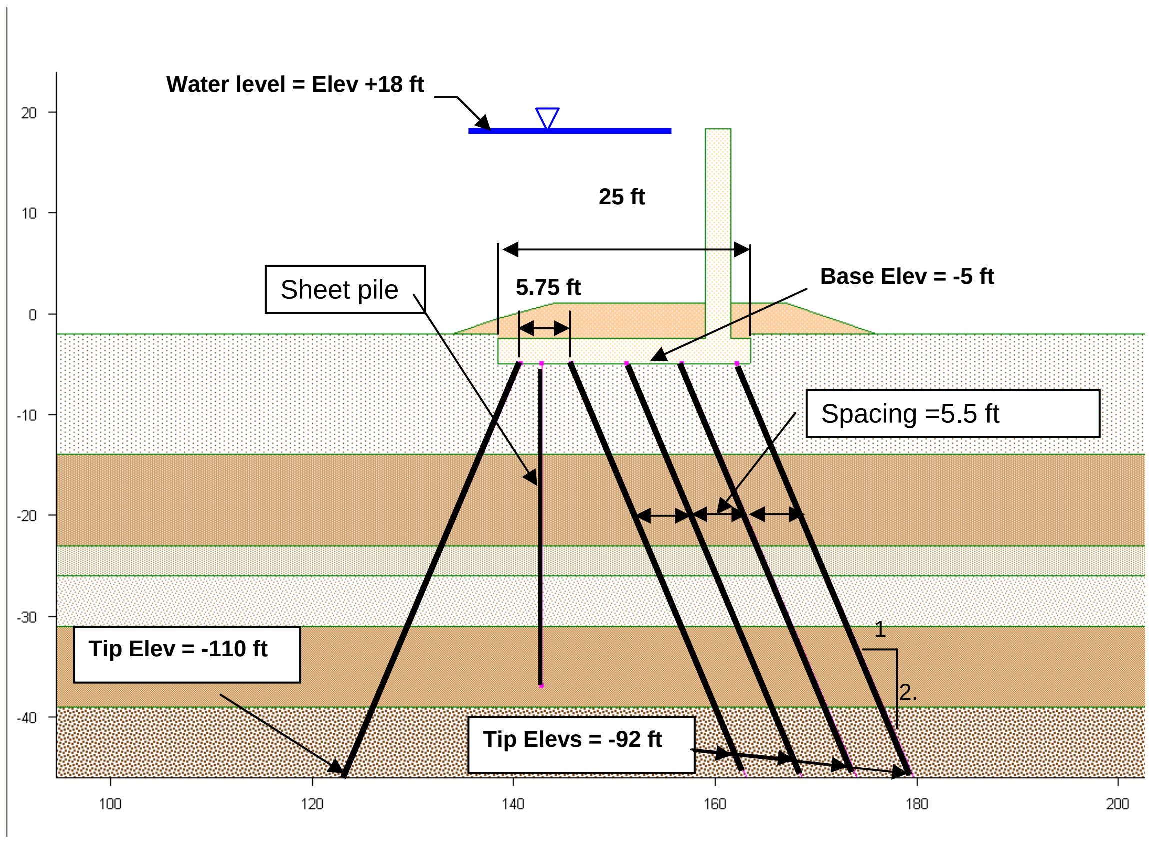

ANALYSIS/COMPUTATION
     Circular
         145.5     25        48
SINgle-stage Computations
RIGht Face of Slope
```

```
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

Design Example #2

A cross section of the wall section used for Example 2 is shown below. The water level used in this example is elevation 18.0 and the design situation is assumed to be a top of wall load case. The wall geometry including the wall dimensions and the pile layout is presented in Figure 1. The spacing of the piles in the out of plane direction is 5-ft. The piles tips extend to Elevation -110 ft. The soil profile and shear strengths for the foundation are shown in Figure 2.



Cross-sectional view of pile layout

Figure 1. Wall Geometry.

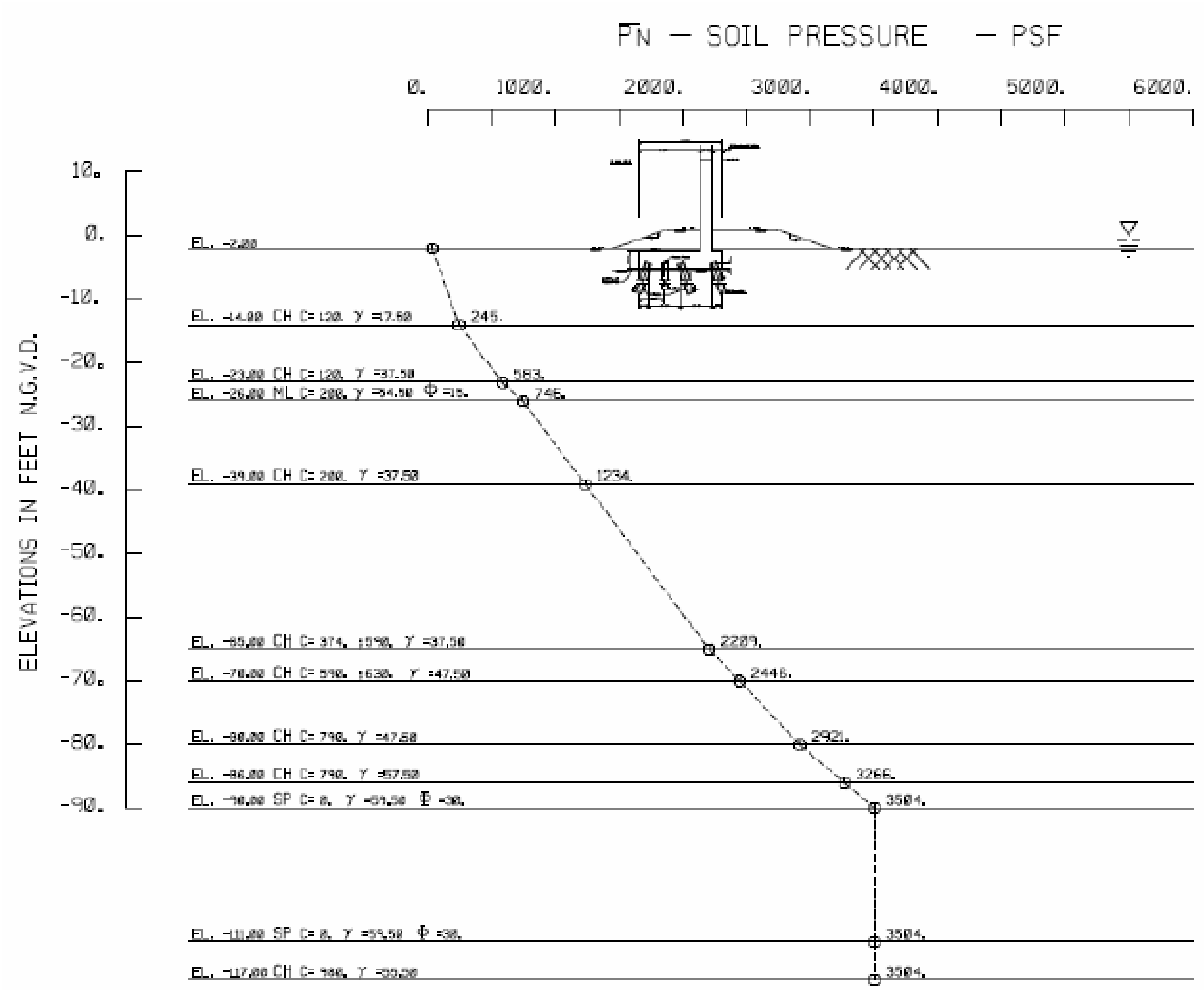


Figure 2. Soil Profile.

Step 1 Initial Slope Stability Analysis

Perform a Spencer's method slope stability analysis to determine the critical slip surface with the water load only on the ground surface and no piles. The required factor of safety according to the Hurricane and Storm Damage Reduction System Design Guidelines for the top of wall load condition is 1.4. For the design example, the critical failure surface is shown in Figure 1 where the factor of safety is 0.529. Because this value is less than the required value of 1.4, the T-Wall will need to carry an unbalanced load in addition to any loads on the structure.

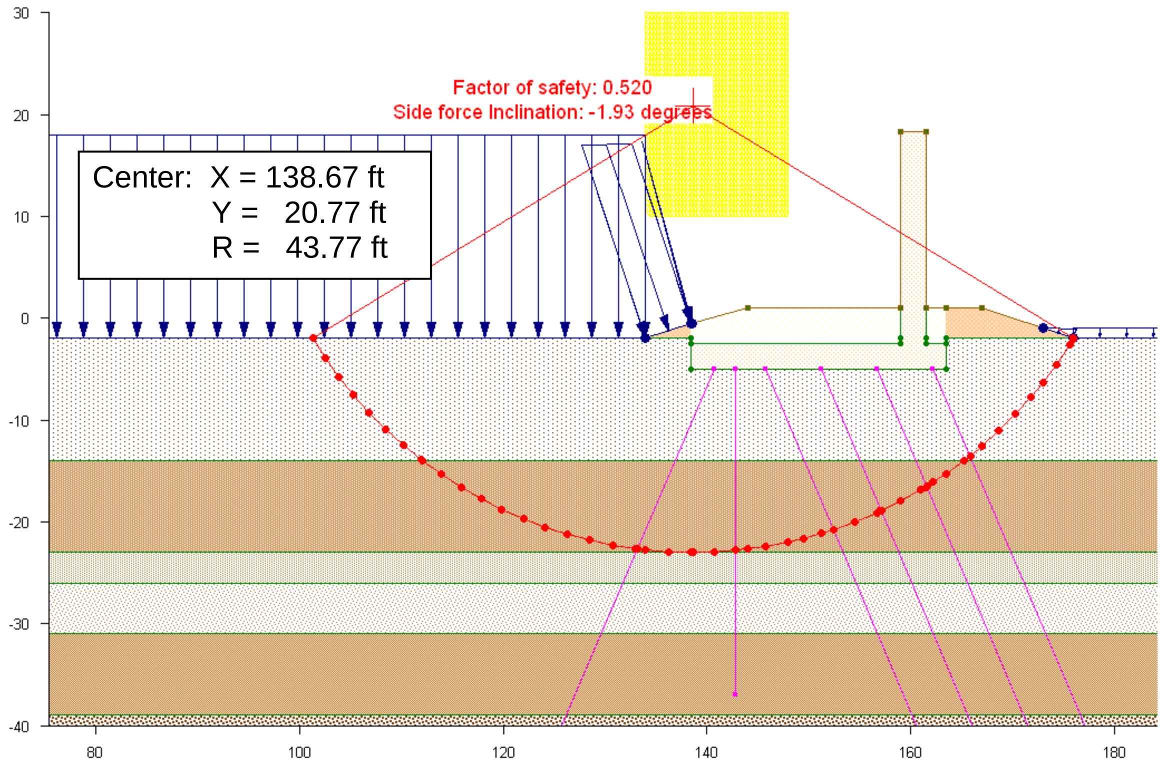


Figure 3. Spencer's analysis of the T-Wall without piles.

Step 2 Unbalanced Force Computations

Step 2 involves the determination of the (unbalanced) forces needed to provide the required global stability factor of safety. The base of the T-Wall is at elevation -5 ft. The critical failure surface extends down to elevation -23' in this example. The ground surface above the heel of the T-wall is at Elevation – 0.5 ft. In the design procedure, the unbalanced load is assumed to act halfway between these two elevations and at the x-coordinate of the heel of the T-wall. Thus, a horizontal line load is applied at elevation -11.75 ft at the x-coordinate along a vertical line passing through the heel of the T-wall. A trial and error process showed that an unbalanced force of 17480 lb/ft would result in a factor of safety of 1.4 as shown in Figure 2.

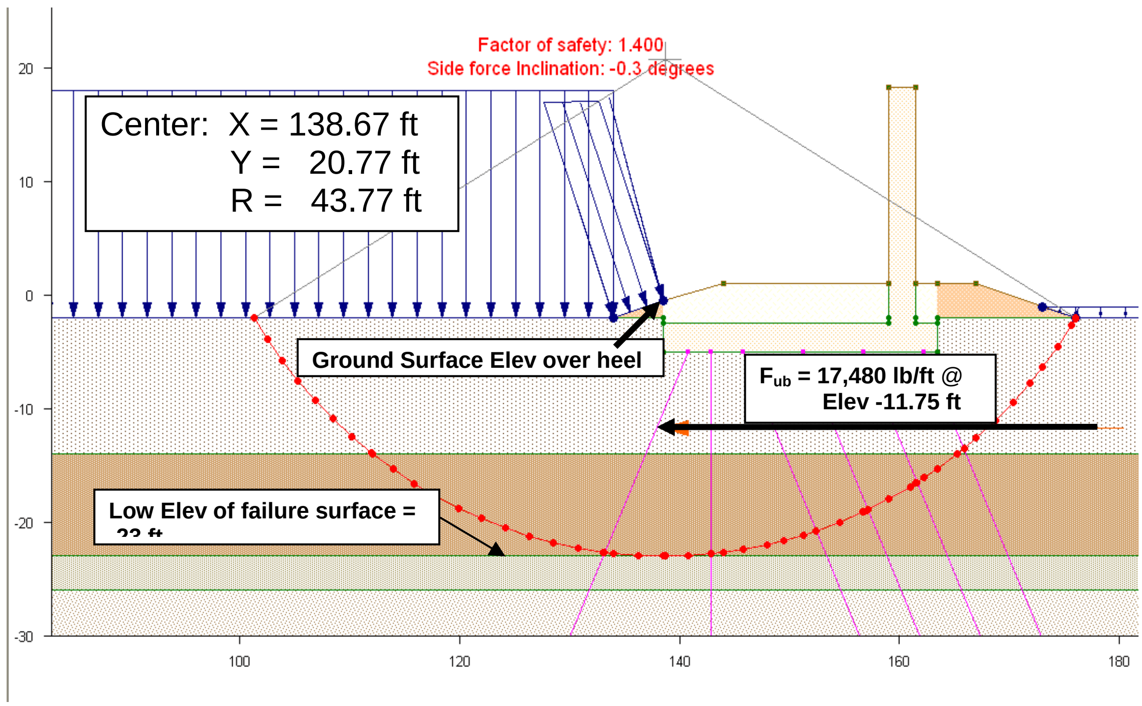


Figure 4. Spencer's analysis of the T-Wall with an unbalanced load to increase global stability.

It should be noted that unbalanced load was determined from a fixed grid search.for the critical as shown in Figure 2. Step 2 provides that if the pile foundation of the T-Wall can safely carry the unbalanced load on the structure, the global stability will meet the required factor of safety. The UTexas4 input files for Figures 1 and 2 are attached at the end of this example.

Step 3 Allowable Pile Capacity Analysis

3.1 For the preliminary analysis, allowable pile capacities determined by engineers in New Orleans District for the original design of this project are shown in Figure 3 for ultimate loads vs. depth. Since this is a top of wall load case, a 50% over stress is allowed according to the Hurricane and Storm Protection System Design Guidelines. For the case with load test data, the net factor would be 2.0/1.5 = 1.333. For the case with calculated capacities, the allowable load factor would be 3.0/1.5 = 2.0.

The allowable loads for compression pile can be determined using the chart on Figure 5 which plots pile load test results. This test was performed with casing above the critical failure surface to preclude contribution of skin friction above that point. The tip elevation of the piles is equal to Elevation -92.5 ft. where the ultimate load is 74 tons.

Allowable Compressive load = (74 tons x 2 kips/ton/ 2) x 1.5

= 111 kips

In the preceding calculation and in accordance with the Hurricane and Storm Protection Guidelines, the factor of safety was equal to 2 because the allowable capacity was determined from load tests and the 50% overstress is permitted as well.

The allowable tension load was determined from prior calculations provided by MVN that are shown in the lower panel of Figure 6. For a tip Elevation of -110-ft, the ultimate capacity is 120 tons. The capacity at elevation -23 is about 7 tons. Therefore, the tension capacity can be estimated as 120-7 = 113 tons. From this, the allowable capacity is determined as follows:

Allowable Tensile Load = (113 tons x 2 kips/ton/3) x 1.5

= 113 kips

In this calculation and in accordance with the Hurricane and Storm Protection Guidelines, the factor of safety was equal to 3 because the allowable capacity was determined by calculations based on the skin friction between the soil and the pile and the pile length.. The 50 % overstress factor was set to 1.5.

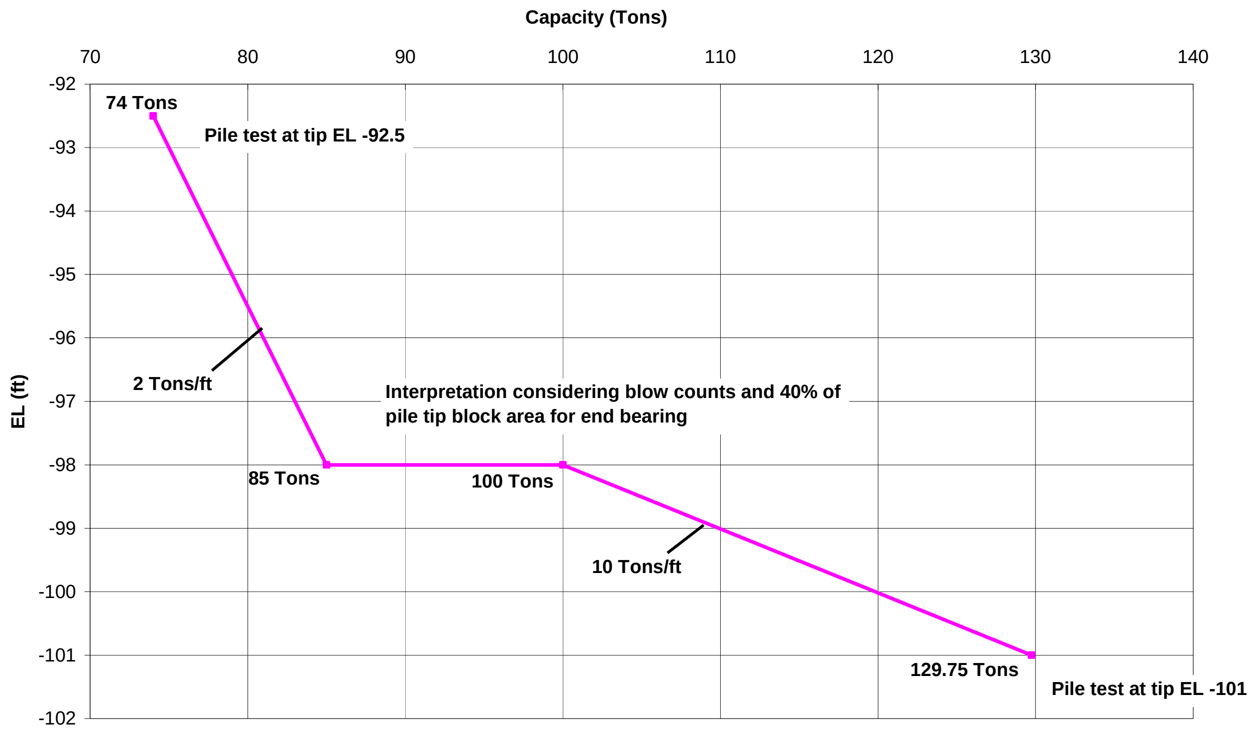


Figure 5. Pile Load Test Data

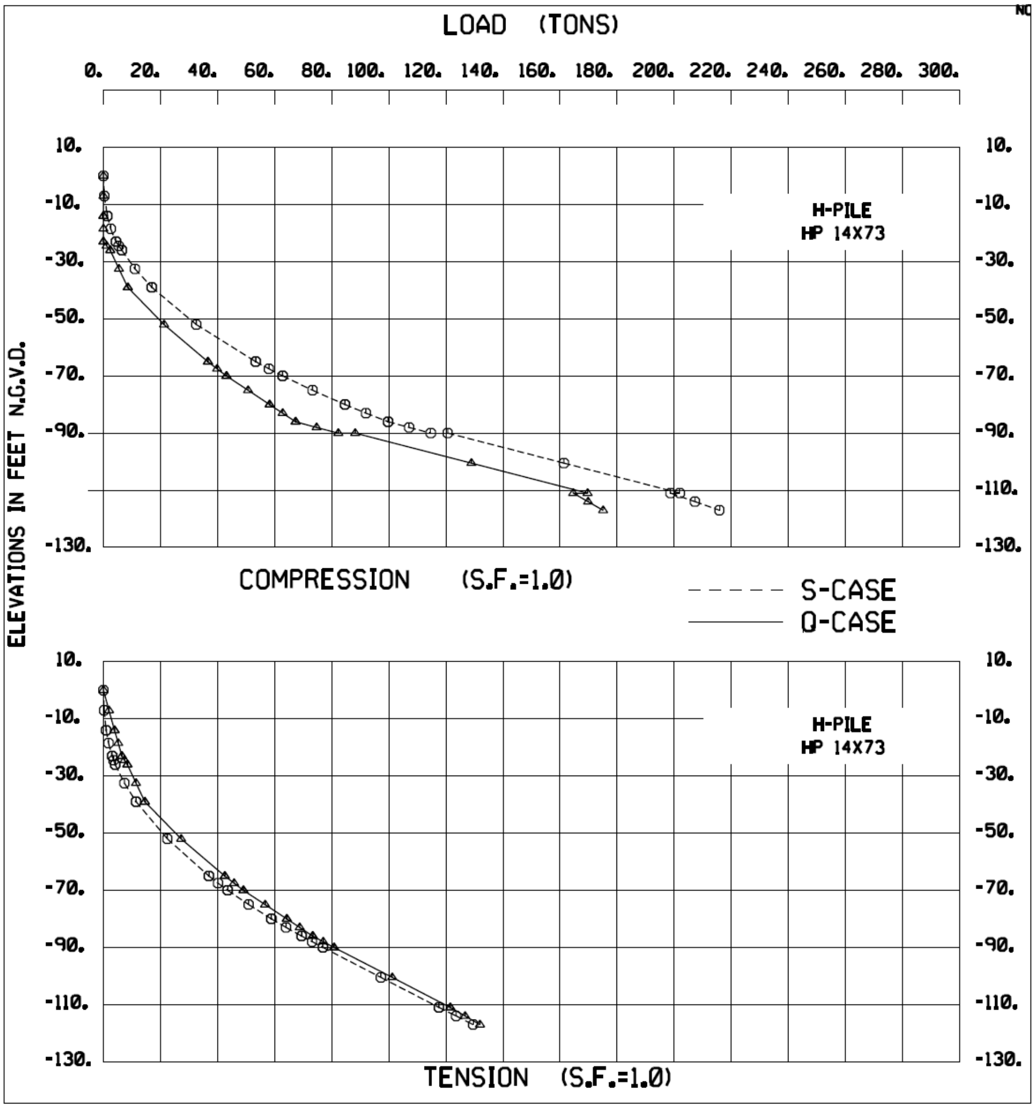


Figure 6. Ultimate Axial Capacity with Depth, Calculated

3.2 The allowable shear load is determined from pile head deflection versus lateral load plot on Figure 7 computed using the ENSOFT program LPILE. The ultimate load was determined to be 24.5 kips. The allowable load is determined to be 8.2 kips after dividing by the factor of safety of 3.0. However, the allowable load can be increased by 50% due to the 50% overstress allowed for the top of wall condition provided by the Hurricane and Storm Protection Guidelines. Thus, the allowable shear computed as follows:

Allowable pile shear = (24.5 kips / 3) x 1.5 = 12.25 kips

A summary of the allowable loads for the piles extending to Elevation -110 ft is presented in Table 1 below.

Table 1. Allowable Pile Capacities for Design Example 2 for Piles Extending to Elevation -110 ft	
Load Type	Allowable Load (kips)
Axial Compressive Load	194.6
Axial Tensile Load	120
Shear	12.25

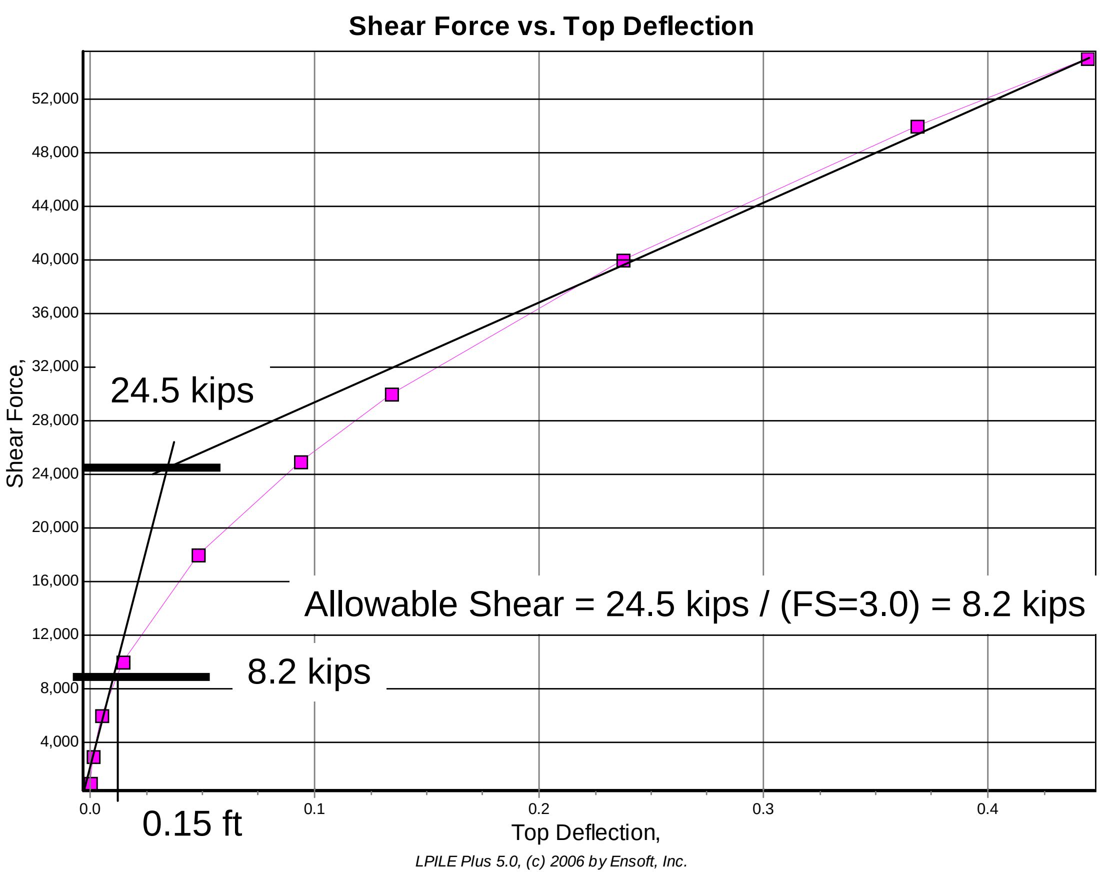


Figure 7. LPILE analysis of Pile head deflection vs shear force at critical surface to determine allowable shear force in piles.

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. The unbalanced force is converted to an "equivalent" force applied to the bottom of the T-wall, $F_{cap,}$ as calculated as shown below (See Figure 8):

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

Where:

F_{ub} = unbalanced force computed in step 2.

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

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

$$R = \sqrt[4]{\frac{EI}{Es}}$$

E = Modulus of Elasticity of Pile (lb/in^2)

I = Moment of Inertia of Pile (in^4)

Es = Modulus of Subgrade Reaction (lb/in^2) below critical failure surface. In New Orleans District this equates to the values listed as K_HB.

For the solution:
Piles = HP 14x73. I = 729 in^4, E = 29,000,000 psi

Soils – Importance of lateral resistance decreases rapidly with depth, therefore only first three layers are input – with the third assumed to continue to the bottom of the pile. The parameters were developed from soil borings from the New Orleans District and are as shown in Figure 9.

Silt, ϕ = 15, C = 200 psf, γ_{sat} = 117 pcf, K_HB ave. = k =167 psi
Clay 1, ϕ = 0 , C = 200 psf, γ_{sat} = 100 pcf, K_HB = k = 88.8 psi
Clay 2, ϕ = 0 , C = 374 psf, γ_{sat} = 100 pcf, K_HB = k = 165.06 psi

The top layer of silt under the critical failure surface is stiffer but only three feet thick. Will use a k = 100 psi.

R therefore is equal to 120 in = 10 feet

P_{cap} = 17,480 * (22.5/2 + 10) / (18 + 10) = 13,266 lb/ft

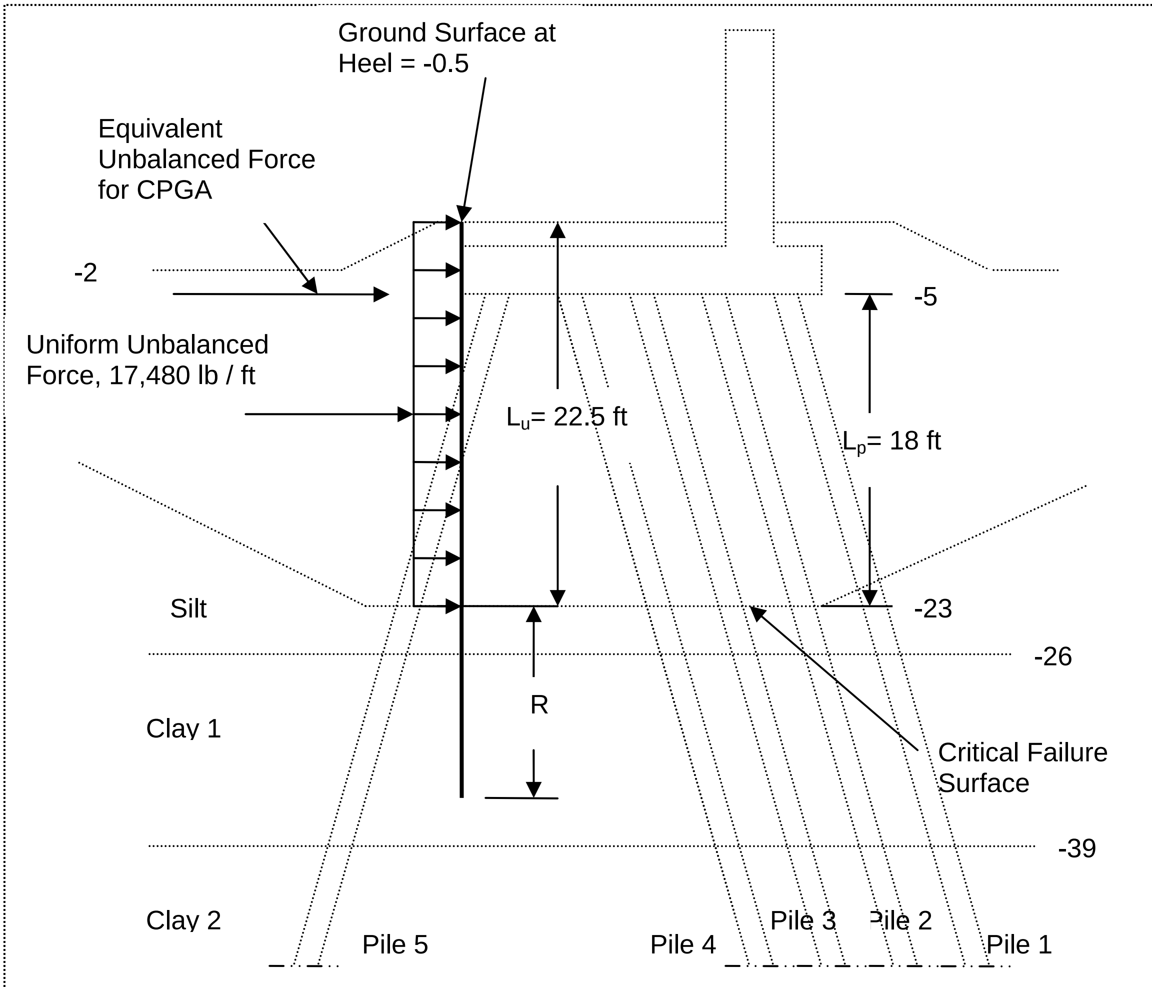


Figure 8. Equivalent Force Computation for Preliminary Design With CPGA

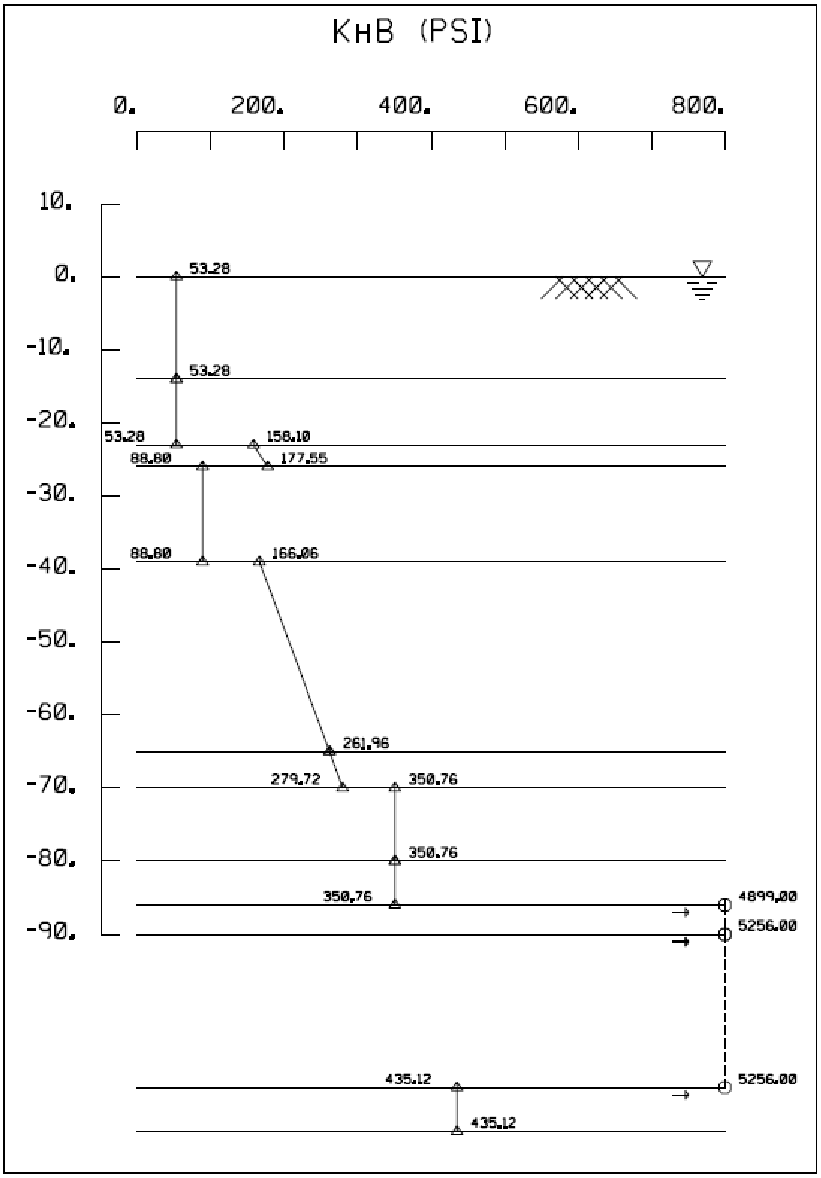


Figure 9. Soil Stiffness with Depth

4.2 This unbalanced force is then analyzed with appropriate load cases in CPGA. Generally 8 to 20 load cases may be analyzed depending on expected load conditions. For this example, only the water at top of wall case is analyzed but both pervious and impervious foundation conditions are evaluated. See the spreadsheet calculations in Attachment 3 for the computation of the input for CPGA. The model is a 5 foot strip of the pile foundation.

For the CPGA analysis, the soil modulus, Es is input at a very low value, 0.00001 psi, because the factor of safety is less than 1.0.

The CPGA output is shown in Attachment 4. A summary of results for the two load conditions analyzed are shown below:

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF
1	.0	.0	6.8	.0	-4.0	.0	.06	.02
2	.0	.0	47.2	.0	-3.8	.0	.42	.15
3	.0	.0	87.6	.0	-3.7	.0	.79	.28
4	.0	.0	127.9	.0	-3.5	.0	1.15	.41
5	.0	.0	-125.0	.0	3.5	.0	1.11	.40

LOAD CASE - 2

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF
1	.0	.0	22.3	.0	-3.4	.0	.20	.07
2	.0	.0	56.9	.0	-3.3	.0	.51	.18
3	.0	.0	91.4	.0	-3.2	.0	.82	.29
4	.0	.0	126.0	.0	-3.0	.0	1.14	.40
5	.0	.0	-97.8	.0	3.1	.0	.87	.31

Where:
F1 = Shear in pile at pile cap perpendicular to wall
F2 = Shear in Pile at Pile Cap parallel to wall
F3 = Axial Load in Pile
M1 = Maximum moment in pile perpendicular to wall
M2 = Maximum moment in pile parallel to wall
M3 = Torsion in pile
ALF= Axial load factor – computed axial load divided by allowable load
CBF= Combined Bending factor – combined computed axial and bending forces relative to allowable forces

From the CPGA analysis, axial loads in the piles are somewhat over the allowable values. Still they are close to being OK, and knowing that the initial design using CPGA is conservative compared to the more exact Group 7 analysis, this configuration will be carried forward into the Group 7 analysis.

Computed deflections from the CPGA analysis are shown below:

PILE CAP DISPLACEMENTS

LOAD CASE	DX IN	DZ IN	R RAD
1	-.7899E+00	-.3207E+00	-.1201E-02
2	-.6897E+00	-.2476E+00	-.1028E-02

These deflections are less than the allowable vertical deflection (DZ) of 0.5 inches X an overstress factor of 1.5 = 0.75" and the allowable horizontal deflection (DX) of 0.75 inches X an allowable overstress factor of 1.5 = 1.125 inches from the Hurricane and Storm Damage Reduction Design Guidelines.

4.4 Sheet pile design. Seepage design of the sheet pile is not performed for this example.

4.5 Check for resistance against flow through. Since the pile spacing is uniform, we will analyze one row of piles parallel with the loading rather than the entire monolith.

a. Compute the resistance of the flood side row of piles.

$$\sum P_{all} = \frac{n\sum P_{ult}}{1.5}$$

Where:

n = number of piles in the row within a monolith. Or, for monoliths with uniformly spaced pile rows, n = 1. Use 1 for this example

$P_{ult} = \beta(9S_u b)$

S_u = soil shear strength

b = pile width = 14"

β = group reduction factor pile spacing parallel to the load - since the piles batter opposite to each other, there group affects are not computed.

For the soils under the slab, S_u = 120 psf

Therefore: P_{ult} = 9(120 psf)(14 in/12 in/ft) = 1,260 lb/ft

ΣP_{ult} = summation of P_{ult} over the height L_p, as defined in paragraph 4.1

For single layer soil is P_{ult} multiplied by L_p (18 ft) - That is the condition here since the shear strength is constant from the base to the critical failure surface.

ΣP_{ult} = 1,260 lb/ft (18 ft) = 22,680 lb

ΣP_{all} = 1(22,680 lb)/1.5 = 15,120 lb

b. Compute the load acting on the piles below the pile cap.

$$F_{up} = wf_{ub}L_p$$

Where:

w = Monolith width. Since we are looking at one row of piles in this example, w = the pile spacing perpendicular to the unbalanced force (s_t) = 5 ft.

$$f_{ub} = \frac{F_{ub}}{L_u}$$

F_{ub} = Total unbalanced force per foot from Step 2 = 17,480 lb/ft
L_u = 22.5 ft
L_p = 18 ft

f_{ub} = 17,480 lb/ft / 22.5 ft = 777 lb/ft/ft

F_p = 5 ft(777lb/ft/ft)(18ft) = 69,930lb

c. Check the capacity of the piles 50% of F_p = 69,930 lb(0.50) = 34,965 lb

The capacity ΣP_{all} = 15,120 lb < 34,965 lb so the flood side row of piles is not adequate and the capacity of the rest of the pile rows must be added. The capacity ΣP_{all} is the same as computed for the flood side row of piles except as modified by the group reduction factor. Since the batter of the flood side and next row of piles is opposite, the flood side pile can be considered as single pile and the next row of piles as a lead row of piles. The next rows of piles would be trailing piles. The row spacing is 5'6".

Using a row spacing of 5'6", the group reduction factor (β) for the lead piles is

$$\beta = 0.7(s/b)^{0.26} \quad ; or = 1.0 \text{ for } s/b > 4.0 \qquad (5)$$

Where:
s = spacing between piles parallel to loading

For s = 5'6" and b=14" for HP14x73 piles, s/b = 4.71
Since s/b = 4.71 < 4.0, β = 1.0 for the lead pile

For trailing piles, the reduction factor, β, is:

$$\beta = 0.48(s/b)^{0.38} \quad ; or = 1.0 \text{ for } s/b > 7.0 \qquad (6)$$

$$\beta = 0.48(4.71)^{0.38} = 0.87$$

Shortcutting the math in the equations presented in the previous page, for the trailing piles, ΣP_{all} = β ΣP_{all} = 0.87 * 15,120 = 13,154 lb

Summing ΣP_{all} for all 5 pile rows , the total allowable unbalanced force is:

15,120 + 15,120 + 13,154 + 13,154 + 13,154 = 69,702 lb

Since F_p = 69,930 lb, the difference is 228 lb, or about 0.3%. For the purposes of this example, this is considered close enough.

4.6 Second flow through 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:

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

Where:

A_pS_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. – See Figure 10. S_u =120 psf

A_pS_u = (18(22.5+36.5)/2)(120 psf) = 64,152 lb

FS = Target factor of safety used in Steps 1 and 2. – 1.5

s_t= the spacing of the piles transverse (perpendicular) to the unbalanced force 5 ft

b = pile width – 14 inches

$f_{pb}L_p$ = (777 lb/ft)(18 ft) = 13,986 lb

$$\frac{A_pS_u}{FS}\left[\frac{2}{(s_t - b)}\right] = \frac{64{,}152}{1.5}\left[\frac{2}{5-\left(\frac{14}{12}\right)}\right] = 22{,}314 \text{ lb}$$

Therefore, capacity against flow through is OK

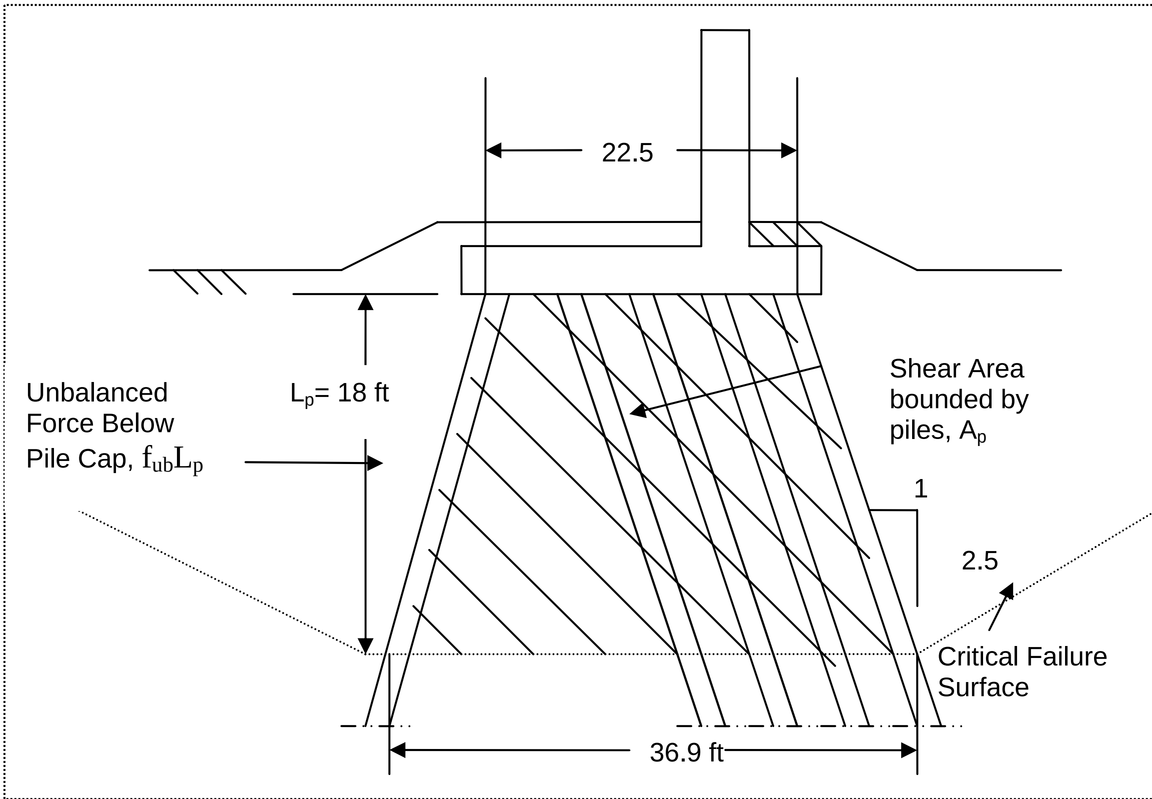


Figure 10. Shear Area for Flow-through Check

Step 5 <u>Pile Group Analysis</u>

5.1 A Group 7 analysis is performed using all loads applied to the T-wall structure. Critical load cases from step 4 would be used. In this example, only one load case with two foundation conditions is shown.

5.2 The loads applied in the Group 7 model include the distributed loads representing the unbalanced force that acts directly on the piles and also the water loads and self-weight of the wall that acts directly on the structure. In Group 7 these loads are resultant horizontal and vertical forces and the moments per width of spacing that act on the T-wall base (pile cap). They also include the unbalance force from the base of the cap to the top of soil, converted to a force and moment at the base of the structure. These forces are calculated using a worksheet or Excel spreadsheet and are shown at then end of the spreadsheets shown in Attachment 3. For this analysis the resultant forces per 5-ft of pile spacing were:

Pervious Foundation Condition

Vertical force	=	134,114 lb
Horizontal force	=	97,636 lb
Moment	=	7,347,343 in-lbs

Impervious Foundation Condition

Vertical force	=	184,583 lb
Horizontal force	=	97,636 lb
Moment	=	15,636,093 in-lbs

5.3 The unbalanced load below the bottom of the footing is applied directly as distributed loads on the pile. Check if ($n\Sigma P_{ult}$) of the flood side pile row is greater than 50% F_p, (from 4.5)

.

$(n\Sigma P_{ult}) = 1\ (22{,}680) = 22{,}680$ lb

50% F_p = 34,965 lb

Since $n\Sigma P_{ult}$ < 50% F_p ,distribute P_{ult} onto the flood side (left) row of piles.

P_{ult} = 1,260 p/ft = 105 lb/in

The remainder of F_p is divided among the remaining piles = 69,930 – 22,680 = 47,250 lb

This is distributed onto each pile according to a ratio of the group factors shown in table 2 (pile numbers as shown in figure 6) as computed in step 4.5. Since the load will be applied to the piles in Group 7 as a distributed load in lb/in, First, the total load will be divided into the load applied to one vertical inch

= 47,250 lb / (18ft /12in/ft) = 218.8 lb/in.

The sum of the distribution factors is 0.87+0.87+0.87+1.0 = 3.61.

The force on the trailing piles is 218.8 lb/in * 0.87/3.61 = 52.7 lb/in
The force on the leading pile is 218.8 lb/in * 1.0/3.61 = 60.6 lb/in

Table 2. Pile Reduction Factors and Ultimate Distributed Loads for each Pile

Pile	(s/b)	Pile type	β	Load, lb/in
1	4.71	Trailing	0.87	52.7
2	4.71	Trailing	0.87	52.7
3	4.71	Trailing	0.87	52.7
4	4.71	Lead	1.0	60.6
5	4.71	Single	1.0	105

5.4 Thus, all the loads including the pile cap loads and the distributed loads are identified and and a Group 7 analysis is performed using all the loads applied to the T-wall system. The group 7 model is shown in Figure 11.

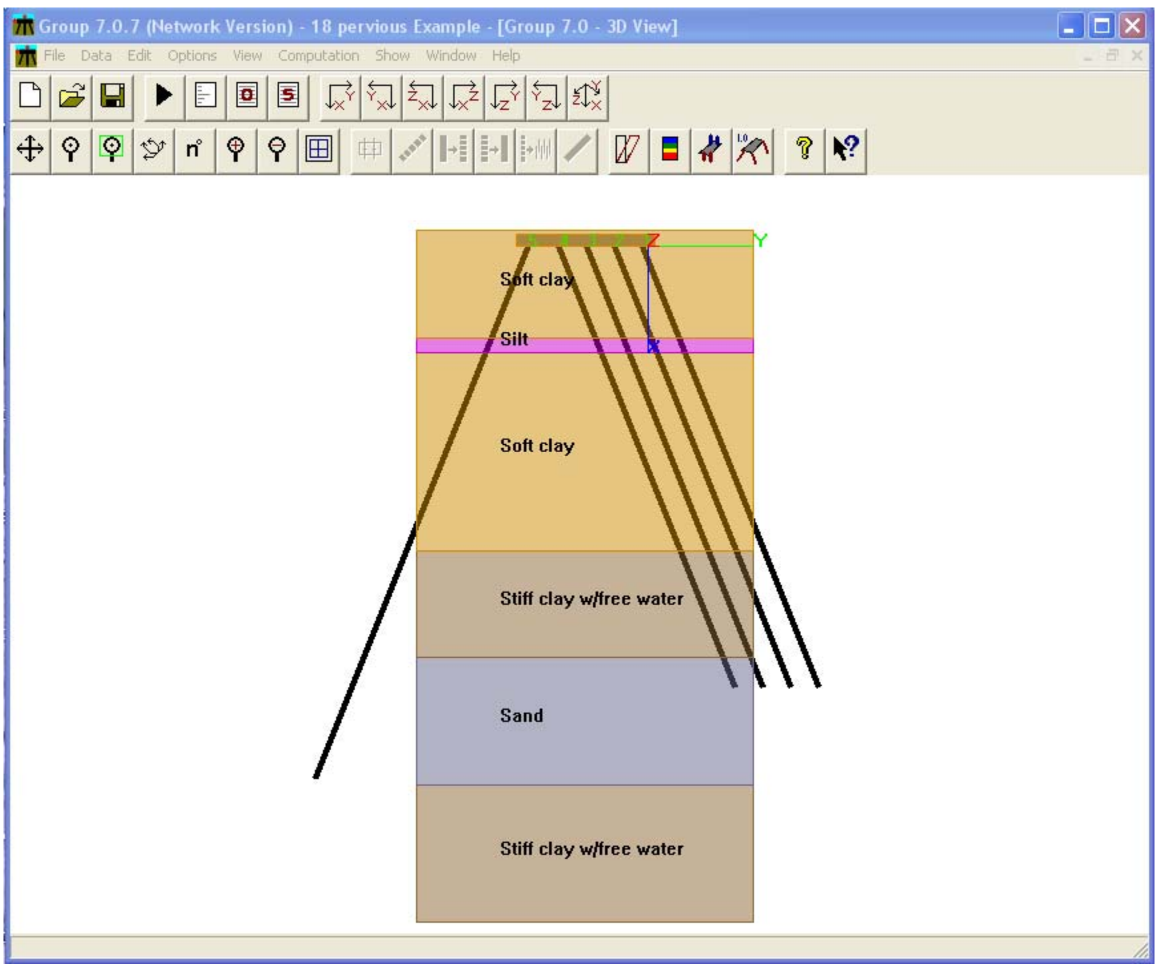


Figure 11. Group 7 Model

5.2 Since the factor of safety without piles was less than one, the lateral stiffness of the soil from the bottom of the pile cap to the top of the critical failure surface at -23 feet will be set to zero by using very small numbers for the ultimate shear strength of the soil. The lateral soil reaction against the pile (not including the applied soil loads) is shown in Figure 12

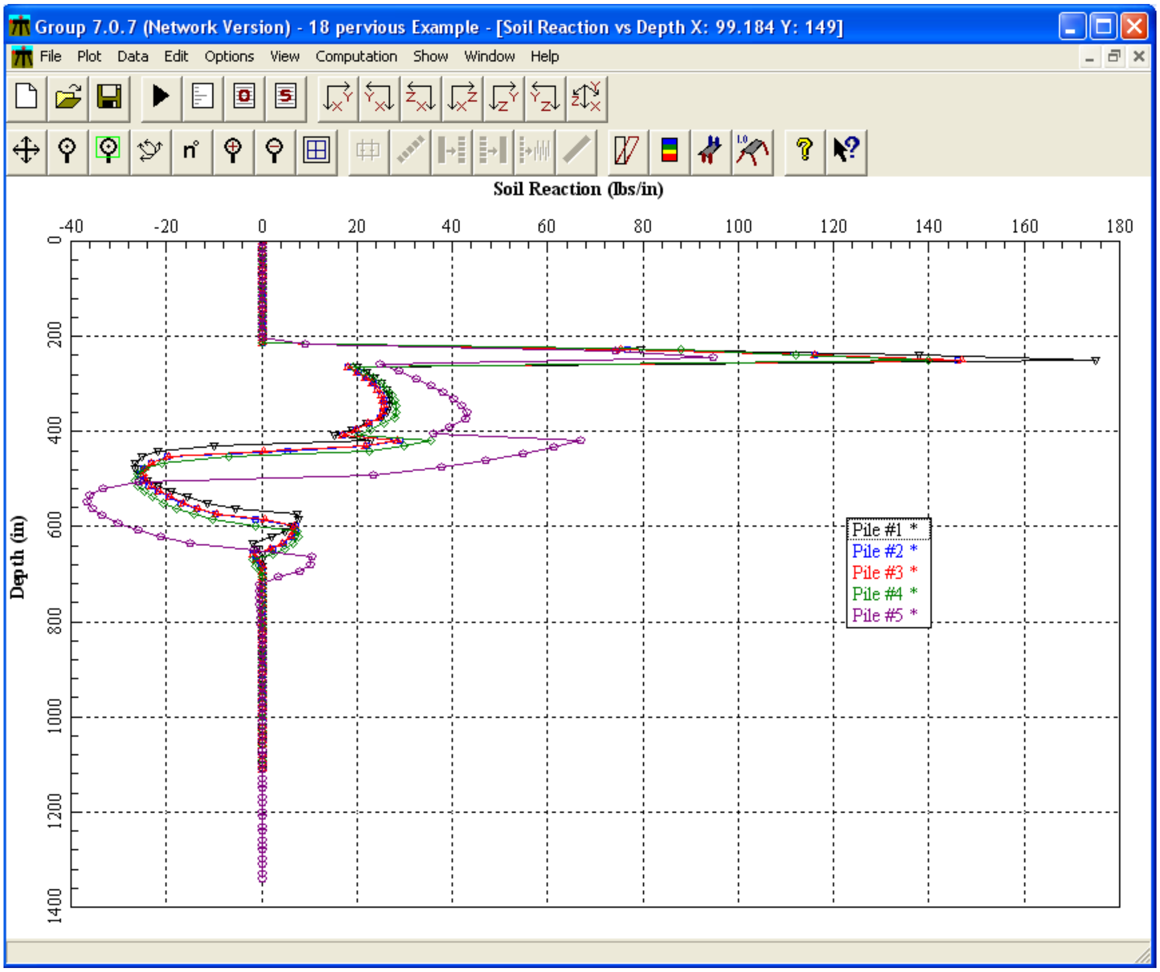


Figure 12 Soil Reaction on Piles with Depth

The pile responses to the applied loads are the sought after information from the Group 7 analysis to determine if the design requirements are achieved for a given pile layout. An illustration of the moment in the piles versus depth for this iteration shown in Figure 13 for the pervious sheet pile condition. An illustration of the shear is shown in Figure 14.

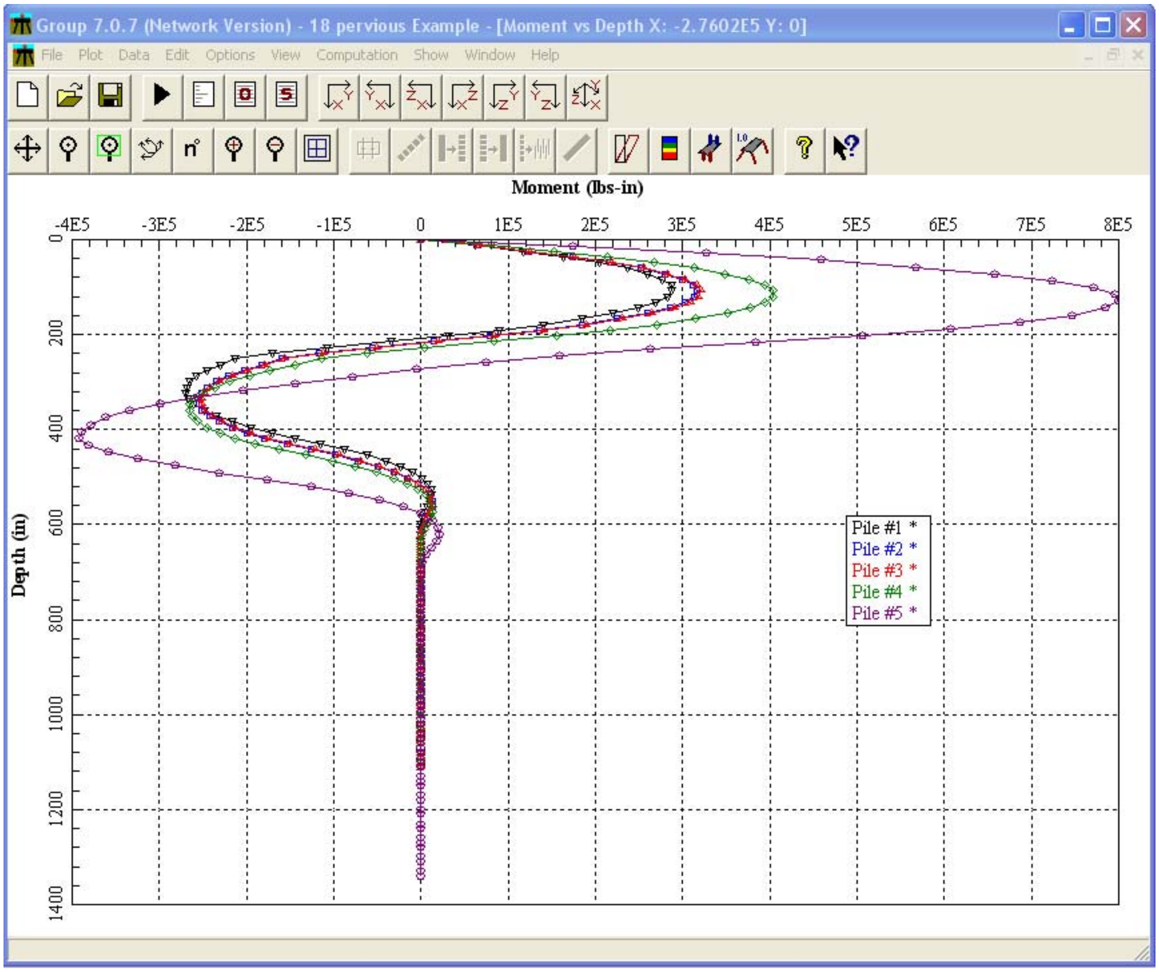


Figure 13 Moment in Piles With Depth

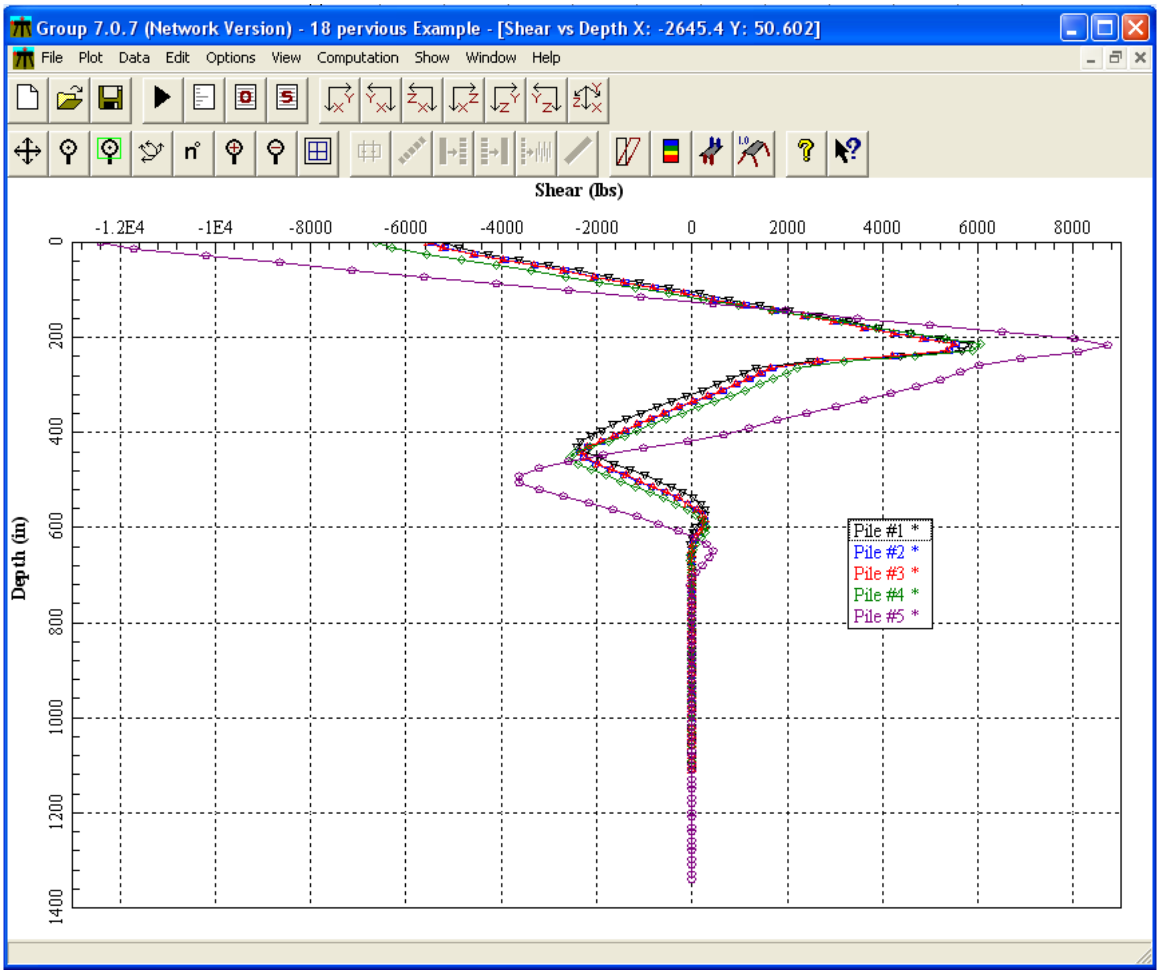


Figure 14. Shear diagrams for each of the four piles.

Grouped displacements of the pile cap from the Group 7 analysis are listed in Table 4.

Table 4. Grouped Pile Foundation displacements from Group 7 analysis			
	Vert. Displacement, Inches	Hor. Displacement, Inches	Rotation Radians
Pervious	-0.2120	0.5254	0.0008644
Impervious	-0.1549	0.4424	0.0007479

These deflections are less than the allowable vertical deflection (DZ) of 0.5 inches and only slightly greater than the allowable horizontal deflection (DX) of 0.75 inches from the Hurricane and Storm Damage Reduction Design Guidelines, even with out increases allowed for the top of wall load case. Figure 13 below shows displacement with depth.

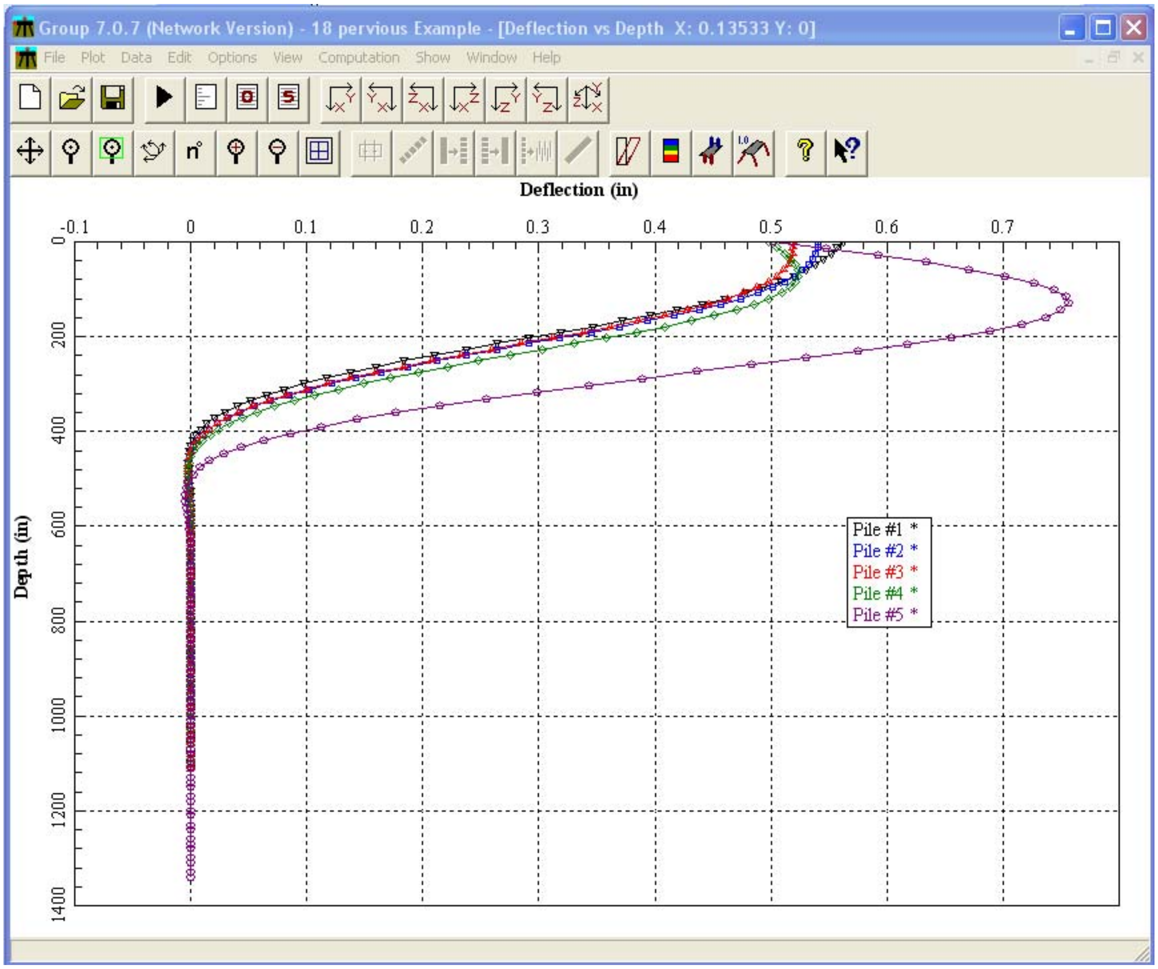


Figure 15. Deflection with Depth for the Pervious Foundation Condition.

5.3 Specifically, the deflections, axial loads and shear and bending moments in the piles are what must be evaluated to determine if the design requirements are met. The results of the Group 7 analysis are reported where the pile responses for the full loading conditions on T-wall systems are listed are listed in Table 5.

Table 5. Axial, shear and moments in piles computed by Group 7 for full loading conditions that include distributed loads applied directly to piles and resultant horizontal, vertical and moments due to water loads.

Pervious Case				
Pile Number	Pile Location	Axial (kips)	Shear (kips)	Maximum Moment In-kips
1	Right	8.21 (C)	5.82	288
2	Right-center	49.7 (C)	5.54	321
3	Center	80.8 (C)	5.49	473
4	Left-center	112 (C)	6.04	404
5	Left	-111 (T)	8.71	800
Impervious Case				
1	Right	24.7 (C)	5.68	303
2	Right-center	57.5 (C)	5.47	326
3	Center	84.5 (C)	5.43	331
4	Left-center	111 (C)	6.0	414
5	Left	-84.2 (T)	8.65	808

The axial forces and shear in Table 5 are then compared with allowable pile capacities summarized in Table 1 as determined in Step 3. The results of the comparison show that:

a. The axial compressive forces in the Piles 1, 2 and 3 are both less than the axial compressive pile capacity of 111 kips for both the pervious and impervious conditions. The axial force in pile 4 is slightly over for the pervious case and could be regarded as OK or the piles could be driven slightly deeper.
b. The axial tensile forces from the left (flood side) Pile 5 are less than the allowable tensile force of 113 kips..
c. The shear forces in each of the three piles are lower than the allowable shear of 12.2 kips for both foundation conditions.
d. Moment and axial forces in the piles would also be checked for structural strength according to criteria in the Hurricane and Storm Damage Reduction System Design Guidelines and EM1110-2-2906.

Step 6 <u>Pile Group Analysis (unbalanced force)</u>

6.1 A Group 7 analysis was performed with the unbalance force applied directly to the piles. The uniform unbalanced force above the base of the wall is added as a force and moment at the base of the wall. The distributed loads are statically equivalent to the unbalanced force of 17,480 lb/ft. No loads are applied to the cap except unbalance forces. The p-y springs are set to 0 to the critical failure surface by setting the ultimate shear stress of these soils at a very low value. The distributed loads were computed in the previous step and shown in Table 6. The pile cap forces were computed in the Excel spreadsheet of Attachement 3::

Py = 17,480 lb
Mz = -471,960 in-lb

The pile responses from the Group 7 analysis are shown in Table 10 below:

Table 6. Axial and shear Pile loads per 5-ft of width computed by Group 7 for static equivalent to unbalanced load only.

Pile	Axial (lb)	Shear (lb)
1	-44,800 (T)	5,650
2	-1,780 (T)	5,460
3	42,100 (C)	5,400
4	75,500 (C)	5,980
5	-75,800 (T)	8,590

Step 7 <u>Pile Reinforced Slope Stability Analysis</u>

7.1 The UT4 pile reinforcement analysis using the circle from Step 5 is performed to determine if the target Factor of Safety of 1.4 is achieved. The piles are treated as reinforcements in the UT4 and the shear and axial forces from Step 6 are used to determine these forces. The forces in Table 6 must be converted to unit width conditions by divided by the 5-ft pile spacing to be used as the axial and shear forces in the pile reinforcements in UT4. Additionally, the sign must be changed because compressive forces are negative in UT4. The UT4 forces used for pile reinforceement are shown in the Table 6. The results of the analysis are shown in Figure 16. The factor of safety is 1.526 which is greater than the target factor of safety of 1.4 for global stability. Since the compute factor of safety is slightly below the required value an additional iteration is required. The unbalanced force will be adjusted slightly to improve the global factor of safety.

Table 11. Axial and shear Pile reinforcement forces per unit width for input into UTEXAS4.		
Pile	Axial (lb)	Shear (lb)
1	8,960 (T)	1,130
2	356 (T)	1,092
3	-8,420 (C)	1,080
4	-15,100 (C)	1,196
5	15,160 (T)	1,718

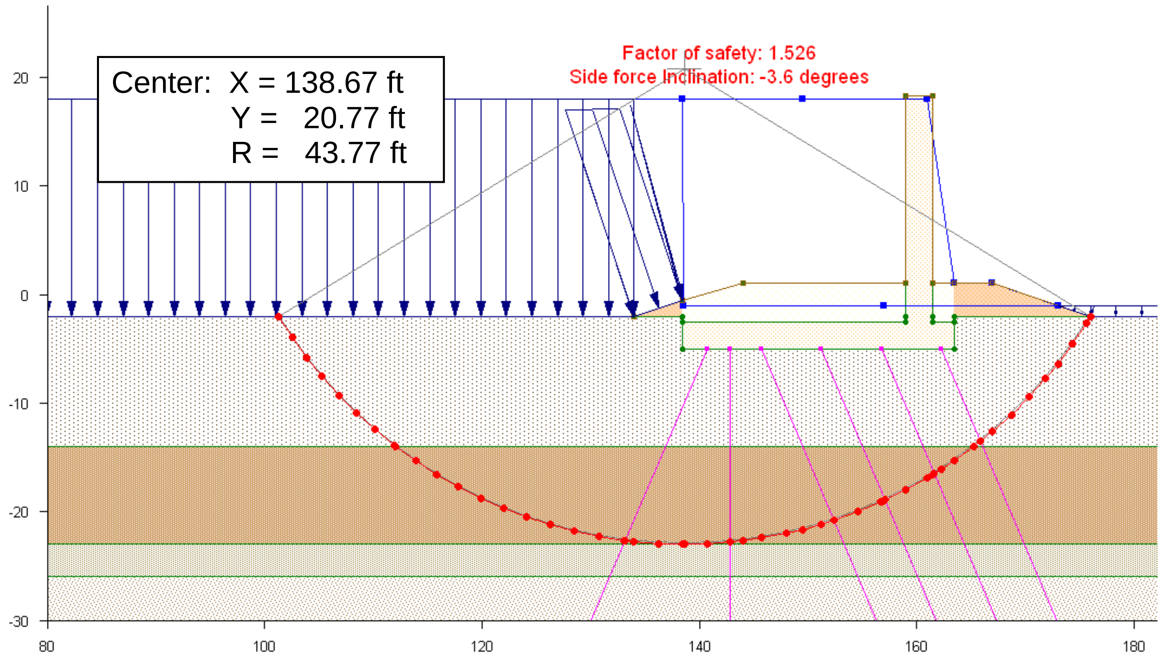


Figure 16. Factor of safety computed using pile forces from Group 7 analysis And critical circle from fixed grid analysis

Attachment 1 – UTexas analysis without piles that results in Figure 1.
Search for Critical Circle

```
EADING
    T-Wall Deep Seated Analysis
    Step 2 Search for unbalanced load

PROFILE LINES
         1     1 Layer 3 (CH) - Floodside
                  .00      -2.00
               134.00      -2.00
               138.50      -2.00

         2     1 Layer 3 (CH) - Landside
               163.50      -2.00
               375.00      -2.00

         3     2 Compacted Fill - FS
               134.00      -2.00
               138.50       -.50

         4     2 Compacted Fill - LS
               163.50       1.00
               167.00       1.00
               176.00      -2.00

         5     3 T-Wall
               138.50      -5.00
               138.50      -2.50
               159.00      -2.50
               159.00      -2.00
               159.00      18.30
               161.50      18.30
               161.50      1.00
               161.50      -2.00
               161.50      -2.50
               163.50      -2.50
               163.50      -5.00

         6     1 Layer 3 (CH) - Under Wall
               138.50      -5.00
               163.50      -5.00

         7     4 Layer 4 (CH)
                  .00     -14.00
               375.00     -14.00

         8     5 Layer 5 (ML)
                  .00     -23.00
               375.00     -23.00

         9     6 Layer 6 (CH)
                  .00     -26.00
               375.00     -26.00

        10     7 Layer 7 (CH)
```

```
               .00     -31.00
            375.00     -31.00

      11      8 Layer 8 (CH)
               .00     -39.00
            375.00     -39.00

      12      9 Layer 9 (CH)
               .00     -65.00
            375.00     -65.00

      13     10 Compacted Fill - Above T Wall Base  FS
            138.50       -.50
            144.00       1.00
            159.00       1.00

      14     10 Compacted Fill - Above T Wall Base  LS
            161.50       1.00
            163.50       1.00


MATERIAL PROPERTIES
     1 Layer 3 (CH)
          80.00 Unit Weight
          Conventional Shear
              120.00        .00
          No Pore Pressure
     2 Compacted Fill
          110.00 Unit Weight
          Conventional Shear
              500.00        .00
          No Pore Pressure
     3 T Wall
          .00 Unit Weight
          Very Strong
     4 Layer 4 (CH)
          100.00 Unit Weight
          Conventional Shear
              120.00        .00
          No Pore Pressure
     5 Layer 5 (ML)
          117.00 Unit Weight
          Conventional Shear
              200.00      15.00
          Piezometric Line
          1
     6 Layer 6 (CH)
          100.00 Unit Weight
          Conventional Shear
              200.00        .00
          No Pore Pressure
     7 Layer 7 (CH)
          100.00 Unit Weight
          Linear Increase
              217.00       8.10
          No Pore Pressure
     8 Layer 8 (CH)
```

```
          100.00 Unit Weight
          Linear Increase
              374.00      8.30
          No Pore Pressure
     9 Layer 9 (CH)
          100.00 Unit Weight
          Linear Increase
              590.00      8.00
          No Pore Pressure
     10 Compacted Fill - Above T-Wall Base
          .00 Unit Weight
          Conventional Shear
                 .00       .00
          No Pore Pressure

PIEZOMETRIC LINES
         1      62.40 Water Level
                 .00     18.00
              138.50     18.00
              138.51     -1.00
              157.00     -1.00
              375.00     -1.00

         2      62.40 Piezometeric levels in ML
                 .00     18.00
              149.50     18.00
              161.00     18.00
              163.50      1.00
              167.00      1.00
              173.00     -1.00
              375.00     -1.00


DISTRIBUTED LOADS
      1
REINFORCEMENT LINES
              1        .00         2
100.00       -100.0      0      0.
140.75       -5.000  0   0

              2         .00        2
145.75       -5.000         0      0.
182.55       -92.00         0.     0.

              3         .00        2
151.25       -5.000       0.     0.
188.05       -92.00       0.     0.

              4         .00        2
156.75       -5.000           0.    0.
193.55       -92.0        0.    0.

              5         .00        2
162.25   -5.000       0.    0.
199.30   -92.00       0.    0.

              6         .00        1
```

```
142.875     -5.00 0.0   0.0
142.875     -37.00      0.0   0.0


ANALYSIS/COMPUTATION
       Circular Search 2
          40.00     40.00
         134.00     10.00
         148.00     10.00
         148.00     30.00
         134.00     30.00
           2.00       .01
     Tangent
         -23.00

SINgle-stage Computations
RIGht Face of Slope
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

Attachment 2 – UTexas analysis with unbalanced load that results in Figure 2. Search for the unbalanced Load

```
HEADING
    T-Wall Deep Seated Analysis
    Step 2 Search for unbalanced load

PROFILE LINES
         1    1 Layer 3 (CH) - Floodside
                 .00      -2.00
              134.00      -2.00
              138.50      -2.00

         2    1 Layer 3 (CH) - Landside
              163.50      -2.00
              375.00      -2.00

         3    2 Compacted Fill - FS
              134.00      -2.00
              138.50       -.50

         4    2 Compacted Fill - LS
              163.50       1.00
              167.00       1.00
              176.00      -2.00

         5    3 T-Wall
              138.50      -5.00
              138.50      -2.50
              159.00      -2.50
              159.00      -2.00
              159.00      18.30
              161.50      18.30
              161.50      1.00
              161.50      -2.00
              161.50      -2.50
              163.50      -2.50
              163.50      -5.00

         6    1 Layer 3 (CH) - Under Wall
              138.50      -5.00
              163.50      -5.00

         7    4 Layer 4 (CH)
                 .00     -14.00
              375.00     -14.00

         8    5 Layer 5 (ML)
                 .00     -23.00
              375.00     -23.00

         9    6 Layer 6 (CH)
                 .00     -26.00
              375.00     -26.00
```

```
      10     7 Layer 7 (CH)
                .00     -31.00
             375.00     -31.00

      11     8 Layer 8 (CH)
                .00     -39.00
             375.00     -39.00

      12     9 Layer 9 (CH)
                .00     -65.00
             375.00     -65.00

      13    10 Compacted Fill - Above T Wall Base  FS
             138.50       -.50
             144.00       1.00
             159.00       1.00

      14    10 Compacted Fill - Above T Wall Base  LS
             161.50       1.00
             163.50       1.00


MATERIAL PROPERTIES
     1 Layer 3 (CH)
          80.00 Unit Weight
          Conventional Shear
              120.00        .00
          No Pore Pressure
     2 Compacted Fill
          110.00 Unit Weight
          Conventional Shear
              500.00        .00
          No Pore Pressure
     3 T Wall
          .00 Unit Weight
          Very Strong
     4 Layer 4 (CH)
          100.00 Unit Weight
          Conventional Shear
              120.00        .00
          No Pore Pressure
     5 Layer 5 (ML)
          117.00 Unit Weight
          Conventional Shear
              200.00      15.00
          Piezometric Line
          1
     6 Layer 6 (CH)
          100.00 Unit Weight
          Conventional Shear
              200.00        .00
          No Pore Pressure
     7 Layer 7 (CH)
          100.00 Unit Weight
          Linear Increase
              217.00       8.10
          No Pore Pressure
```

```
     8 Layer 8 (CH)
          100.00 Unit Weight
          Linear Increase
              374.00      8.30
          No Pore Pressure
     9 Layer 9 (CH)
          100.00 Unit Weight
          Linear Increase
              590.00      8.00
          No Pore Pressure
     10 Compacted Fill - Above T-Wall Base
          .00 Unit Weight
          Conventional Shear
                 .00       .00
          No Pore Pressure

PIEZOMETRIC LINES
         1      62.40 Water Level
                 .00     18.00
              138.50     18.00
              138.51     -1.00
              157.00     -1.00
              375.00     -1.00

         2      62.40 Piezometeric levels in ML
                 .00     18.00
              149.50     18.00
              161.00     18.00
              163.50      1.00
              167.00      1.00
              173.00     -1.00
              375.00     -1.00


DISTRIBUTED LOADS
      1
LINE LOAD
  1 138.5 -11.75 -17480. 0 1

REINFORCEMENT LINES
              1       .00         2
100.00      -100.0      0      0.
140.75      -5.000  0   0

              2        .00        2
145.75      -5.000         0      0.
182.55      -92.00         0.     0.

              3        .00        2
151.25      -5.000       0.     0.
188.05      -92.00       0.     0.

              4        .00        2
156.75      -5.000           0.    0.
193.55      -92.0       0.    0.

              5        .00        2
```

```
162.25  -5.000       0.   0.
199.30  -92.00       0.   0.

              6        .00        1
142.875     -5.00 0.0   0.0
142.875     -37.00      0.0   0.0


ANALYSIS/COMPUTATION
       Circular
      138.67   20.77  43.77
SINgle-stage Computations
RIGht Face of Slope
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

UPDATED 23 OCT 07

Attachment 3 Structural Loads for CPGA and Group Analyses

US Army Corps of Engineers Saint Paul Distict	PROJECT TITLE: T-Wall Design Example	COMPUTED BY: KDH	DATE: 07/31/07	SHEET:
	SUBJECT TITLE: Water at El. 18', Pervious	CHECKED BY:	DATE:	

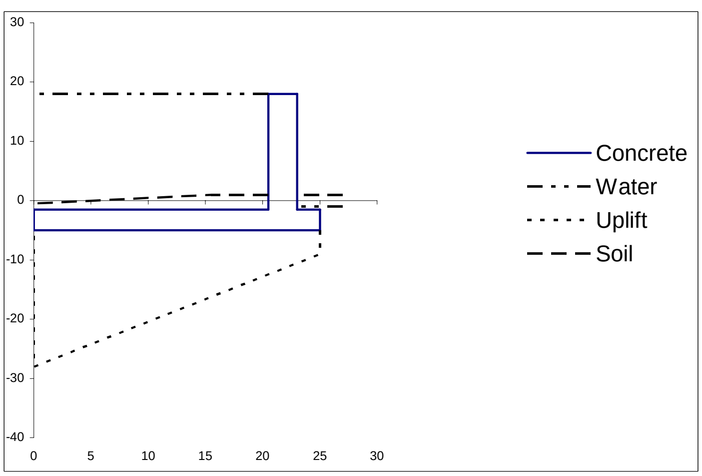
Input for CPGA pile analysis **Pervious Foundation Assumption**

Upstream Water Elevation	*18*	ft	Back Fill Soil Elevation	*1*	ft
Downstream Water Elevation	*-1*	ft	Front Fill Soil Elevation	*1*	ft
Wall Top Elevation	*18*	ft	Gamma Water	*0.0625*	kcf
Structure Bottom Elevation	*-5*	ft	Gamma Concrete	*0.15*	kcf
Base Width	*25*	ft	Gamma Sat. Backfill	*0.110*	kcf
Toe Width	*2*	ft	Distance to Backfill Break	*5.0*	ft
Wall Thickness	*2.5*	ft	Slope of Back Fill	*0.18*	
Base Thickness	*3.5*	ft	Soil Elevation at Heel	-0.50	ft

Vertical Forces								
Component	Height	x1	x2	Gamma	Force	Arm	Moment	
Stem Concrete	19.5	20.5	23	0.15	7.31	21.75	159.0	
Heel Concrete	3.5	0	23	0.15	12.08	11.5	138.9	
Toe Concrete	3.5	23	25	0.15	1.05	24	25.2	
Heel Water	17	0	20.5	0.0625	21.78	10.25	223.3	
Toe Water	0.5	23	25	0.0625	0.06	24	1.5	
Heel Soil	2.5	0	20.5	0.110	5.64	10.25	57.8	
-Triangle	1.50	0	15.5	-0.048	-0.55	5.17	-2.9	
Toe Soil	2.5	23	25	0.110	0.55	24	13.2	
Rect Uplift	-4	0	25	0.0625	-6.25	12.5	-78.1	
Tri Uplift	-19	0	25	0.0625	-14.84	8.3	-123.7	
Sum Vertical Forces					26.8		414.2	ft-k

Horizontal Forces								
Component	H1	H2	Gamma	Lat. Coeff.	Force	Arm	Moment	
Driving Water	18	-5	0.0625	1	16.53	7.67	126.74	
Resisting Water	-1	-5	0.0625	1	-0.50	1.33	-0.67	
Lateraral soil forces assumed equal and negligible								
Sum Horizontal Forces					16.03	7.86	126.07	ft-k

Total Structural Forces	Net Vert. Force	Arm	Moment	
About Heel	26.82	20.14	540.25	ft-k



Net Vertical Arm		
From Toe	4.86	ft

Moment About Toe
-130.3 ft-k

Model Width
5 ft

US Army Corps of Engineers / Saint Paul Distict	PROJECT TITLE: T-Wall Design Example	COMPUTED BY: KDH	DATE: 07/31/07	SHEET:
	SUBJECT TITLE: Water at El. 18', Pervious	CHECKED BY:	DATE:	

Calculation of Unbalanced Force

Unbalanced Force. F_{ub}	*17,480*	lb/ft	From UTexas Analysis
Elevation of Critical Surface	*-23.0*	ft	From UTexas Analysis
Length - Ground to Crit. Surface, Lu	22.5	ft	(assume failure surface is normal to pile)
Length - Base to Crit. Surface, Lp	18	ft	
Pile Moment of Inertia. I	*729*	in^4	HP14x73
Pile Modulus of Elasticity E	*29,000,000*	lb/in^2	
Soil Modulus of Subgrade Reaction, k	*100*	lb/in^2	
Soil Stiffness Parameter, R	121	in	$(EI / k)^{1/4}$
Equivalent Unbalanced Force	13,273	lb/ft	$F_{ub} * (L_u/2 + R) / (L_p + R)$

CPGA Input

PX	-146.52	kips
PY		
PZ	134.11	kips
MX	0	
MY	-651.61	kip-ft
MZ	0	

Group Input

4 Pile Rows Parallel to Wall Face

Unbalanced Loading on Piles for Group Analysis

Total	324	lb/in	F_{ub} * Model Width /L_u
50%	162	lb/in	For Pile Row on Flood Side
17%	54	lb/in	

Note: Applied to length of pile from bottom of cap to top of critical surface. 18

Unbalanced Loads on Wall for Group Analysis of Just Unbalanced Forces

Distance From Base to Ground Surface, Ds 4.50 ft

PX	0	lb	
PY	17,480	lb	F_{ub} * Model Width / L_u * Ds
PZ	0	lb	
MX	0		
MY	0		
MZ	-471,960	lb-in	-PZ * Ds/2

Total Loads for Group Analysis

PX	134,114	lb	
PY	97,636	lb	PYub + Sum Horizontal * Model Width
PZ	0	lb	
MX	0		
MY	0		
MZ	7,347,343	lb-in	

US Army Corps of Engineers Saint Paul Distict	PROJECT TITLE: T-Wall Design Example	COMPUTED BY: KDH	DATE: 07/31/07	SHEET:
	SUBJECT TITLE: Water at El. 18', Impervious	CHECKED BY:	DATE:	

Input for CPGA pile analysis **Impervious Foundation Assumption**

Upstream Water Elevation	*18*	ft	Back Fill Soil Elevation	*1*	ft
Downstream Water Elevation	*-1*	ft	Front Fill Soil Elevation	*1*	ft
Wall Top Elevation	*18*	ft	Gamma Water	*0.0625*	kcf
Structure Bottom Elevation	*-5*	ft	Gamma Concrete	*0.15*	kcf
Base Width	*25*	ft	Gamma Sat. Backfill	*0.110*	kcf
Toe Width	*2*	ft	Distance to Backfill Break	*5.0*	ft
Wall Thickness	*2.5*	ft	Slope of Back Fill	*0.18*	
Base Thickness	*3.5*	ft	Soil Elevation at Heel	-0.50	ft

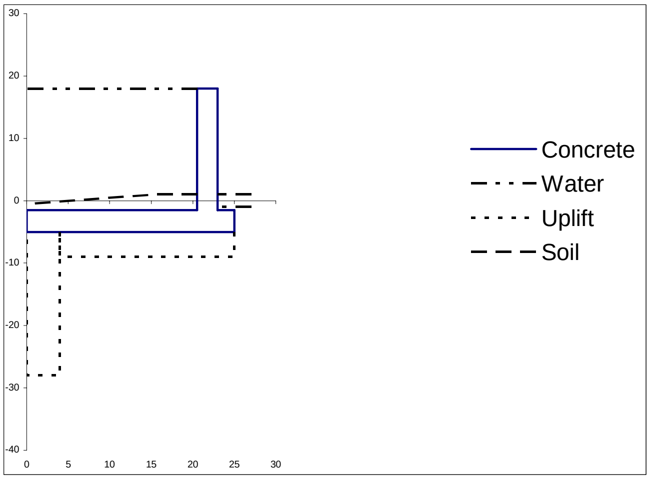
Vertical Forces

Component	Height	x1	x2	Gamma	Force	Arm	Moment	
Stem Concrete	19.5	20.5	23	0.15	7.31	21.75	159.0	
Heel Concrete	3.5	0	23	0.15	12.08	11.5	138.9	
Toe Concrete	3.5	23	25	0.15	1.05	24	25.2	
Heel Water	17	0	20.5	0.0625	21.78	10.25	223.3	
Toe Water	0.5	23	25	0.0625	0.06	24	1.5	
Heel Soil	2.5	0	20.5	0.110	5.64	10.25	57.8	
-Triangle	1.50	0	15.5	-0.048	-0.55	5.17	-2.9	
Toe Soil	2.5	23	25	0.110	0.55	24	13.2	
Prot. Side Uplift	-4	*4*	25	0.0625	-5.25	14.5	-76.1	
Flood Side Uplift	-23	0	4	0.0625	-5.75	2	-11.5	
Sum Vertical Forces					36.9	kip	528.4	ft-k

Horizontal Forces

Component	H1	H2	Gamma	Lat. Coeff.	Force	Arm	Moment	
Driving Water	18	-5	0.0625	1	16.53	7.67	126.74	
Resisting Water	-1	-5	0.0625	1	-0.50	1.33	-0.67	
Lateraral soil forces assumed equal and negligible								
Sum Horizontal Forces					16.03	kip	126.07	ft-k

Total Structural Forces	Net Vert. Force	Arm	Moment	
About Heel	36.92	17.73	654.45	ft-k



Net Vertical Arm		
From Toe	7.27	ft

Moment About Toe
-268.5 ft-k

Model Width
5 ft

UPDATED 23 OCT 07

US Army Corps of Engineers Saint Paul Distict	PROJECT TITLE: T-Wall Design Example	COMPUTED BY: KDH	DATE: 07/31/07	SHEET:	
	SUBJECT TITLE: Water at El. 18', Impervious	CHECKED BY:	DATE:		

Calculation of Unbalanced Force

Unbalanced Force. F_{ub}	*17,480*	lb/ft	From UTexas Analysis
Elevation of Critical Surface	*-23*	ft	From UTexas Analysis
Length - Ground to Crit. Surface, Lu	22.5	ft	(assume failure surface is normal to pile)
Length - Base to Crit. Surface, Lp	18	ft	
Pile Moment of Inertia. I	*729*	in^4	HP14x73
Pile Modulus of Elasticity E	*29,000,000*	lb/in^2	
Soil Modulus of Subgrade Reaction, k	*100*	lb/in^2	
Soil Stiffness Parameter, R	121	in	$(EI / k)^{1/4}$
Equivalent Unbalanced Force	13,273	lb/ft	$F_{ub} * (L_u/2 +R) / (L_p +R)$

CPGA Input

PX	-146.52	kips
PY		
PZ	184.58	kips
MX	0	
MY	-1,342.34	kip-ft
MZ	0	

Group Input

4 Pile Rows Parallel to Wall Face

Unbalanced Loading on Piles for Group Analysis

Total	324	lb/in	F_{ub} * Model Width /L_u
50%	162	lb/in	For Pile on Protected Side
17%	54	lb/in	

Note: Applied to length of pile from bottom of cap to top of critical surface. 18 ft

Unbalanced Loads on Wall for Group Analysis of Just Unbalanced Forces

Distance From Base to Ground Surface, Ds 4.50 ft

PX	0	lb	
PY	17,480	lb	F_{ub} * Model Width / L_u * Ds
PZ	0	lb	
MX	0		
MY	0		
MZ	-471,960	lb-in	-PZ * Ds/2

Total Loads for Group Analysis

PX	184,583	lb	
PY	97,636	lb	PYub + Sum Horizontal * Model Width
PZ	0	lb	
MX	0		
MY	0		
MZ	15,636,093	lb-in	

Attachment 4 - Preliminary Analysis with CPGA

Input File:

```
10 T-wall Example, Water on FS 18, Group Reducton Test - with group
15 3.5 ft slab, hp 14 x 73 piles, pinned head, 2.5:1 batter
20 PROP 29000 261 729 21.4 1.0 0 all
30 SOIL ES 0.00001 "TIP" 87.5 0 1 2 3
32 SOIL ES 0.00001 "TIP" 87.5 0 4
37 SOIL ES 0.00001 "TIP" 105.0 0 5
40 PIN all
50 ALLOW H 111.0 113.0 315.8  315.8  520.6  1573.1 all
70 BATTER 2.5 all
80 ANGLE 180 1 2 3 4
180 PILE 1  1.250 0.00 0.00
201 PILE 2  6.75 0.00 0.00
202 PILE 3  12.25 0.00 0.00
203 PILE 4  17.75 0.00 0.00
205 PILE 5  23.75 0.00 0.00
230 LOAD 1 -146.52  0.0  134.11  0.00  -651.61
255 LOAD 2 -146.52  0.0  184.58  0.00 -1342.34
334 FOUT 1 2 3 4 5 6 7 MVN18G5.out
335 PFO ALL
```

Output:

```
 *********************************
 * CASE PROGRAM    #  X0080       *  CPGA - CASE PILE GROUP ANALYSIS PROGRAM
 * VERSION NUMBER # 1993/03/29   *  RUN DATE 31-JUL-2007   RUN TIME 16.36.10
 *********************************


 T-WALL EXAMPLE, WATER ON FS 18, GROUP REDUCTON TEST - WITH GROUP


 THERE ARE     5 PILES AND
               2 LOAD CASES IN THIS RUN.

 ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                    X          Y          Z
                                  -----      -----      -----
 WITH DIAGONAL COORDINATES = (      1.25 ,      .00 ,      .00 )
                             (     23.75 ,      .00 ,      .00 )


 ********************************************************************************

          PILE PROPERTIES AS INPUT


       E            I1            I2            A           C33          B66
      KSI         IN**4         IN**4         IN**2
   .29000E+05   .26100E+03   .72900E+03   .21400E+02   .10000E+01   .00000E+00

 THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -
```

```
    ALL

********************************************************************************

         SOIL DESCRIPTIONS AS INPUT


   ES      ESOIL      LENGTH       L            LU
         K/IN**2                   FT           FT
         .10000E-04    T       .87500E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   1    2    3


   ES      ESOIL      LENGTH       L            LU
         K/IN**2                   FT           FT
         .10000E-04    T       .87500E+02    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   4


   ES      ESOIL      LENGTH       L            LU
         K/IN**2                   FT           FT
         .10000E-04    T       .10500E+03    .00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

   5

********************************************************************************

         PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM        X          Y          Z      BATTER   ANGLE   LENGTH  FIXITY
          FT         FT         FT                        FT

   1      1.25        .00        .00     2.50   180.00    94.24    P
   2      6.75        .00        .00     2.50   180.00    94.24    P
   3     12.25        .00        .00     2.50   180.00    94.24    P
   4     17.75        .00        .00     2.50   180.00    94.24    P
   5     23.75        .00        .00     2.50      .00   113.09    P
                                                         ------
                                                         490.05

********************************************************************************

                        APPLIED LOADS

LOAD     PX        PY        PZ          MX          MY          MZ
CASE      K         K         K         FT-K        FT-K        FT-K
```

```
  1    -146.5         .0     134.1          .0      -651.6           .0
  2    -146.5         .0     184.6          .0     -1342.3           .0


*******************************************************************************

         ORIGINAL PILE GROUP STIFFNESS MATRIX

  .36589E+03   .26469E-04  -.59923E+03   .00000E+00   .41347E+05   .30175E-02
  .26469E-04   .32977E-01  -.66172E-04   .00000E+00   .75436E-02   .48872E+01
 -.59923E+03  -.66172E-04   .22866E+04   .00000E+00  -.32808E+06  -.75436E-02
  .00000E+00   .00000E+00   .00000E+00   .00000E+00   .00000E+00   .00000E+00
  .41347E+05   .75436E-02  -.32808E+06   .00000E+00   .66918E+08   .12203E+01
  .30175E-02   .48872E+01  -.75436E-02   .00000E+00   .12203E+01   .10222E+04


S(4,4)=0.  PROBLEM WILL BE TREATED AS TWO DIMENSIONAL IN THE X-Z PLANE.

LOAD CASE    1.  NUMBER OF FAILURES =    2.  NUMBER OF PILES IN TENSION =    1.

LOAD CASE    2.  NUMBER OF FAILURES =    1.  NUMBER OF PILES IN TENSION =    1.

*******************************************************************************

         PILE CAP DISPLACEMENTS

LOAD
CASE       DX          DZ          R
           IN          IN         RAD

   1   -.7899E+00  -.3207E+00  -.1201E-02
   2   -.6897E+00  -.2476E+00  -.1028E-02

*******************************************************************************
              ELASTIC CENTER INFORMATION


ELASTIC CENTER IN PLANE X-Z          X             Z
                                    FT            FT
                                  16.62        -17.81

LOAD    MOMENT IN
CASE    X-Z PLANE
   1  .70738E+07
   2  .29723E+08

*******************************************************************************

         PILE FORCES IN LOCAL GEOMETRY

             M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
```

```
* INDICATES PILE FAILURE
# INDICATES CBF BASED ON MOMENTS DUE TO
          (F3*EMIN) FOR CONCRETE PILES
B INDICATES BUCKLING CONTROLS
```

LOAD CASE - 1

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF	
1	.0	.0	6.8	.0	-4.0	.0	.06	.02	
2	.0	.0	47.2	.0	-3.8	.0	.42	.15	
3	.0	.0	87.6	.0	-3.7	.0	.79	.28	
4	.0	.0	127.9	.0	-3.5	.0	1.15	.41	*
5	.0	.0	-125.0	.0	3.5	.0	1.11	.40	*

LOAD CASE - 2

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF	
1	.0	.0	22.3	.0	-3.4	.0	.20	.07	
2	.0	.0	56.9	.0	-3.3	.0	.51	.18	
3	.0	.0	91.4	.0	-3.2	.0	.82	.29	
4	.0	.0	126.0	.0	-3.0	.0	1.14	.40	*
5	.0	.0	-97.8	.0	3.1	.0	.87	.31	

****

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	-2.5	.0	6.3	.0	.0	.0
2	-17.5	.0	43.8	.0	.0	.0
3	-32.5	.0	81.3	.0	.0	.0
4	-47.5	.0	118.8	.0	.0	.0
5	-46.4	.0	-116.0	.0	.0	.0

LOAD CASE - 2

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	-8.3	.0	20.7	.0	.0	.0
2	-21.1	.0	52.8	.0	.0	.0
3	-34.0	.0	84.9	.0	.0	.0

4	-46.8	.0	117.0	.0	.0	.0
5	-36.3	.0	-90.8	.0	.0	.0

Attachment 5. Group 7 Output File for Pervious Condition

```
==============================================================================

                GROUP for Windows, Version 7.0.7

                 Analysis of A Group of Piles
              Subjected to Axial and Lateral Loading

               (c) Copyright ENSOFT, Inc., 1987-2006
                     All Rights Reserved

==============================================================================


This program is licensed to:

k
c

Path to file locations:      C:\KDH\New Orleans\T-walls\Group\
Name of input data file:     18 pervious Example.gpd
Name of output file:         18 pervious Example.gpo
Name of plot output file:    18 pervious Example.gpp
Name of runtime file:        18 pervious Example.gpr
Name of output summary file: 18 pervious Example.gpt


------------------------------------------------------------------------------
                          Time and Date of Analysis
------------------------------------------------------------------------------

               Date:  July 31, 2007     Time:  14:43: 5
 PILE GROUP ANALYSIS PROGRAM-GROUP
 PC VERSION 6.0 (C) COPYRIGHT ENSOFT,INC. 2000

 THE PROGRAM WAS COMPILED USING MICROSOFT FORTRAN
 POWERSTATION 4.0 (C) COPYRIGHT MICROSOFT CORPORATION, 1996.



     T-wall Examplel : F.S. 18.0, P.S. -1.0,  Pervious Foundation Condition



                *****     INPUT INFORMATION     *****




     * TABLE C *  LOAD AND CONTROL PARAMETERS


     UNITS--

          V LOAD,LBS     H LOAD,LBS    MOMENT,LBS-IN
```

```
   0.1341E+06     0.9764E+05     0.7347E+07

GROUP NO. 1


             DISTRIBUTED LOAD CURVE          2 POINTS

                        X,IN       LOAD,LBS/IN
                        0.00         0.527E+02
                      216.00         0.527E+02

GROUP NO. 2


             DISTRIBUTED LOAD CURVE          2 POINTS

                        X,IN       LOAD,LBS/IN
                        0.00         0.527E+02
                      216.00         0.527E+02

GROUP NO. 3


             DISTRIBUTED LOAD CURVE          2 POINTS

                        X,IN       LOAD,LBS/IN
                        0.00         0.527E+02
                      216.00         0.527E+02

GROUP NO. 4


             DISTRIBUTED LOAD CURVE          2 POINTS

                        X,IN       LOAD,LBS/IN
                        0.00         0.606E+02
                      216.00         0.606E+02

GROUP NO. 5


             DISTRIBUTED LOAD CURVE          2 POINTS

                        X,IN       LOAD,LBS/IN
                        0.00         0.105E+03
                      216.00         0.105E+03


* THE LOADING IS STATIC *


  KPYOP =  0      (CODE TO GENERATE P-Y CURVES)

  ( KPYOP = 1 IF P-Y YES; = 0 IF P-Y NO; = -1 IF P-Y ONLY )
```

```
* CONTROL PARAMETERS *
    TOLERANCE ON CONVERGENCE OF FOUNDATION REACTION      =  0.100E-04 IN
    TOLERANCE ON DETERMINATION OF DEFLECTIONS            =  0.100E-04 IN
    MAX NO OF ITERATIONS ALLOWED FOR FOUNDATION ANALYSIS =     100
    MAXIMUM NO. OF ITERATIONS ALLOWED FOR PILE ANALYSIS  =     100



* TABLE D *   ARRANGEMENT OF PILE GROUPS

  GROUP  CONNECT  NO OF PILE PILE NO  L-S CURVE  P-Y CURVE
    1      PIN         1        1         1           0
    2      PIN         1        1         1           0
    3      PIN         1        1         1           0
    4      PIN         1        1         1           0
    5      PIN         1        2         2           0

  GROUP          VERT,IN     HOR,IN    SLOPE,IN/IN  GROUND,IN SPRING,LBS-IN
    1         0.0000E+00 -0.1500E+02  0.3805E+00 -0.3600E+02  0.0000E+00
    2         0.0000E+00 -0.8100E+02  0.3805E+00 -0.3600E+02  0.0000E+00
    3         0.0000E+00 -0.1470E+03  0.3805E+00 -0.3600E+02  0.0000E+00
    4         0.0000E+00 -0.2130E+03  0.3805E+00 -0.3600E+02  0.0000E+00
    5         0.0000E+00 -0.2850E+03 -0.3805E+00 -0.3600E+02  0.0000E+00
    6         0.0000E+00  0.0000E+00  0.0000E+00  0.0000E+00  0.0000E+00



* TABLE E *   PILE GEOMETRY AND PROPERTIES
              PILE TYPE = 1 - DRIVEN PILE
                        = 2 - DRILLED SHAFT

  PILE   SEC  INC       LENGTH, IN     E   ,LBS/IN**2 PILE TYPE
    1     1    94       0.1124E+04     0.2900E+08        1
    2     1    94       0.1357E+04     0.2900E+08        1

  PILE    FROM,IN      TO,IN      DIAM,IN   AREA,IN**2    I,IN**4

    1  0.0000E+00  0.1124E+04  0.1400E+02  0.2140E+02  0.7290E+03

      * THE PILE ABOVE IS OF LINEARLY ELASTIC MATERIAL *

    2  0.0000E+00  0.1357E+04  0.1400E+02  0.2140E+02  0.7290E+03

      * THE PILE ABOVE IS OF LINEARLY ELASTIC MATERIAL *



* TABLE F *   AXIAL LOAD VS SETTLEMENT

(THE LOAD-SETTLEMENT CURVE OF SINGLE PILE IS GENERATED INTERNALLY)

  NUM OF CURVES  2

   CURVE   1          NUM OF POINTS = 19

       POINT       AXIAL LOAD,LBS      SETTLEMENT, IN
```

1	-0.1891E+06	-0.2251E+01
2	-0.1787E+06	-0.1234E+01
3	-0.1735E+06	-0.7251E+00
4	-0.1415E+06	-0.2707E+00
5	-0.1307E+06	-0.2010E+00
6	-0.4273E+05	-0.5355E-01
7	-0.2066E+05	-0.2609E-01
8	-0.4091E+04	-0.5188E-02
9	-0.4091E+03	-0.5188E-03
10	0.0000E+00	0.0000E+00
11	0.7980E+03	0.9819E-03
12	0.4913E+04	0.6167E-02
13	0.2352E+05	0.2946E-01
14	0.4697E+05	0.5852E-01
15	0.1339E+06	0.2068E+00
16	0.1454E+06	0.2779E+00
17	0.1824E+06	0.7411E+00
18	0.1908E+06	0.1256E+01
19	0.2052E+06	0.2280E+01

CURVE 2 NUM OF POINTS = 19

POINT	AXIAL LOAD,LBS	SETTLEMENT, IN
1	-0.2895E+06	-0.2450E+01
2	-0.2689E+06	-0.1413E+01
3	-0.2586E+06	-0.8941E+00
4	-0.1956E+06	-0.3808E+00
5	-0.1747E+06	-0.2904E+00
6	-0.7760E+05	-0.9714E-01
7	-0.3898E+05	-0.4799E-01
8	-0.7512E+04	-0.9355E-02
9	-0.7512E+03	-0.9355E-03
10	0.0000E+00	0.0000E+00
11	0.7529E+03	0.9375E-03
12	0.7529E+04	0.9375E-02
13	0.3907E+05	0.4810E-01
14	0.7775E+05	0.9734E-01
15	0.1749E+06	0.2908E+00
16	0.1960E+06	0.3816E+00
17	0.2594E+06	0.8961E+00
18	0.2701E+06	0.1415E+01
19	0.2908E+06	0.2453E+01

* TABLE H * SOIL DATA FOR AUTO P-Y CURVES

SOILS INFORMATION

AT THE GROUND SURFACE = -36.00 IN

6 LAYER(S) OF SOIL

LAYER 1
THE SOIL IS A SOFT CLAY
X AT THE TOP OF THE LAYER = -36.00 IN

```
X AT THE BOTTOM OF THE LAYER      =     216.00 IN
MODULUS OF SUBGRADE REACTION      =  0.100E+00 LBS/IN**3

LAYER  2
THE SOIL IS A SILT
X AT THE TOP OF THE LAYER         =     216.00 IN
X AT THE BOTTOM OF THE LAYER      =     252.00 IN
MODULUS OF SUBGRADE REACTION      =  0.300E+02 LBS/IN**3

LAYER  3
THE SOIL IS A SOFT CLAY
X AT THE TOP OF THE LAYER         =     252.00 IN
X AT THE BOTTOM OF THE LAYER      =     720.00 IN
MODULUS OF SUBGRADE REACTION      =  0.300E+02 LBS/IN**3

LAYER  4
THE SOIL IS A STIFF CLAY BELOW THE WATER TABLE
X AT THE TOP OF THE LAYER         =     720.00 IN
X AT THE BOTTOM OF THE LAYER      =     973.00 IN
MODULUS OF SUBGRADE REACTION      =  0.100E+03 LBS/IN**3

LAYER  5
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER         =     973.00 IN
X AT THE BOTTOM OF THE LAYER      =    1273.00 IN
MODULUS OF SUBGRADE REACTION      =  0.600E+02 LBS/IN**3

LAYER  6
THE SOIL IS A STIFF CLAY BELOW THE WATER TABLE
X AT THE TOP OF THE LAYER         =    1273.00 IN
X AT THE BOTTOM OF THE LAYER      =    1600.00 IN
MODULUS OF SUBGRADE REACTION      =  0.100E+03 LBS/IN**3


DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                  16 POINTS

            X,IN   WEIGHT,LBS/IN**3
        -36.0000     0.1010E-01
        108.0000     0.1010E-01
        108.0000     0.2170E-01
        216.0000     0.2170E-01
        216.0000     0.3150E-01
        252.0000     0.3150E-01
        252.0000     0.2170E-01
        720.0000     0.2170E-01
        720.0000     0.2750E-01
        900.0000     0.2750E-01
        900.0000     0.3330E-01
        972.0000     0.3330E-01
        972.0000     0.3440E-01
       1273.0000     0.3440E-01
       1273.0000     0.3210E-01
       1600.0000     0.3210E-01


DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
```

16 POINTS

X IN	C LBS/IN**2	PHI,DEGREES	E50	FMAX LBS/IN**2	TIPMAX LBS/IN**2
-36.00	0.1000E-04	0.000	0.2500E-01	0.0000E+00	0.0000E+00
216.00	0.1000E-04	0.000	0.2500E-01	0.0000E+00	0.0000E+00
216.00	0.1390E+01	15.000	0.2500E-01	0.2400E+01	0.0000E+00
252.00	0.1390E+01	15.000	0.2500E-01	0.2700E+01	0.0000E+00
252.00	0.1390E+01	0.000	0.2500E-01	0.1390E+01	0.0000E+00
408.00	0.1390E+01	0.000	0.2500E-01	0.1390E+01	0.0000E+00
408.00	0.2590E+01	0.000	0.2000E-01	0.2590E+01	0.0000E+00
720.00	0.4100E+01	0.000	0.1000E-01	0.4100E+01	0.0000E+00
720.00	0.4100E+01	0.000	0.1000E-01	0.4100E+01	0.0000E+00
780.00	0.4300E+01	0.000	0.1000E-01	0.4300E+01	0.0000E+00
780.00	0.5500E+01	0.000	0.1000E-01	0.5500E+01	0.0000E+00
973.00	0.5500E+01	0.000	0.1000E-01	0.5500E+01	0.0000E+00
973.00	0.0000E+00	30.000	0.0000E+00	0.1300E+02	0.0000E+00
1273.00	0.0000E+00	30.000	0.0000E+00	0.1400E+02	0.0000E+00
1273.00	0.6800E+01	0.000	0.1000E-01	0.6800E+01	0.0000E+00
1600.00	0.6800E+01	0.000	0.1000E-01	0.6800E+01	0.0000E+00

REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS

GROUP NO	P-FACTOR	Y-FACTOR
1	1.00	1.00
2	0.87	1.00
3	0.87	1.00
4	0.87	1.00
5	0.89	1.00

T-wall Examplel : F.S. 18.0, P.S. -1.0, Pervious Foundation Condition

***** COMPUTATION RESULTS *****

VERT. LOAD, LBS	HORI. LOAD, LBS	MOMENT,IN-LBS
0.1341E+06	0.9764E+05	0.7347E+07

DISPLACEMENT OF GROUPED PILE FOUNDATION

VERTICAL,IN	HORIZONTAL,IN	ROTATION,RAD
-0.2120E+00	0.5254E+00	0.8644E-03

NUMBER OF ITERATIONS = 4

```
 * TABLE I *   COMPUTATION ON INDIVIDUAL PILE

   * PILE GROUP *  1

 PILE TOP DISPLACEMENTS AND REACTIONS

       THE GLOBAL STRUCTURE COORDINATE SYSTEM
       --------------------------------------

 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

-0.199E+00  0.525E+00 -.408E-03 0.978E+04-0.237E+04 0.000E+00   0.383E+03

       THE LOCAL MEMBER COORDINATE SYSTEM
       ------------------------------------

 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

 0.103E-01  0.562E+00 -.408E-03 0.820E+04-0.584E+04 0.000E+00   0.383E+03

          LATERALLY LOADED PILE
```

X	DEFLECTION	MOMENT	SHEAR	SOIL REACTION	TOTAL STRESS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS	LBS/IN	LBS/IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
0.00	0.562E+00	0.000E+00	-0.521E+04	0.543E-03	0.383E+03	0.211E+11
11.96	0.557E+00	0.622E+05	-0.489E+04	0.542E-03	0.981E+03	0.211E+11
23.91	0.552E+00	0.117E+06	-0.426E+04	0.540E-03	0.151E+04	0.211E+11
35.87	0.545E+00	0.164E+06	-0.363E+04	0.538E-03	0.196E+04	0.211E+11
47.83	0.538E+00	0.204E+06	-0.300E+04	0.536E-03	0.234E+04	0.211E+11
59.79	0.530E+00	0.236E+06	-0.237E+04	0.533E-03	0.265E+04	0.211E+11
71.74	0.520E+00	0.260E+06	-0.174E+04	0.530E-03	0.288E+04	0.211E+11
83.70	0.508E+00	0.277E+06	-0.111E+04	0.525E-03	0.304E+04	0.211E+11
95.66	0.494E+00	0.287E+06	-0.481E+03	0.521E-03	0.313E+04	0.211E+11
107.62	0.478E+00	0.288E+06	0.149E+03	0.515E-03	0.315E+04	0.211E+11
119.57	0.460E+00	0.283E+06	0.779E+03	0.509E-03	0.310E+04	0.211E+11
131.53	0.441E+00	0.269E+06	0.141E+04	0.501E-03	0.297E+04	0.211E+11
143.49	0.419E+00	0.249E+06	0.204E+04	0.493E-03	0.277E+04	0.211E+11
155.45	0.396E+00	0.220E+06	0.267E+04	0.484E-03	0.250E+04	0.211E+11
167.40	0.372E+00	0.184E+06	0.330E+04	0.474E-03	0.215E+04	0.211E+11
179.36	0.346E+00	0.141E+06	0.393E+04	0.462E-03	0.174E+04	0.211E+11
191.32	0.319E+00	0.900E+05	0.456E+04	0.450E-03	0.125E+04	0.211E+11
203.28	0.292E+00	0.315E+05	0.519E+04	0.437E-03	0.686E+03	0.211E+11
215.23	0.264E+00	-0.346E+05	0.582E+04	0.423E-03	0.715E+03	0.211E+11
227.19	0.237E+00	-0.108E+06	0.566E+04	0.795E+02	0.142E+04	0.211E+11
239.15	0.210E+00	-0.170E+06	0.436E+04	0.138E+03	0.202E+04	0.211E+11
251.11	0.184E+00	-0.213E+06	0.249E+04	0.175E+03	0.243E+04	0.211E+11
263.06	0.160E+00	-0.230E+06	0.133E+04	0.200E+02	0.260E+04	0.211E+11

```
     275.02  0.138E+00 -0.245E+06  0.108E+04  0.220E+02  0.274E+04  0.211E+11
     286.98  0.117E+00 -0.257E+06  0.806E+03  0.238E+02  0.285E+04  0.211E+11
     298.94  0.979E-01 -0.265E+06  0.513E+03  0.251E+02  0.292E+04  0.211E+11
     310.89  0.806E-01 -0.269E+06  0.207E+03  0.261E+02  0.297E+04  0.211E+11
     322.85  0.651E-01 -0.270E+06 -0.109E+03  0.267E+02  0.298E+04  0.211E+11
     334.81  0.514E-01 -0.267E+06 -0.430E+03  0.269E+02  0.295E+04  0.211E+11
     346.77  0.395E-01 -0.260E+06 -0.751E+03  0.267E+02  0.288E+04  0.211E+11

     NUMBER OF ITERATIONS IN LLP =    18


       * PILE GROUP *   2


 PILE TOP DISPLACEMENTS AND REACTIONS


       THE GLOBAL STRUCTURE COORDINATE SYSTEM
       --------------------------------------

 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

-0.142E+00  0.525E+00 -.879E-04 0.485E+05 0.128E+05 0.000E+00   0.232E+04


       THE LOCAL MEMBER COORDINATE SYSTEM
       ------------------------------------

 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

 0.633E-01  0.541E+00 -.879E-04 0.497E+05-0.611E+04 0.000E+00   0.232E+04


          LATERALLY LOADED PILE


        X  DEFLECTION    MOMENT     SHEAR      SOIL       TOTAL    FLEXURAL
                                              REACTION    STRESS   RIGIDITY
       IN      IN        LBS-IN      LBS      LBS/IN   LBS/IN**2  LBS-IN**2
      ***** ********** ********** ********** ********** ********** **********
       0.00  0.541E+00  0.000E+00 -0.548E+04  0.464E-03  0.232E+04  0.211E+11
      11.96  0.540E+00  0.655E+05 -0.517E+04  0.464E-03  0.295E+04  0.211E+11
      23.91  0.538E+00  0.123E+06 -0.454E+04  0.464E-03  0.351E+04  0.211E+11
      35.87  0.536E+00  0.174E+06 -0.391E+04  0.463E-03  0.399E+04  0.211E+11
      47.83  0.532E+00  0.217E+06 -0.328E+04  0.462E-03  0.440E+04  0.211E+11
      59.79  0.527E+00  0.252E+06 -0.265E+04  0.460E-03  0.474E+04  0.211E+11
      71.74  0.521E+00  0.279E+06 -0.202E+04  0.458E-03  0.501E+04  0.211E+11
      83.70  0.512E+00  0.299E+06 -0.139E+04  0.456E-03  0.520E+04  0.211E+11
      95.66  0.501E+00  0.312E+06 -0.758E+03  0.453E-03  0.532E+04  0.211E+11
     107.62  0.489E+00  0.316E+06 -0.128E+03  0.449E-03  0.536E+04  0.211E+11
     119.57  0.474E+00  0.313E+06  0.502E+03  0.444E-03  0.533E+04  0.211E+11
     131.53  0.457E+00  0.303E+06  0.113E+04  0.439E-03  0.523E+04  0.211E+11
     143.49  0.438E+00  0.285E+06  0.176E+04  0.433E-03  0.506E+04  0.211E+11
     155.45  0.417E+00  0.259E+06  0.239E+04  0.426E-03  0.481E+04  0.211E+11
     167.40  0.394E+00  0.225E+06  0.302E+04  0.418E-03  0.449E+04  0.211E+11
```

179.36	0.370E+00	0.184E+06	0.365E+04	0.409E-03	0.409E+04	0.211E+11
191.32	0.345E+00	0.135E+06	0.428E+04	0.400E-03	0.362E+04	0.211E+11
203.28	0.318E+00	0.790E+05	0.491E+04	0.389E-03	0.308E+04	0.211E+11
215.23	0.291E+00	0.152E+05	0.554E+04	0.378E-03	0.247E+04	0.211E+11
227.19	0.264E+00	-0.562E+05	0.540E+04	0.768E+02	0.286E+04	0.211E+11
239.15	0.238E+00	-0.117E+06	0.425E+04	0.116E+03	0.344E+04	0.211E+11
251.11	0.212E+00	-0.160E+06	0.268E+04	0.146E+03	0.386E+04	0.211E+11
263.06	0.187E+00	-0.183E+06	0.170E+04	0.182E+02	0.408E+04	0.211E+11
275.02	0.164E+00	-0.203E+06	0.147E+04	0.202E+02	0.428E+04	0.211E+11
286.98	0.142E+00	-0.221E+06	0.122E+04	0.219E+02	0.444E+04	0.211E+11
298.94	0.121E+00	-0.235E+06	0.948E+03	0.233E+02	0.458E+04	0.211E+11
310.89	0.102E+00	-0.245E+06	0.662E+03	0.244E+02	0.468E+04	0.211E+11

NUMBER OF ITERATIONS IN LLP = 18

* PILE GROUP * 3

PILE TOP DISPLACEMENTS AND REACTIONS

THE GLOBAL STRUCTURE COORDINATE SYSTEM

XDISPL,IN	YDISPL,IN	SLOPE	AXIAL,LBS	LAT,LBS	BM,LBS-IN	STRESS,LBS/IN**2
-0.850E-01	0.525E+00	-.116E-05	0.773E+05	0.243E+05	0.000E+00	0.378E+04

THE LOCAL MEMBER COORDINATE SYSTEM

XDISPL,IN	YDISPL,IN	SLOPE	AXIAL,LBS	LAT,LBS	BM,LBS-IN	STRESS,LBS/IN**2
0.116E+00	0.519E+00	-.116E-05	0.808E+05	-0.617E+04	0.000E+00	0.378E+04

LATERALLY LOADED PILE

X IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
0.00	0.519E+00	0.000E+00	-0.554E+04	0.458E-03	0.378E+04	0.211E+11
11.96	0.519E+00	0.662E+05	-0.522E+04	0.458E-03	0.441E+04	0.211E+11
23.91	0.519E+00	0.125E+06	-0.459E+04	0.458E-03	0.498E+04	0.211E+11
35.87	0.518E+00	0.176E+06	-0.396E+04	0.458E-03	0.547E+04	0.211E+11
47.83	0.515E+00	0.219E+06	-0.333E+04	0.457E-03	0.588E+04	0.211E+11
59.79	0.511E+00	0.255E+06	-0.270E+04	0.456E-03	0.623E+04	0.211E+11
71.74	0.505E+00	0.283E+06	-0.207E+04	0.454E-03	0.650E+04	0.211E+11
83.70	0.498E+00	0.303E+06	-0.144E+04	0.452E-03	0.669E+04	0.211E+11
95.66	0.488E+00	0.316E+06	-0.811E+03	0.449E-03	0.681E+04	0.211E+11

107.62	0.476E+00	0.321E+06	-0.181E+03	0.445E-03	0.686E+04	0.211E+11
119.57	0.462E+00	0.318E+06	0.449E+03	0.441E-03	0.683E+04	0.211E+11
131.53	0.446E+00	0.308E+06	0.108E+04	0.435E-03	0.673E+04	0.211E+11
143.49	0.428E+00	0.290E+06	0.171E+04	0.429E-03	0.656E+04	0.211E+11
155.45	0.408E+00	0.264E+06	0.234E+04	0.423E-03	0.631E+04	0.211E+11
167.40	0.386E+00	0.230E+06	0.297E+04	0.415E-03	0.599E+04	0.211E+11
179.36	0.362E+00	0.189E+06	0.360E+04	0.406E-03	0.559E+04	0.211E+11
191.32	0.338E+00	0.140E+06	0.423E+04	0.397E-03	0.513E+04	0.211E+11
203.28	0.312E+00	0.840E+05	0.486E+04	0.386E-03	0.458E+04	0.211E+11
215.23	0.285E+00	0.200E+05	0.549E+04	0.375E-03	0.397E+04	0.211E+11
227.19	0.259E+00	-0.516E+05	0.536E+04	0.753E+02	0.427E+04	0.211E+11
239.15	0.233E+00	-0.112E+06	0.421E+04	0.116E+03	0.486E+04	0.211E+11
251.11	0.208E+00	-0.156E+06	0.264E+04	0.147E+03	0.528E+04	0.211E+11
263.06	0.183E+00	-0.179E+06	0.165E+04	0.181E+02	0.550E+04	0.211E+11
275.02	0.160E+00	-0.200E+06	0.142E+04	0.201E+02	0.569E+04	0.211E+11
286.98	0.138E+00	-0.217E+06	0.117E+04	0.217E+02	0.586E+04	0.211E+11
298.94	0.118E+00	-0.231E+06	0.905E+03	0.232E+02	0.600E+04	0.211E+11
310.89	0.995E-01	-0.242E+06	0.621E+03	0.243E+02	0.610E+04	0.211E+11
322.85	0.825E-01	-0.249E+06	0.326E+03	0.250E+02	0.617E+04	0.211E+11
334.81	0.671E-01	-0.252E+06	0.244E+02	0.255E+02	0.620E+04	0.211E+11
346.77	0.534E-01	-0.252E+06	-0.281E+03	0.256E+02	0.619E+04	0.211E+11

NUMBER OF ITERATIONS IN LLP = 16

* PILE GROUP * 4

PILE TOP DISPLACEMENTS AND REACTIONS

THE GLOBAL STRUCTURE COORDINATE SYSTEM

XDISPL,IN	YDISPL,IN	SLOPE	AXIAL,LBS	LAT,LBS	BM,LBS-IN	STRESS,LBS/IN**2
-0.279E-01	0.525E+00	0.585E-03	0.107E+06	0.347E+05	0.000E+00	0.523E+04

THE LOCAL MEMBER COORDINATE SYSTEM

XDISPL,IN	YDISPL,IN	SLOPE	AXIAL,LBS	LAT,LBS	BM,LBS-IN	STRESS,LBS/IN**2
0.169E+00	0.498E+00	0.585E-03	0.112E+06	-0.736E+04	0.000E+00	0.523E+04

LATERALLY LOADED PILE

X IN *****	DEFLECTION IN **********	MOMENT LBS-IN **********	SHEAR LBS **********	SOIL REACTION LBS/IN **********	TOTAL STRESS LBS/IN**2 **********	FLEXURAL RIGIDITY LBS-IN**2 **********
0.00	0.498E+00	0.000E+00	-0.664E+04	0.452E-03	0.523E+04	0.211E+11

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      11.96  0.505E+00  0.802E+05 -0.628E+04  0.454E-03  0.600E+04  0.211E+11
      23.91  0.512E+00  0.152E+06 -0.555E+04  0.456E-03  0.668E+04  0.211E+11
      35.87  0.517E+00  0.214E+06 -0.483E+04  0.457E-03  0.729E+04  0.211E+11
      47.83  0.521E+00  0.268E+06 -0.410E+04  0.459E-03  0.780E+04  0.211E+11
      59.79  0.523E+00  0.313E+06 -0.338E+04  0.459E-03  0.823E+04  0.211E+11
      71.74  0.523E+00  0.349E+06 -0.265E+04  0.459E-03  0.858E+04  0.211E+11
      83.70  0.521E+00  0.376E+06 -0.193E+04  0.459E-03  0.884E+04  0.211E+11
      95.66  0.516E+00  0.394E+06 -0.120E+04  0.457E-03  0.902E+04  0.211E+11
     107.62  0.509E+00  0.404E+06 -0.479E+03  0.455E-03  0.910E+04  0.211E+11
     119.57  0.498E+00  0.404E+06  0.246E+03  0.452E-03  0.911E+04  0.211E+11
     131.53  0.485E+00  0.395E+06  0.970E+03  0.448E-03  0.902E+04  0.211E+11
     143.49  0.470E+00  0.377E+06  0.169E+04  0.443E-03  0.885E+04  0.211E+11
     155.45  0.451E+00  0.351E+06  0.242E+04  0.437E-03  0.860E+04  0.211E+11
     167.40  0.431E+00  0.315E+06  0.314E+04  0.430E-03  0.826E+04  0.211E+11
     179.36  0.408E+00  0.271E+06  0.387E+04  0.423E-03  0.783E+04  0.211E+11
     191.32  0.384E+00  0.217E+06  0.459E+04  0.414E-03  0.732E+04  0.211E+11
     203.28  0.358E+00  0.155E+06  0.532E+04  0.404E-03  0.672E+04  0.211E+11
     215.23  0.330E+00  0.843E+05  0.604E+04  0.394E-03  0.604E+04  0.211E+11
     227.19  0.303E+00  0.467E+04  0.588E+04  0.880E+02  0.527E+04  0.211E+11
     239.15  0.275E+00 -0.624E+05  0.469E+04  0.112E+03  0.583E+04  0.211E+11
     251.11  0.248E+00 -0.114E+06  0.318E+04  0.140E+03  0.632E+04  0.211E+11
     263.06  0.221E+00 -0.144E+06  0.222E+04  0.193E+02  0.662E+04  0.211E+11
     275.02  0.196E+00 -0.173E+06  0.198E+04  0.214E+02  0.689E+04  0.211E+11
     286.98  0.172E+00 -0.197E+06  0.171E+04  0.234E+02  0.712E+04  0.211E+11
     298.94  0.149E+00 -0.219E+06  0.142E+04  0.250E+02  0.733E+04  0.211E+11
     310.89  0.127E+00 -0.236E+06  0.112E+04  0.263E+02  0.750E+04  0.211E+11
     322.85  0.107E+00 -0.250E+06  0.796E+03  0.273E+02  0.763E+04  0.211E+11
     334.81  0.889E-01 -0.260E+06  0.466E+03  0.280E+02  0.772E+04  0.211E+11
     346.77  0.724E-01 -0.265E+06  0.129E+03  0.283E+02  0.778E+04  0.211E+11
     358.72  0.577E-01 -0.266E+06 -0.209E+03  0.282E+02  0.779E+04  0.211E+11
     NUMBER OF ITERATIONS IN LLP =    21


       * PILE GROUP *  5


 PILE TOP DISPLACEMENTS AND REACTIONS


       THE GLOBAL STRUCTURE COORDINATE SYSTEM
       --------------------------------------

 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

 0.343E-01  0.525E+00 0.321E-02-0.108E+06 0.282E+05 0.000E+00   0.518E+04


       THE LOCAL MEMBER COORDINATE SYSTEM
       ------------------------------------

 XDISPL,IN   YDISPL,IN   SLOPE   AXIAL,LBS  LAT,LBS  BM,LBS-IN
STRESS,LBS/IN**2

-0.163E+00  0.501E+00 0.321E-02-0.111E+06-0.140E+05 0.000E+00   0.518E+04
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LATERALLY LOADED PILE

X	DEFLECTION	MOMENT	SHEAR	SOIL REACTION	TOTAL STRESS	FLEXURAL RIGIDITY
IN	IN	LBS-IN	LBS	LBS/IN	LBS/IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
0.00	0.501E+00	0.000E+00	-0.124E+05	0.468E-03	0.518E+04	0.211E+11
14.44	0.547E+00	0.174E+06	-0.117E+05	0.482E-03	0.685E+04	0.211E+11
28.87	0.592E+00	0.327E+06	-0.102E+05	0.495E-03	0.832E+04	0.211E+11
43.31	0.633E+00	0.458E+06	-0.865E+04	0.506E-03	0.958E+04	0.211E+11
57.74	0.670E+00	0.568E+06	-0.714E+04	0.515E-03	0.106E+05	0.211E+11
72.18	0.701E+00	0.657E+06	-0.562E+04	0.523E-03	0.115E+05	0.211E+11
86.62	0.726E+00	0.724E+06	-0.410E+04	0.529E-03	0.121E+05	0.211E+11
101.05	0.744E+00	0.771E+06	-0.259E+04	0.534E-03	0.126E+05	0.211E+11
115.49	0.754E+00	0.796E+06	-0.107E+04	0.536E-03	0.128E+05	0.211E+11
129.93	0.756E+00	0.800E+06	0.443E+03	0.537E-03	0.129E+05	0.211E+11
144.36	0.750E+00	0.784E+06	0.196E+04	0.535E-03	0.127E+05	0.211E+11
158.80	0.737E+00	0.746E+06	0.347E+04	0.532E-03	0.123E+05	0.211E+11
173.23	0.716E+00	0.687E+06	0.499E+04	0.527E-03	0.118E+05	0.211E+11
187.67	0.689E+00	0.607E+06	0.651E+04	0.520E-03	0.110E+05	0.211E+11
202.11	0.655E+00	0.506E+06	0.802E+04	0.512E-03	0.100E+05	0.211E+11
216.54	0.617E+00	0.384E+06	0.871E+04	0.906E+01	0.886E+04	0.211E+11
230.98	0.575E+00	0.263E+06	0.811E+04	0.743E+02	0.771E+04	0.211E+11
245.41	0.530E+00	0.159E+06	0.689E+04	0.948E+02	0.671E+04	0.211E+11
259.85	0.483E+00	0.746E+05	0.603E+04	0.247E+02	0.590E+04	0.211E+11
274.29	0.436E+00	-0.466E+04	0.564E+04	0.287E+02	0.522E+04	0.211E+11
288.72	0.389E+00	-0.779E+05	0.520E+04	0.323E+02	0.593E+04	0.211E+11
303.16	0.343E+00	-0.145E+06	0.471E+04	0.354E+02	0.657E+04	0.211E+11
317.60	0.298E+00	-0.204E+06	0.418E+04	0.381E+02	0.714E+04	0.211E+11
332.03	0.255E+00	-0.256E+06	0.362E+04	0.403E+02	0.763E+04	0.211E+11
346.47	0.215E+00	-0.299E+06	0.302E+04	0.419E+02	0.805E+04	0.211E+11
360.90	0.177E+00	-0.334E+06	0.241E+04	0.430E+02	0.839E+04	0.211E+11
375.34	0.143E+00	-0.361E+06	0.179E+04	0.428E+02	0.864E+04	0.211E+11
389.78	0.112E+00	-0.379E+06	0.120E+04	0.395E+02	0.882E+04	0.211E+11
404.21	0.855E-01	-0.389E+06	0.652E+03	0.361E+02	0.891E+04	0.211E+11
418.65	0.625E-01	-0.392E+06	-0.916E+02	0.669E+02	0.894E+04	0.211E+11
433.09	0.434E-01	-0.382E+06	-0.102E+04	0.613E+02	0.884E+04	0.211E+11
447.52	0.280E-01	-0.359E+06	-0.185E+04	0.548E+02	0.863E+04	0.211E+11
461.96	0.161E-01	-0.325E+06	-0.259E+04	0.471E+02	0.830E+04	0.211E+11
476.39	0.749E-02	-0.282E+06	-0.320E+04	0.377E+02	0.789E+04	0.211E+11
490.83	0.162E-02	-0.231E+06	-0.364E+04	0.234E+02	0.740E+04	0.211E+11
505.27	-0.196E-02	-0.176E+06	-0.362E+04	-0.258E+02	0.687E+04	0.211E+11
519.70	-0.382E-02	-0.126E+06	-0.320E+04	-0.332E+02	0.639E+04	0.211E+11
534.14	-0.444E-02	-0.830E+05	-0.270E+04	-0.360E+02	0.598E+04	0.211E+11
548.57	-0.424E-02	-0.478E+05	-0.218E+04	-0.366E+02	0.564E+04	0.211E+11
563.01	-0.356E-02	-0.203E+05	-0.166E+04	-0.356E+02	0.537E+04	0.211E+11
577.45	-0.269E-02	-0.204E+03	-0.116E+04	-0.335E+02	0.518E+04	0.211E+11
591.88	-0.181E-02	0.129E+05	-0.697E+03	-0.303E+02	0.530E+04	0.211E+11
606.32	-0.107E-02	0.197E+05	-0.289E+03	-0.261E+02	0.537E+04	0.211E+11
620.76	-0.512E-03	0.211E+05	0.518E+02	-0.211E+02	0.538E+04	0.211E+11
635.19	-0.167E-03	0.181E+05	0.313E+03	-0.150E+02	0.535E+04	0.211E+11
649.63	-0.166E-06	0.120E+05	0.431E+03	-0.147E+01	0.529E+04	0.211E+11
664.06	0.478E-04	0.565E+04	0.366E+03	0.105E+02	0.523E+04	0.211E+11
678.50	0.401E-04	0.147E+04	0.216E+03	0.102E+02	0.519E+04	0.211E+11
692.94	0.178E-04	-0.582E+03	0.838E+02	0.808E+01	0.518E+04	0.211E+11
707.37	0.135E-05	-0.947E+03	-0.337E+00	0.357E+01	0.519E+04	0.211E+11
721.81	-0.579E-05	-0.569E+03	-0.233E+02	-0.393E+00	0.518E+04	0.211E+11

736.24	-0.733E-05	-0.274E+03	-0.168E+02	-0.506E+00	0.518E+04	0.211E+11
750.68	-0.616E-05	-0.841E+02	-0.100E+02	-0.433E+00	0.518E+04	0.211E+11
765.12	-0.416E-05	0.154E+02	-0.476E+01	-0.298E+00	0.518E+04	0.211E+11
779.55	-0.231E-05	0.528E+02	-0.139E+01	-0.168E+00	0.518E+04	0.211E+11
793.99	-0.982E-06	0.552E+02	0.352E+00	-0.729E-01	0.518E+04	0.211E+11
808.43	-0.199E-06	0.424E+02	0.987E+00	-0.150E-01	0.518E+04	0.211E+11
822.86	0.167E-06	0.265E+02	0.100E+01	0.128E-01	0.518E+04	0.211E+11
837.30	0.271E-06	0.134E+02	0.758E+00	0.211E-01	0.518E+04	0.211E+11
851.73	0.243E-06	0.465E+01	0.466E+00	0.193E-01	0.518E+04	0.211E+11
866.17	0.169E-06	-0.705E-01	0.229E+00	0.136E-01	0.518E+04	0.211E+11
880.61	0.957E-07	-0.195E+01	0.741E-01	0.785E-02	0.518E+04	0.211E+11
895.04	0.419E-07	-0.220E+01	-0.772E-02	0.349E-02	0.518E+04	0.211E+11
909.48	0.965E-08	-0.172E+01	-0.388E-01	0.816E-03	0.518E+04	0.211E+11
923.91	-0.565E-08	-0.107E+01	-0.412E-01	-0.485E-03	0.518E+04	0.211E+11
938.35	-0.104E-07	-0.526E+00	-0.311E-01	-0.906E-03	0.518E+04	0.211E+11
952.79	-0.995E-08	-0.172E+00	-0.182E-01	-0.879E-03	0.518E+04	0.211E+11
967.22	-0.781E-08	-0.128E-03	-0.683E-02	-0.700E-03	0.518E+04	0.211E+11
981.66	-0.567E-08	0.253E-01	-0.132E-02	-0.638E-04	0.518E+04	0.211E+11
996.10	-0.378E-08	0.374E-01	-0.528E-03	-0.454E-04	0.518E+04	0.211E+11
1010.53	-0.226E-08	0.401E-01	0.878E-05	-0.289E-04	0.518E+04	0.211E+11
1024.97	-0.113E-08	0.369E-01	0.328E-03	-0.154E-04	0.518E+04	0.211E+11
1039.40	-0.371E-09	0.305E-01	0.478E-03	-0.532E-05	0.518E+04	0.211E+11
1053.84	0.905E-10	0.230E-01	0.506E-03	0.137E-05	0.518E+04	0.211E+11
1068.28	0.326E-09	0.158E-01	0.459E-03	0.518E-05	0.518E+04	0.211E+11
1082.71	0.406E-09	0.966E-02	0.373E-03	0.676E-05	0.518E+04	0.211E+11
1097.15	0.390E-09	0.499E-02	0.275E-03	0.681E-05	0.518E+04	0.211E+11
1111.59	0.326E-09	0.173E-02	0.183E-03	0.593E-05	0.518E+04	0.211E+11
1126.02	0.244E-09	-0.279E-03	0.107E-03	0.463E-05	0.518E+04	0.211E+11
1140.46	0.165E-09	-0.133E-02	0.497E-04	0.326E-05	0.518E+04	0.211E+11
1154.89	0.992E-10	-0.170E-02	0.114E-04	0.204E-05	0.518E+04	0.211E+11
1169.33	0.501E-10	-0.164E-02	-0.110E-04	0.107E-05	0.518E+04	0.211E+11
1183.77	0.172E-10	-0.137E-02	-0.214E-04	0.380E-06	0.518E+04	0.211E+11
1198.20	-0.214E-11	-0.102E-02	-0.238E-04	-0.489E-07	0.518E+04	0.211E+11
1212.64	-0.114E-10	-0.680E-03	-0.215E-04	-0.270E-06	0.518E+04	0.211E+11
1227.07	-0.140E-10	-0.397E-03	-0.171E-04	-0.342E-06	0.518E+04	0.211E+11
1241.51	-0.127E-10	-0.186E-03	-0.123E-04	-0.320E-06	0.518E+04	0.211E+11
1255.95	-0.954E-11	-0.416E-04	-0.823E-05	-0.248E-06	0.518E+04	0.211E+11
1270.38	-0.597E-11	0.510E-04	-0.529E-05	-0.160E-06	0.518E+04	0.211E+11
1284.82	-0.291E-11	0.110E-03	-0.165E-05	-0.344E-06	0.518E+04	0.211E+11
1299.26	-0.936E-12	0.982E-04	0.163E-05	-0.112E-06	0.518E+04	0.211E+11
1313.69	0.709E-13	0.629E-04	0.238E-05	0.856E-08	0.518E+04	0.211E+11
1328.13	0.458E-12	0.294E-04	0.191E-05	0.559E-07	0.518E+04	0.211E+11
1342.56	0.556E-12	0.753E-05	0.102E-05	0.686E-07	0.518E+04	0.211E+11
1357.00	0.580E-12	0.000E+00	0.357E-22	0.722E-07	0.518E+04	0.211E+11

NUMBER OF ITERATIONS IN LLP =   16

Design Example #3

A cross section of the wall section used for Example 3 is shown in Figure 1, and is based on a wall constructed in New Orleans at Gainard Woods. The water level used in this example is elevation 17.0' and assumed to be a top of wall load case. The target factor of safety was chosen to be 1.5 in this example rather than the required 1.4 (for demonstration purposes) to provide a greater disparity from the without pile factor of safety. The water level on the protected side is assumed to be at the bottom of footing as the ground slopes toward a canal on the protected side. The soil information for this example is listed in Table 1.

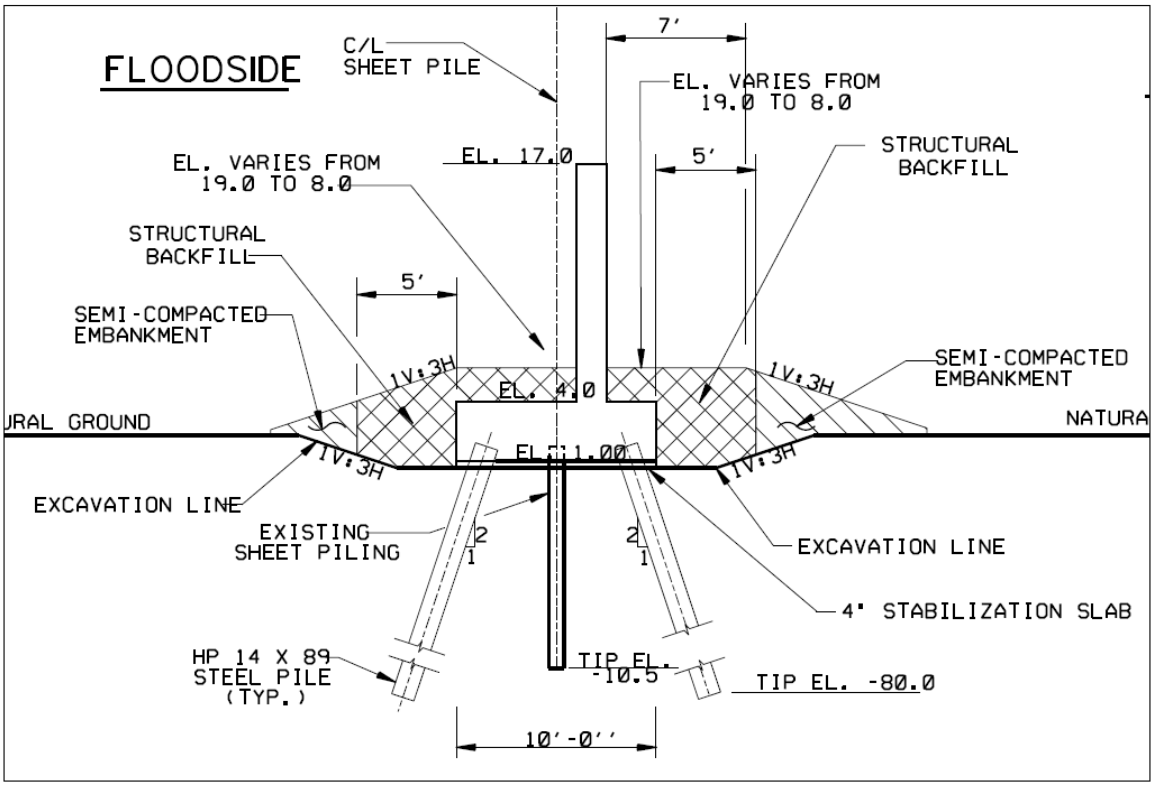


Figure 1. Wall Geometry.

Table 1. Soil Properties

Top of Layer Elevation, ft	Saturated Unit Weight, pcf	Undrained Shear Strength, psf	Friction Angle, Phi
4	108	400	0
2	86	300	0
-7	98	300	0
-10	100	300	0
-22	120	0	30
-27	100	320	0
-40	100	450	0
-45	100	450	0

Step 1 Initial Slope Stability Analysis

Perform a Spencer's method slope stability analysis to determine the critical slip surface with the water load only on the ground surface and no piles. UTexas4 was used in this example for all of the slope stability analysis. For the design example, the critical failure surface is shown in Figure 2 where the factor of safety is 1.34. Because this value is less than the required value of 1.5, the T-Wall will need to carry an unbalanced load in addition to any loads on the structure.

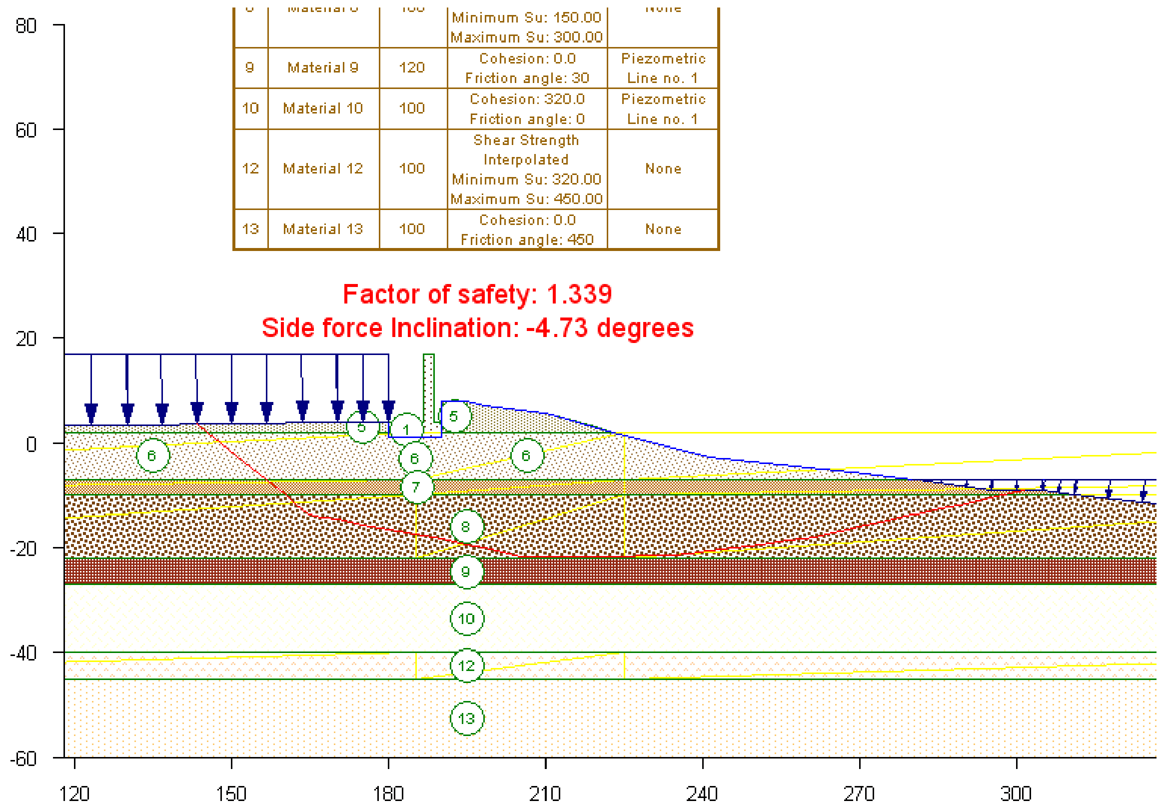


Figure 2. Spencer's analysis of the T-Wall without piles.

Step 2 Unbalanced Force Computations

Determine (unbalanced) forces required to provide the required global stability factor of safety. The critical failure surface extends down to elevation -22' in this example. The elevation of the ground surface at the heel of the T-Wall is at elevation 4'. It is assumed that the unbalanced load is halfway between these two elevations. Apply a line load at elevation -9', at the midpoint of the expected base width (for a non-circular failure surface). A line load of 3800 lb/ft at this location results in F=1.50. The target factor of safety is 1.5 so the computed unbalanced load is slightly too low in this example.

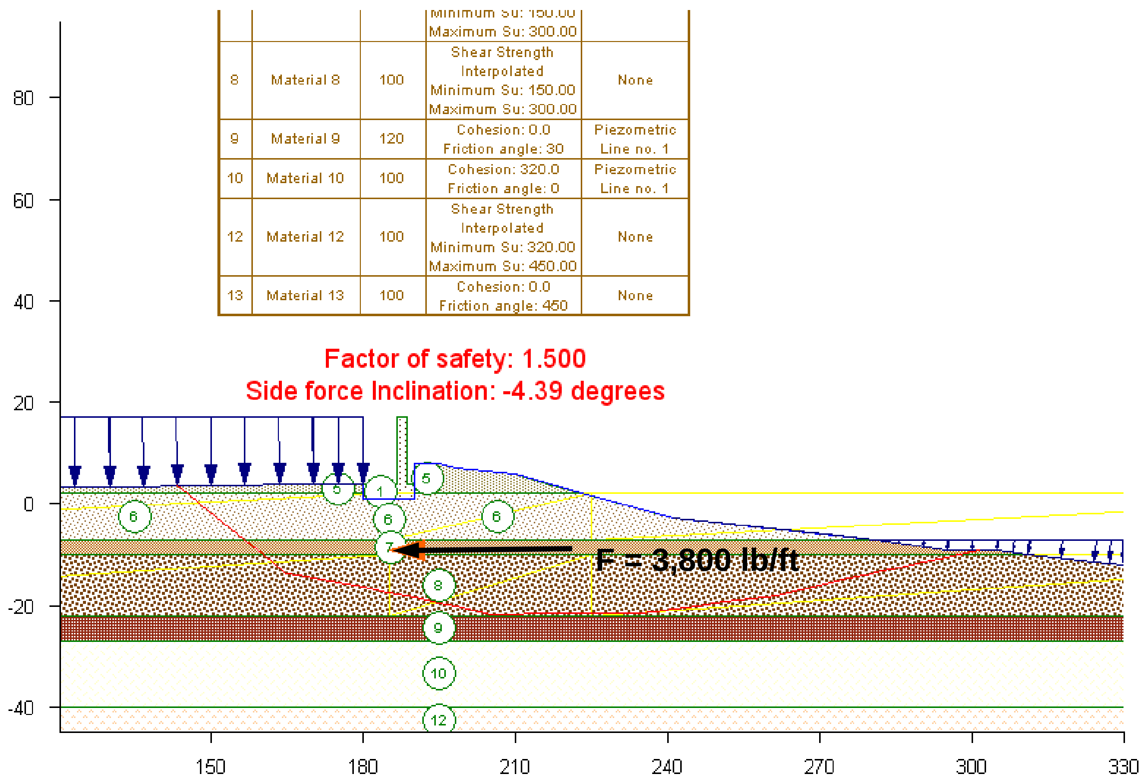


Figure 3. Spencer's analysis of the T-Wall with an unbalanced load to increase global stability (note FS is slightly below target FS=1.5 in this example).

It should be noted that a search for the critical failure surface was performed with the unbalanced load shown in Figure 3. The search ensures that if the pile foundation of the T-Wall can safely carry the unbalanced load in addition to any other loads on the structure, the global stability will meet the required factor of safety. The UTexas4 input files for Figures 2 and 3 are attached at the end of this example.

Step 3 Allowable Pile Capacity Analysis

3.1 For the preliminary analysis, allowable pile capacities determined by engineers in New Orleans District for the original design of this project are shown in Figure 4 for ultimate loads vs. depth. The solid line is for the Q case and the dashed line is for the S case. For water to the top of wall under hurricane surge loadings with fine grained soils, the Q case will be used. No axial capacity is accounted for above the lowest elevation of the critical surface in the graph. Since this is treated as a still water load case, the allowable load factor is 3.0.

From the figures below and knowing that maximum pile loads in compression will be about 65 kips, the required ultimate capacity is 65*3/2kips/ton = 98 tons. This would be a pile driven depth to about 100 feet from Figure 4. The tensile capacity is about the same.

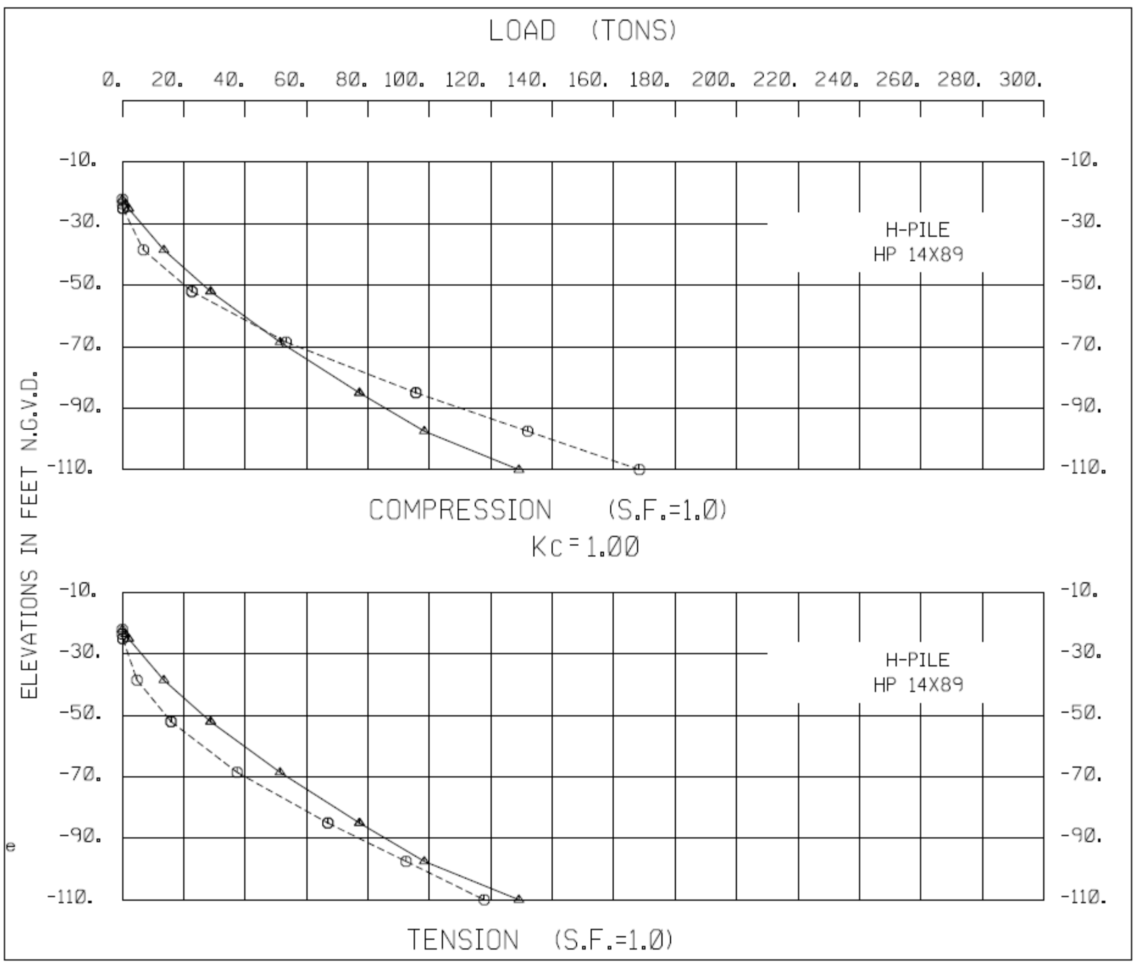


Figure 4. Ultimate Axial Capacity with Depth, Calculated

3.2 The allowable shear load (from LPILE or COM624G) is determined from pile head deflection versus lateral load plot. This was not determined for this problem.

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. The unbalanced force is converted to an "equivalent" force applied to the bottom of the T-wall, F_{cap}, as calculated as shown below (See Figure 5):

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

Where:

F_{ub} = unbalanced force computed in step 2.

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

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

$$R = \sqrt[4]{\frac{EI}{Es}}$$

E = Modulus of Elasticity of Pile (lb/in^2)

I = Moment of Inertia of Pile (in^4)

Es = Modulus of Subgrade Reaction (lb/in^2) below critical failure surface. In New Orleans District this equates to the values listed as K_HB.

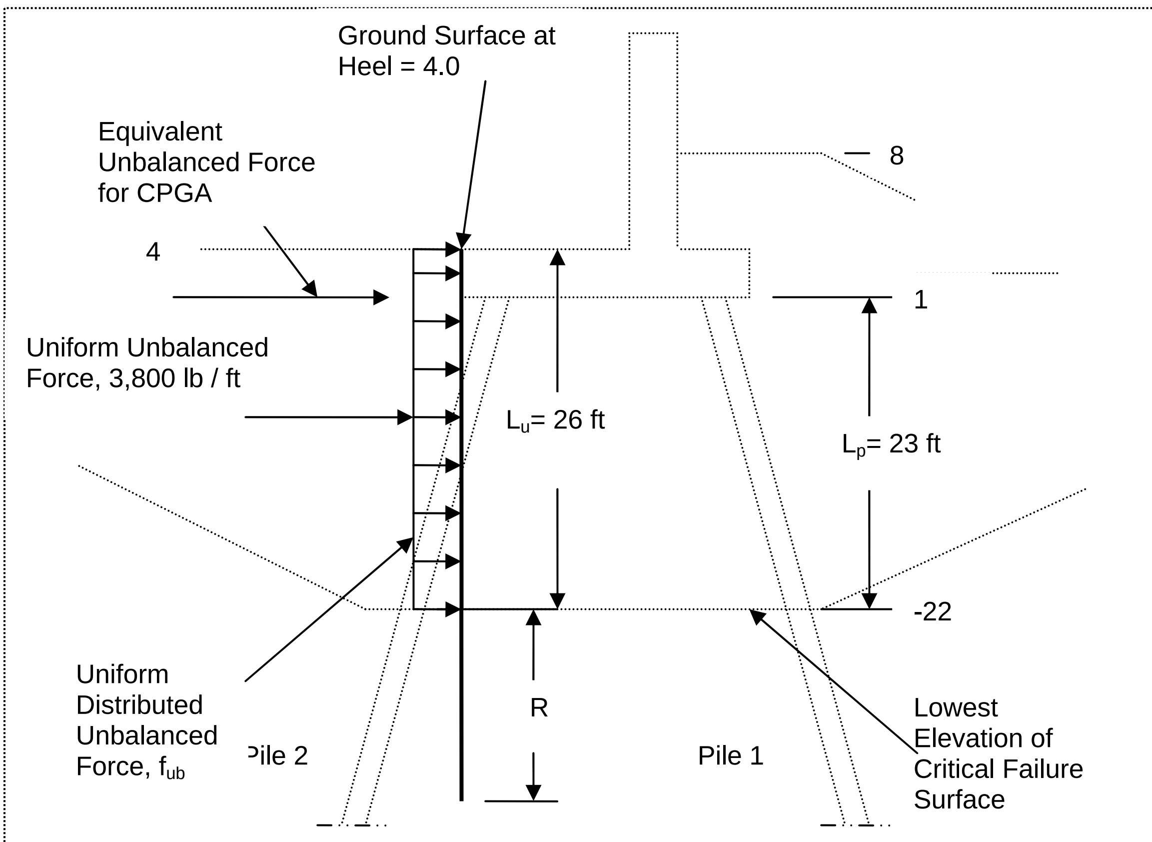


Figure 5. Equivalent Force Computation for Preliminary Design with CPGA

For the solution:
Piles = HP 14x89. $I = 904\ in^4$, $E = 29{,}000{,}000$ psi

Soils – the stiffness, Es, below the failure surface is shown in Figure 6. Based on this a value of 120 psi is used.

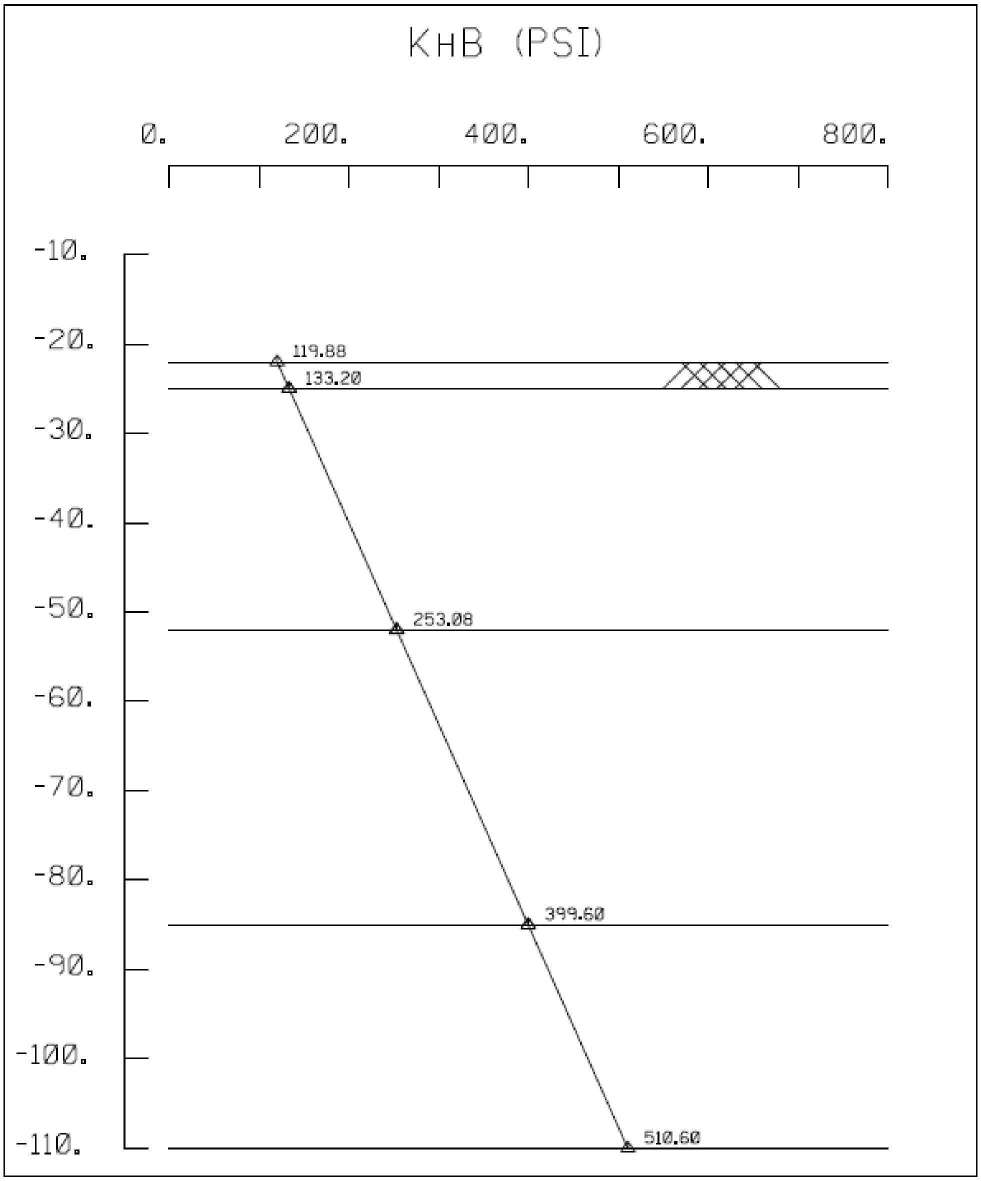


Figure 6. Soil Stiffness with Depth

R therefore is equal to 120 in = 10 feet

$P_{cap} = 3800 * (26/2 + 10) / (23 + 10) = 2648$ lb/ft

4.2 This unbalanced force is then analyzed with appropriate load cases in CPGA. Generally 8 to 20 load cases may be analyzed depending on expected load conditions. For this example, only the water at top of wall case is analyzed but both pervious and impervious foundation conditions are evaluated. See the spreadsheet calculations in Attachment 3 for the computation of the input for CPGA. The model is a 5 foot strip of the pile foundation.

For the CPGA analysis, the soil modulus, Es is adjusted based on the global stability factor of safety. For this example case, the factor of safety is 1.34. Es for CPGA is computed from the ratio of the computed factor of safety to the target factor of safety. At the bottom of the wall footing, the soil has a shear strength of about 300 psf. Es = 0.2222 Qu B. Therefore, Es = 0.2222(300)(14/12) = 78 psi = at the bottom of the wall footing. Computing Es based on reduction of factor of safety:

CPGA Es = (1.34-1.0) / (1.5 – 1.0) * 78 = 46 psi

4.3. Group reductions are according to EM 1110-2-2906. Since the pile spacing is greater than 8B in the direction of load and 2.5B parallel to the load, no reduction is necessary.

The CPGA output is shown in Attachment 4. A summary of results for the two load conditions analyzed are shown below:

```
          PILE FORCES IN LOCAL GEOMETRY

              M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
              * INDICATES PILE FAILURE
              # INDICATES CBF BASED ON MOMENTS DUE TO
                          (F3*EMIN) FOR CONCRETE PILES
              B INDICATES BUCKLING CONTROLS
```

LOAD CASE - 1 Pervious Condition

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF
1	3.7	.0	62.5	.0	-259.0	.0	.96	.25
2	-4.1	.0	-13.7	.0	289.5	.0	.21	.13

LOAD CASE - 2 Impervious Condition

PILE	F1 K	F2 K	F3 K	M1 IN-K	M2 IN-K	M3 IN-K	ALF	CBF
1	2.4	.0	65.0	.0	-171.6	.0	1.00	.23
2	-2.9	.0	-16.2	.0	202.1	.0	.25	.11

```
Where:
F1 =  Shear in pile at pile cap perpendicular to wall
F2 =  Shear in Pile at Pile Cap parallel to wall
F3 =  Axial Load in Pile
M1 =  Maximum moment in pile perpendicular to wall
M2 =  Maximum moment in pile parallel to wall
M3 =  Torsion in pile
ALF=  Axial load factor – computed axial load divided by allowable load
CBF=  Combined Bending factor – combined computed axial and bending
forces relative to allowable forces
```

The pile layout is adequate according to the CPGA analysis.

Computed deflections from the CPGA analysis are shown below:

```
          PILE CAP DISPLACEMENTS

 LOAD
 CASE        DX          DZ          R
             IN          IN          RAD

    1   -.7541E+00  -.2047E+00  -.5023E-02
    2   -.5370E+00  -.4687E-01  -.2391E-02
```

These deflections are a bit more than the allowable vertical deflection (DZ) of 0.5 inches and allowable horizontal deflection (DX) of 0.75 inches from the Hurricane and Storm Damage Reduction Design Guidelines.

4.4 Sheet pile design. Seepage design of the sheet pile is not performed for this example.

4.5 Check for resistance against flow through. Since the pile spacing is uniform, we will analyze one row of piles parallel with the loading rather than the entire monolith.

a. Compute the resistance of the flood side row of piles.

$$\sum P_{all} = \frac{n \sum P_{ult}}{1.5}$$

Where:

n = number of piles in the row within a monolith. Or, for monoliths with uniformly spaced pile rows, n = 1. Use 1 for this example

$P_{ult} = \beta(9S_u b)$

S_u = soil shear strength

b = pile width = 14”

β = group reduction factor pile spacing parallel to the load - since the piles batter opposite to each other, there group affects are not computed.

For the soils under the slab, S_u = 300 psf

Therefore: P_{ult} = 9(300)(14/12) = 3,150 lb/ft

ΣP_{ult} = summation of P_{ult} over the height L_p, as defined in paragraph 4.1
For single layer soil is P_{ult} multiplied by L_p (23 ft) - That is the condition here since the shear strength is constant from the base to the critical failure surface.

ΣP_{ult} = 3,150(23) = 72,450 lb
ΣP_{all} = 1(72,450)/1.5 = 48,300 lb

b. Compute the load acting on the piles below the pile cap.

$$F_{up} = wf_{ub}L_p$$

Where:
w = Monolith width. Since we are looking at one row of piles in this example, w = the pile spacing perpendicular to the unbalanced force (s_t) = 5 ft.

$$f_{ub} = \frac{F_{ub}}{L_u}$$

F_{ub} = Total unbalanced force per foot from Step 2 = 3,800 lb/ft
L_u = 26 ft
L_p = 23 ft

f_{ub} = 3,800/ 26 = 146 lb/ft/ft

F_p = 5(146)(23) = 3,358 lb

c. Check the capacity of the piles 50% of F_p = 3,358(0.50) = 1,679 lb

The capacity ΣP_{all} = 48,300 lb > 1,679 lb so OK for flow through with this check.

4.6 Second flow through 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:

$$f_{ub}L_p \le \frac{A_pS_u}{FS}\left[\frac{2}{(s_t - b)}\right]$$

Where:
A_pS_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. – See Figure 7. S_u =300 psf
A_pS_u = (23(10+25.33)/2)(300 psf) = 122,000 lb
FS = Target factor of safety used in Steps 1 and 2. – 1.5
s_t= the spacing of the piles transverse (perpendicular) to the unbalanced force 5 ft
b = pile width – 14 inches

$f_{pb}L_p =$ (246 lb/ft)(23 ft) = 5,658 lb

$$\frac{A_p S_u}{FS}\left[\frac{2}{(s_t - b)}\right] = \frac{122{,}000}{1.5}\left[\frac{2}{5-\left(\frac{14}{12}\right)}\right] = 42{,}434 \text{ lb}$$

Therefore, capacity against flow through is OK

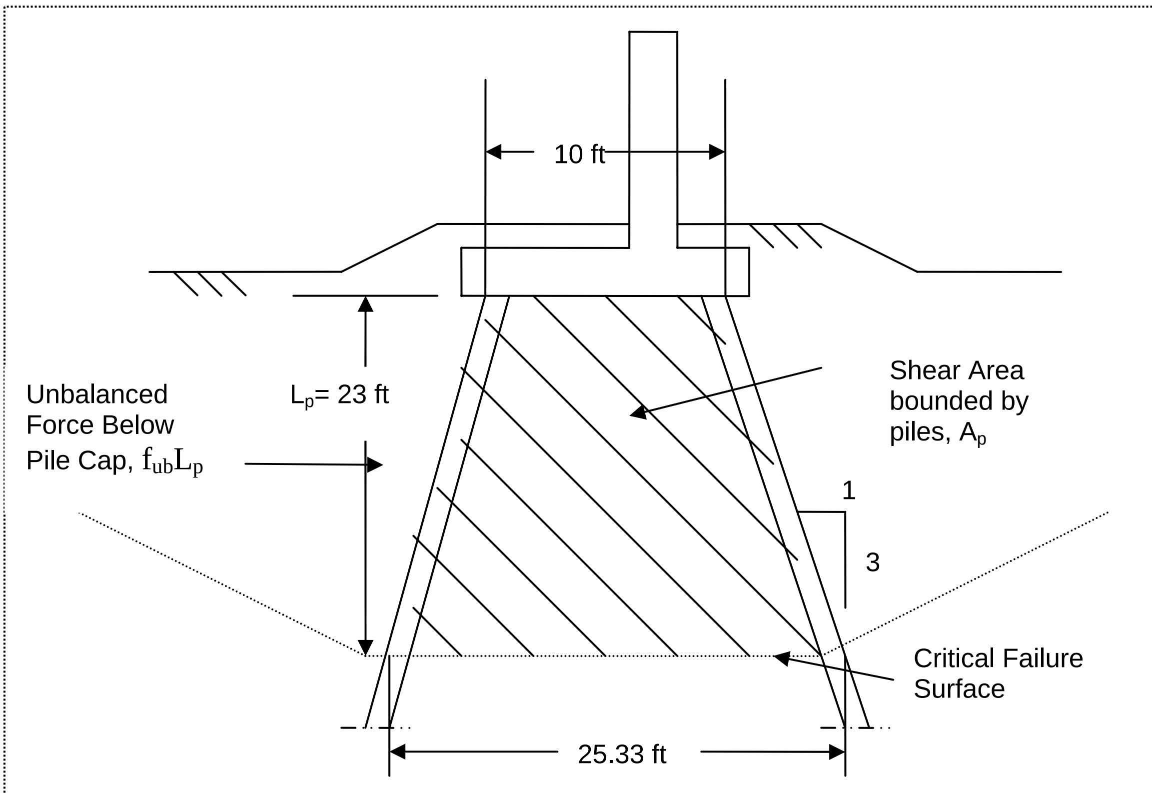


Figure 7. Shear Area for Flow Through Calculation

Step 5 <u>Pile Group Analysis</u>

5.1 A Group 7 analysis is performed using all loads applied to the T-wall structure. Critical load cases from step 4 would be used. In this example, only one load case with two foundation conditions was performed.

5.2 The loads applied in the Group 7 model include the distributed loads representing the unbalanced force that acts directly on the piles and also the water loads and self-weight of the wall that acts directly on the structure. In Group 7 these loads are resultant horizontal and vertical forces and the moments per width of spacing that act on the T-wall base (pile cap). They also include the unbalance force from the base of the cap to the top of soil, converted to a force and moment at the base of the structure. These forces are calculated using a worksheet or Excel spreadsheet and are shown at then end of the spreadsheets shown in Attachment 3. For this analysis the resultant forces per 5-ft of pile spacing were:

Pervious Foundation Condition

Vertical force	=	43,803 lb
Horizontal force	=	29,986 lb
Moment	=	-322,384 in-lbs

Impervious Foundation Condition

Vertical force	=	43,803 lb
Horizontal force	=	29,986 lb
Moment	=	-572,384 in-lbs

5.3 The unbalance load below the bottom of the footing is applied directly as distributed loads on the pile. Check if ($n\Sigma P_{ult}$) of the flood side pile row is greater than 50% F_p, (from 4.5)

.

$(n\Sigma P_{ult}) = 1\ (72{,}450\text{ lb}) = 72{,}450\text{ lb}$

$50\%\ F_p = 1{,}679\text{ lb}$

Therefore distribute 50% of F_p onto each row of piles.

$0.5 f_{ub} s_t = 0.5\ (146\text{ lb/ft/ft})(5\text{ ft}) = 365\text{ lb/ft} = 31\text{ lb/in}$

5.4 The Group 7 model is shown in Figure 8.

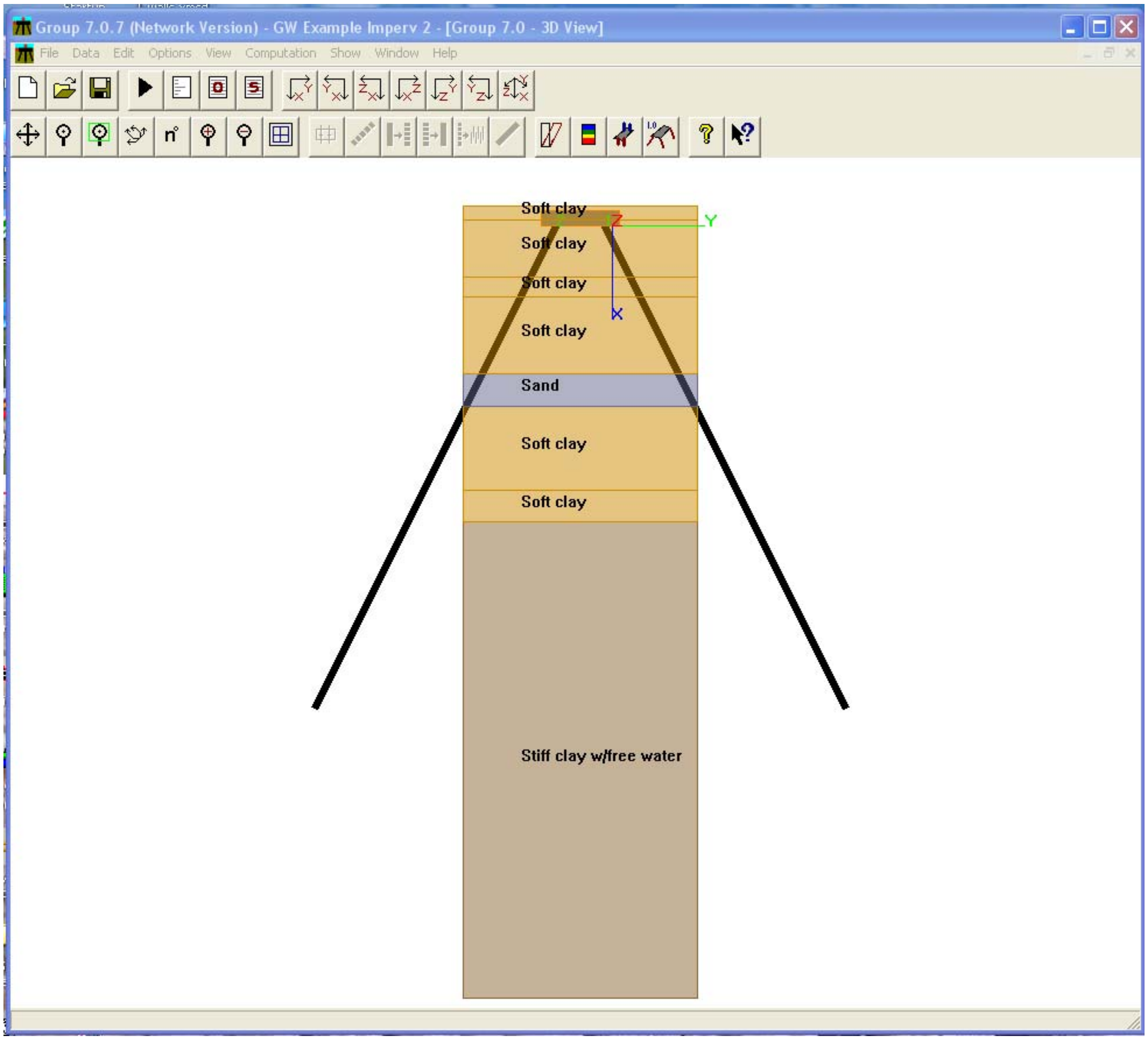


Figure 8. Group 7 Model

5.5 Additionally, in this analysis partial p-y springs can be used because the unreinforced factor of safety of 1.34 is between 1.0 and 1.5. The percentage of the full springs is determined as follows :

Partial spring percentage = (1.339 – 1.000)/ (1.5- 1.0) x 100% = 68%

Thus the strengths of in the top 4 layers, extending to Elevation -22 ft, were reduced to 68% of the undrained shear strength. The reduced undrained shear strength was used to scale the p-y curves above elevation -22 ft only. The results of the Group 7 analysis are listed in Table 1 where the pile responses for the full loading conditions on T-wall systems are listed. The complete Group 7 file for the Pervious Case is shown in Attachment 5.

Impervious Case	Left Pile (Pile #2)	Right Pile (Pile #1)
Axial Force (kips)	-14.5 (T)	62.5 (C)
Shear Force (kips)	1.3	1.5
Max. Moment (k-in)	64.4	118.3
Pervious Case	Left Pile (Pile #2)	Right Pile (Pile #1)
Axial Force (kips)	-14.5(T)	62.5 (C)
Shear Force (kips)	1.3	1.6
Max. Moment (k-in)	64	117.9

Illustration of the moment in the piles with depth is shown in Figure 9. The shear is shown in figure 10.

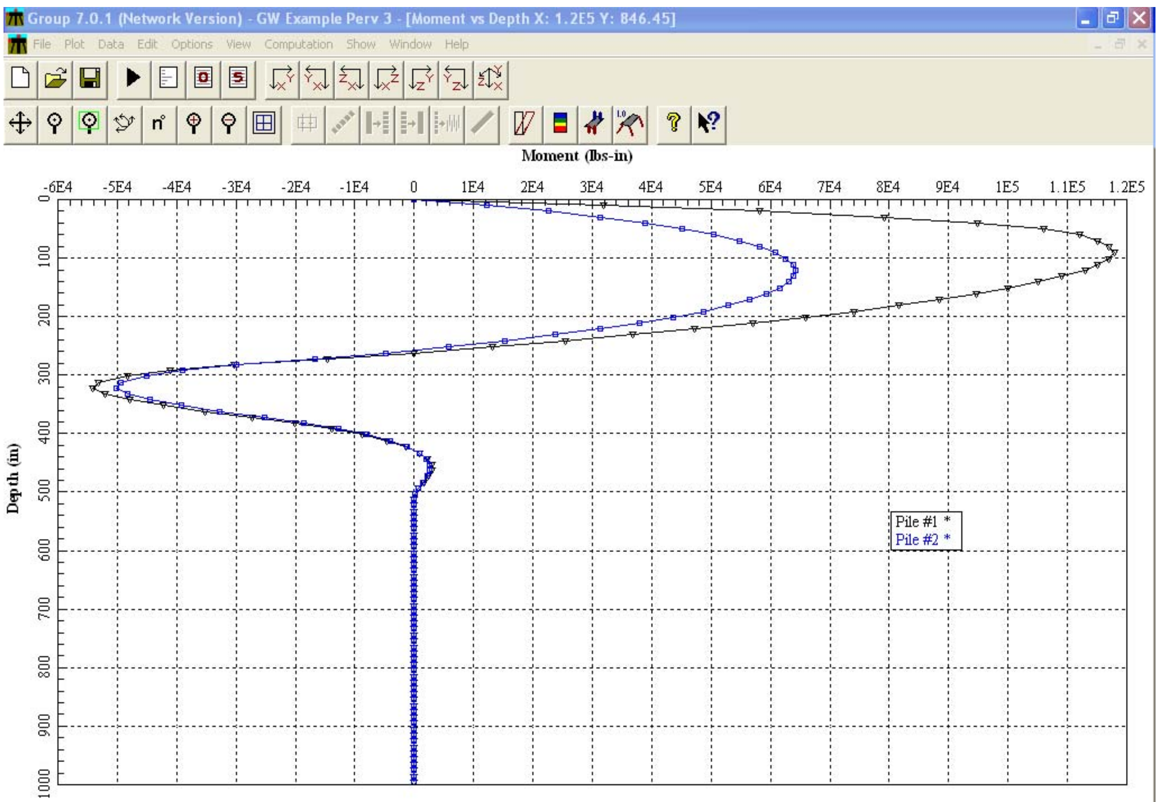


Figure 9. Moment in piles with depth for the pervious case

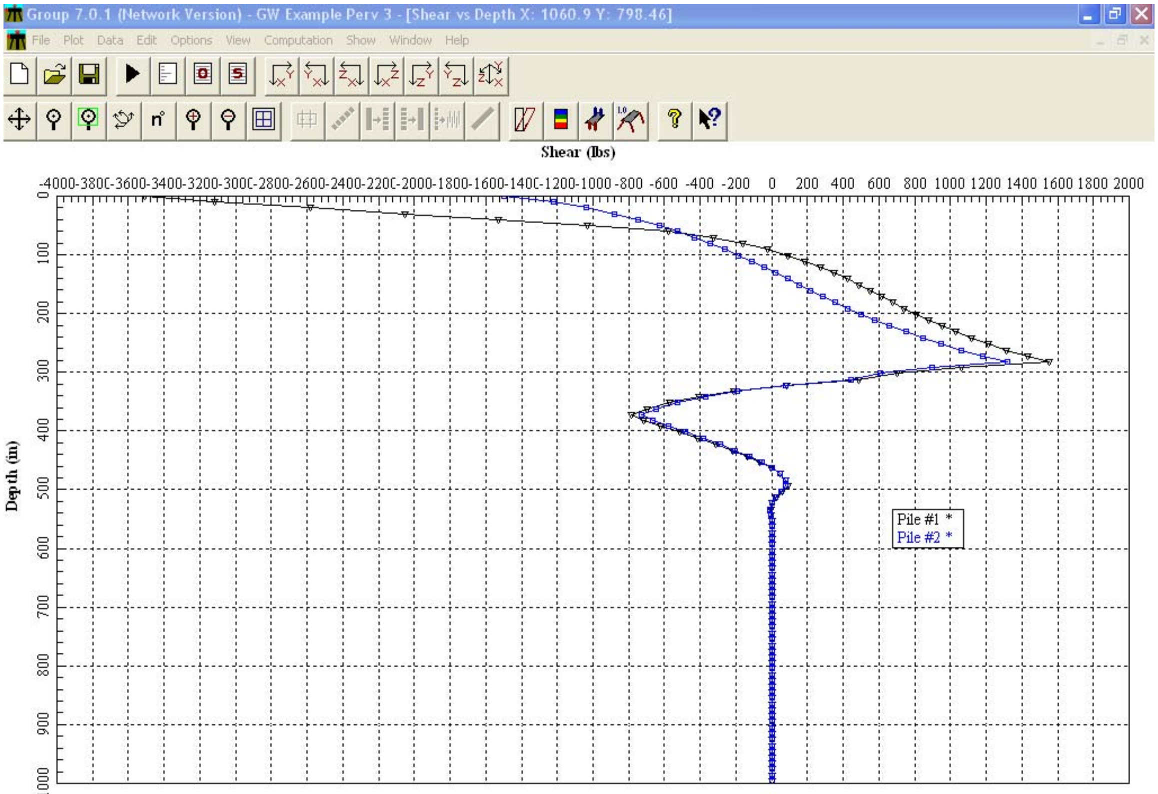


Figure 10. Shear versus depth for the pervious Case.

The axial force is found in the summary text from Group 7.

5.7 The axial forces and shear in Table 1 are then compared with allowable pile capacities determined in Step 3. The results of the comparison show that:

a. the axial compressive forces in the center pile, 62.5 kips, is less than the allowable capacity of 65 kips.
b. the axial tensile force from the left (flood side) pile of -14.5 kips is less than the allowable tensile load of 65 kips.
c. The shear forces in each of the three piles is much lower than the shear computed in examples 1 and 2. LPILE should be used to develop lateral capacity to verify its adequacy.

5.6 Moment and axial forces in the piles would also be checked for structural strength according to criteria in the Hurricane and Storm Damage Reduction System Design Guidelines and EM1110-2-2906.

Displacements from the Group 7 analysis are as follows:

```
PILE CAP DISPLACEMENTS
```

	VERTICAL,IN	HORIZONTAL,IN	ROTATION,RAD
Pervious	0.1129E+00	0.1042E-01	-0.1221E-02
Impervious	0.1129E+00	0.1042E-01	-0.1221E-02

These deflections are much less than the allowable vertical deflection (DZ) of 0.5 inches and allowable horizontal deflection (DX) of 0.75 inches from the Hurricane and Storm Damage Reduction Design Guidelines, even with out increases allowed for the top of wall load case. Figure 11 below shows displacement with depth.

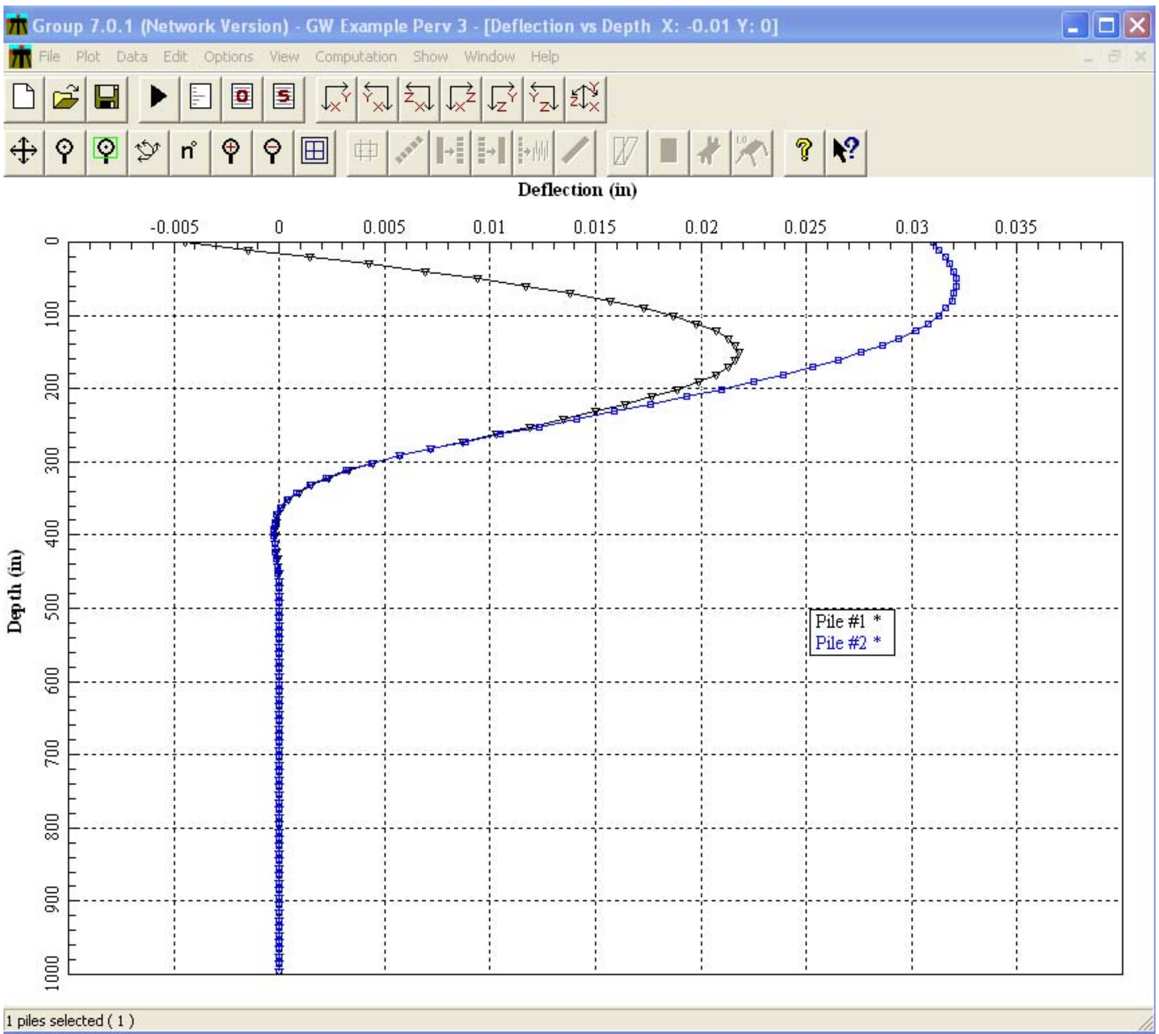


Figure 11. Deflection with Depth for the pervious foundation condition.

Step 6 Pile Group Analysis (unbalanced force)

6.1 Perform a Group 7 analysis with the unbalance force applied directly to the piles. The uniform unbalanced force above the base of the wall is added as a force and moment at the base of the wall. The distributed loads are statically equivalent to the unbalanced force of 3,800 lb/ft. No loads are applied to the cap except unbalance forces above the base of the wall equivalent to 2,192 lb lateral load and -43,803 lb-ft moment. The p-y springs are set to 0 to the critical failure surface by setting the ultimate shear stress of these soils at a very low value. The distributed loads were computed in the previous step and are shown in the Excel spreadsheet computations shown in Attachment 2. Results of the Group analysis are shown below:

Table2. Axial and shear Pile loads per 5-ft of width computed by Group 7 with unbalanced load distributed evenly on two piles

Impervious Case	Left Pile	Right Pile
Axial Force (kips)	-1.0 (T)	0.9 (C)
Shear Force (kips)	-13.2	-13.5

Step 7 Pile Reinforced Slope Stability Analysis

7.1 The UT4 pile reinforcement analysis using the slip surface from Step 5 is performed to determine if the target Factor of Safety of 1.5 is achieved. The piles are treated as reinforcements in the UT4 and the shear and axial forces from Step 6 are used to determine these forces. The forces in Table 2 must be converted to unit width conditions by dividing by the 5-ft pile spacing to be used as the axial and shear forces in the pile reinforcements in UT4. The results of the analysis are shown in Figure 12. The factor of safety is 1.574 which exceeds the target factor of safety of 1.5 . When the computed factor of safety exceeds the target, the global stability of the foundation is verified in this Step. The UTexas file used in this step is shown in attachment 5 of this example.

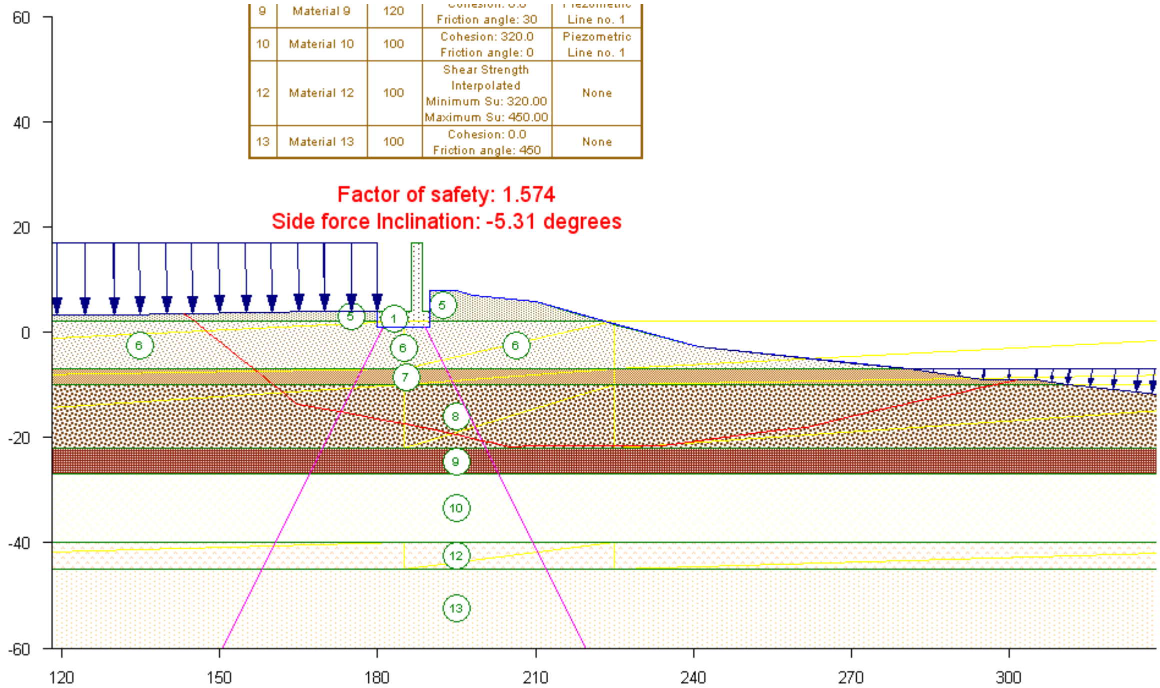


Figure 12. Factor of safety computed using pile forces from Group 7 analysis And critical failure surface from Step 2

Attachment 1 – UTexas analysis without piles that results in Figure 3.

```
HEADING
    T-wall Deep Seated Analysis
    Step 1 Analysis Without Piles

PROFILE LINES
         1    5 Profile 5
                 .00      3.30
              130.00      3.30
              170.00      4.00
              180.00      4.00

         3    1 T-wall
              180.00      4.00
              186.50      4.00
              186.51     17.00
              188.50     17.00
              188.51      4.00
              190.00      4.00

         2    5 Profile 5 PS
              190.00      8.00
              195.00      8.00
              198.00      7.00
              210.00      5.80
              216.20      4.00
              219.50      3.03
              219.60      3.00
              223.00      2.00

         6    6 Profile 6 - FS
                 .00      2.00
              180.00      2.00

         7    6 Profile 6 - Under Wall
              180.00      1.00
              190.00      1.00

         8    6 Profile 6 - PS
              190.00      2.00
              223.00      2.00
              225.00      1.47
              241.00     -2.80
              271.00     -6.00
              280.00     -6.90
              281.00     -7.00

         9    7 Profile 7
                 .00     -7.00
              281.00     -7.00
              295.00     -9.00
              305.00     -9.00
              311.00    -10.00
```

```
     10     8 Profile Line 8
                .00     -10.00
             311.00     -10.00
             324.00     -11.37
             330.00     -12.00
             337.50     -11.50
             345.00     -11.00
             351.00     -10.50
             358.00      -9.30
             400.00      -9.30

     11     9 Profile Line 9
                .00     -22.00
             400.00     -22.00

     12    10 Profile Line 10
                .00     -27.00
             400.00     -27.00

     13    12 Profile Line 12
                .00     -40.00
             400.00     -40.00

     14    13 Profile Line 13
                .00     -45.00
             400.00     -45.00


MATERIAL PROPERTIES
     1 T-wall
          0.00 Unit Weight
          Very Strong
     5 Material 5
          108.00 Unit Weight
          Conventional Shear
              400.00       .00
          No Pore Pressure
     6 Material 6
          86.00 Unit Weight
          Interpolate Strengths
              150.00    300.00
          No Pore Pressure
     7 Material 7
          98.00 Unit Weight
          Interpolate Strengths
              150.00    300.00
          No Pore Pressure
     8 Material 8
          100.00 Unit Weight
          Interpolate Strengths
              150.00    300.00
          No Pore Pressure
     9 Material 9
          120.00 Unit Weight
          Conventional Shear
                 .00     30.00
          Piezometric Line
```

```
          1
     10 Material 10
          100.00 Unit Weight
          Conventional Shear
              320.00       .00
          Piezometric Line
          1
     12 Material 12
          100.00 Unit Weight
          Interpolate Strengths
              320.00    450.00
          No Pore Pressure
     13 Material 13
          100.00 Unit Weight
          Conventional Shear
                 .00    450.00
          No Pore Pressure

PIEZOMETRIC LINES
         1      62.40 Water Level
                 .00     17.00
              180.00     17.00
              180.00      1.00
              190.00      1.00
              190.00      8.00
              195.00      8.00
              198.00      7.00
              210.00      5.80
              223.00      2.00
              241.00     -2.80
              271.00     -6.00
              280.00     -6.90
              400.00     -6.90


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Su - Undrained Shear Strength
            .00      2.00    300.00         6
            .00     -7.00    300.00         6
         185.00      2.00    300.00         6
         185.00     -7.00    300.00         6
         225.00      2.00    150.00         6
         225.00     -7.00    150.00         6
         400.00      2.00    150.00         6
         400.00     -7.00    150.00         6
            .00      -7.00    300.00         7
            .00     -10.00    300.00         7
         185.00      -7.00    300.00         7
         185.00     -10.00    300.00         7
         225.00      -7.00    150.00         7
         225.00     -10.00    150.00         7
         400.00      -7.00    150.00         7
         400.00     -10.00    150.00         7
            .00       -40.00    320.00         12
            .00      -45.00    450.00         12
```

```
         185.00      -40.00    320.00         12
         185.00     -45.00    450.00         12
         225.00      -40.00    320.00         12
         225.00     -45.00    450.00         12
         400.00      -40.00    320.00         12
         400.00     -45.00    450.00         12
            .00      -10.00    300.00         8
            .00     -22.00    300.00         8
         185.00      -10.00    300.00         8
         185.00     -22.00    300.00         8
         225.00      -10.00    150.00         8
         225.00     -22.00    270.00         8
         400.00      -10.00    150.00         8
         400.00     -22.00    270.00         8


ANALYSIS/COMPUTATION
     Noncircular Search
         135.00      4.00
         150.00     -3.00
         166.00    -10.00
         190.00    -17.00
         205.00    -20.00
         234.00    -22.00
         262.00    -20.00
         281.00    -16.40
         302.00    -10.00
         312.80     -5.80

           2.00      0.50     50.00
SINgle-stage Computations
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

Attachment 2 – UTexas analysis with unbalanced load that results in Figure 4.

```
HEADING
    T-wall Deep Seated Analysis
    Step 2 Analysis With Unbalanced Load

PROFILE LINES
         1    5 Profile 5
                 .00      3.30
              130.00      3.30
              170.00      4.00
              180.00      4.00

         3    1 T-wall
              180.00      4.00
              186.50      4.00
              186.51     17.00
              188.50     17.00
              188.51      4.00
              190.00      4.00

         2    5 Profile 5 PS
              190.00      8.00
              195.00      8.00
              198.00      7.00
              210.00      5.80
              216.20      4.00
              219.50      3.03
              219.60      3.00
              223.00      2.00

         6    6 Profile 6 - FS
                 .00      2.00
              180.00      2.00

         7    6 Profile 6 - Under Wall
              180.00      1.00
              190.00      1.00

         8    6 Profile 6 - PS
              190.00      2.00
              223.00      2.00
              225.00      1.47
              241.00     -2.80
              271.00     -6.00
              281.00     -7.00

         9    7 Profile 7
                 .00     -7.00
              281.00     -7.00
              295.00     -9.00
              305.00     -9.00
              311.00    -10.00

        10    8 Profile Line 8
                 .00    -10.00
```

```
            311.00    -10.00
            324.00    -11.37
            330.00    -12.00
            337.50    -11.50
            345.00    -11.00
            351.00    -10.50
            358.00     -9.30
            400.00     -9.30

      11     9 Profile Line 9
               .00    -22.00
            400.00    -22.00

      12    10 Profile Line 10
               .00    -27.00
            400.00    -27.00

      13    12 Profile Line 12
               .00    -40.00
            400.00    -40.00

      14    13 Profile Line 13
               .00    -45.00
            400.00    -45.00


MATERIAL PROPERTIES
     1 T-wall
          0.00 Unit Weight
          Very Strong
     5 Material 5
          108.00 Unit Weight
          Conventional Shear
              400.00       .00
          No Pore Pressure
     6 Material 6
          86.00 Unit Weight
          Interpolate Strengths
              150.00    300.00
          No Pore Pressure
     7 Material 7
          98.00 Unit Weight
          Interpolate Strengths
              150.00    300.00
          No Pore Pressure
     8 Material 8
          100.00 Unit Weight
          Interpolate Strengths
              150.00    300.00
          No Pore Pressure
     9 Material 9
          120.00 Unit Weight
          Conventional Shear
                 .00     30.00
          Piezometric Line
          1
     10 Material 10
```

```
          100.00 Unit Weight
          Conventional Shear
              320.00       .00
          Piezometric Line
          1
     12 Material 12
          100.00 Unit Weight
          Interpolate Strengths
              320.00    450.00
          No Pore Pressure
     13 Material 13
          100.00 Unit Weight
          Conventional Shear
                 .00    450.00
          No Pore Pressure

PIEZOMETRIC LINES
         1      62.40 Water Level
                   .00     17.00
                180.00     17.00
                180.00      1.00
                190.00      1.00
                190.00      8.00
                195.00      8.00
                198.00      7.00
                210.00      5.80
                223.00      2.00
                241.00     -2.80
                281.00     -7.00
                400.00     -7.00


DISTRIBUTED LOADS
      1
LINE LOAD
   1 185.0 -9.0 -3800 0 1

INTERPOLATION DATA
Su - Undrained Shear Strength
            .00       2.00    300.00         6
            .00      -7.00    300.00         6
         185.00       2.00    300.00         6
         185.00      -7.00    300.00         6
         225.00       2.00    150.00         6
         225.00      -7.00    150.00         6
         400.00       2.00    150.00         6
         400.00      -7.00    150.00         6
            .00       -7.00    300.00         7
            .00      -10.00    300.00         7
         185.00       -7.00    300.00         7
         185.00      -10.00    300.00         7
         225.00       -7.00    150.00         7
         225.00      -10.00    150.00         7
         400.00       -7.00    150.00         7
         400.00      -10.00    150.00         7
            .00        -40.00    320.00         12
            .00       -45.00    450.00         12
```

```
          185.00      -40.00    320.00          12
          185.00     -45.00    450.00          12
          225.00      -40.00    320.00          12
          225.00     -45.00    450.00          12
          400.00      -40.00    320.00          12
          400.00     -45.00    450.00          12
             .00      -10.00    300.00           8
             .00     -22.00    300.00           8
          185.00      -10.00    300.00           8
          185.00     -22.00    300.00           8
          225.00      -10.00    150.00           8
          225.00     -22.00    270.00           8
          400.00      -10.00    150.00           8
          400.00     -22.00    270.00           8


ANALYSIS/COMPUTATION
     Noncircular  Search
          143.39      3.53
          150.64     -2.36
          164.69    -13.63
          189.61    -18.28
          205.04    -21.72
          234.03    -21.59
          261.62    -17.99
          280.42    -13.65
          301.55     -9.10
          301.65     -9.00

            2.00      0.50     50.00
SINgle-stage Computations
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

UPDATED 23 OCT 07

Attachment 3 Structural Loads for CPGA and Group Analyses

US Army Corps of Engineers	PROJECT TITLE: T-Wall Design Example	COMPUTED BY: KDH	DATE: 07/03/07	SHEET:
Saint Paul Distict	SUBJECT TITLE: Gainard Woods, Pervious	CHECKED BY:	DATE:	

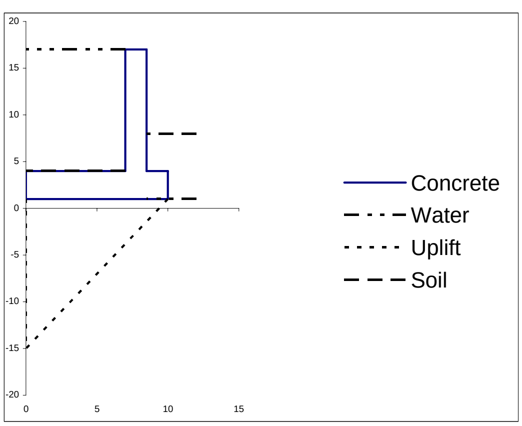
Input for CPGA pile analysis **Pervious Foundation Assumption**

Upstream Water Elevation	17	ft	Back Fill Soil Elevation	4	ft
Downstream Water Elevation	1	ft	Front Fill Soil Elevation	8	ft
Wall Top Elevation	17	ft	Gamma Water	0.0625	kcf
Structure Bottom Elevation	1	ft	Gamma Concrete	0.15	kcf
Base Width	10	ft	Gamma Soil	0.108	kcf
Toe Width	1.5	ft	Distance to Backfill Break	0.0	ft
Wall Thickness	1.5	ft	Slope of Back Fill	0.00	
Base Thickness	3	ft	Soil Elevation at Heel	4.00	ft

Vertical Forces								
Component	Height	x1	x2	Gamma	Force	Arm	Moment	
Stem Concrete	13	7	8.5	0.15	2.93	7.75	22.7	
Heel Concrete	3	0	8.5	0.15	3.83	4.25	16.3	
Toe Concrete	3	8.5	10	0.15	0.68	9.25	6.2	
Heel Water	13	0	7	0.0625	5.69	3.5	19.9	
Toe Water	0	8.5	10	0.0625	0.00	9.25	0.0	
Heel Soil	0	0	7	0.108	0.00	3.5	0.0	
-Triangle	0.00	0	7.0	-0.046	0.00	2.33	0.0	
Toe Soil	4	8.5	10	0.108	0.65	9.25	6.0	
Rect Uplift	0	0	10	0.0625	0.00	5	0.0	
Tri Uplift	-16	0	10	0.0625	-5.00	3.3	-16.7	
Sum Vertical Forces					8.8		54.4	ft-k

Horizontal Forces								
Component	H1	H2	Gamma	Lat. Coeff.	Force	Arm	Moment	
Driving Water	17	1	0.0625	1	8.00	5.33	42.67	
Resisting Water	1	1	0.0625	1	0.00	0.00	0.00	
Driving Soil	4	1	0.046	1	0.20	0.50	0.10	
Resisting Soil	8	1	0.108	1	-2.65	1.83	-4.85	
Sum Horizontal Forces					5.56	6.82	37.92	ft-k

Total Structural Forces	Net Vert. Force	Arm	Moment	
About Heel	8.76	10.54	92.32	ft-k



Net Vertical Arm		ft
From Toe	-0.54	

Moment About Toe
4.7 ft-k

Model Width
5 ft

Calculation of Unbalanced Force

Unbalanced Force. F_{ub}	3,800	lb/ft	From UTexas Analysis
Elevation of Critical Surface	-22	ft	From UTexas Analysis
Length - Ground to Crit. Surface, Lu	26.0	ft	(assume failure surface is normal to pile)
Length - Base to Crit. Surface, Lp	23	ft	
Pile Moment of Inertia. I	904	in^4	
Pile Modulus of Elasticity E	29,000,000	lb/in^2	
Soil Modulus of Subgrade Reaction, k	120	lb/in^2	
Soil Stiffness Parameter, R	122	in	$(EI / k)^{1/4}$
Equivalent Unbalanced Force	2,653	lb/ft	$F_{ub} * (L_u/2 +R) / (L_p +R)$

CPGA Input

PX	-41.06	kips
PY		
PZ	43.80	kips
MX	0	
MY	23.58	kip-ft
MZ	0	

Group Input

2 Pile Rows Parallel to Wall Face

Unbalanced Loading on Piles for Group Analysis

Total	61	lb/in	F_{ub} * Model Width /L_u
50%	30	lb/in	For Pile on Protected Sied
50%	30	lb/in	

Note: Applied to length of pile from bottom of cap to top of critical surface. 23

Unbalanced Loads on Wall for Group Analysis of Just Unbalanced Forces

Distance From Base to Ground Surface, Ds 3.00 ft

PX	0	lb	
PY	2,192	lb	F_{ub} * Model Width / L_u * Ds
PZ	0	lb	
MX	0		
MY	0		
MZ	-39,462	lb-in	-PZ * Ds/2

Total Loads for Group Analysis

PX	43,803	lb	
PY	29,986	lb	PYub + Sum Horizontal * Model Width
PZ	0	lb	
MX	0		
MY	0		
MZ	-322,384	lb-in	

UPDATED 23 OCT 07

US Army Corps of Engineers Saint Paul Distict	PROJECT TITLE: T-Wall Design Example	COMPUTED BY: KDH	DATE: 07/03/07	SHEET:
	SUBJECT TITLE: Gainard Woods, Impervious	CHECKED BY:	DATE:	

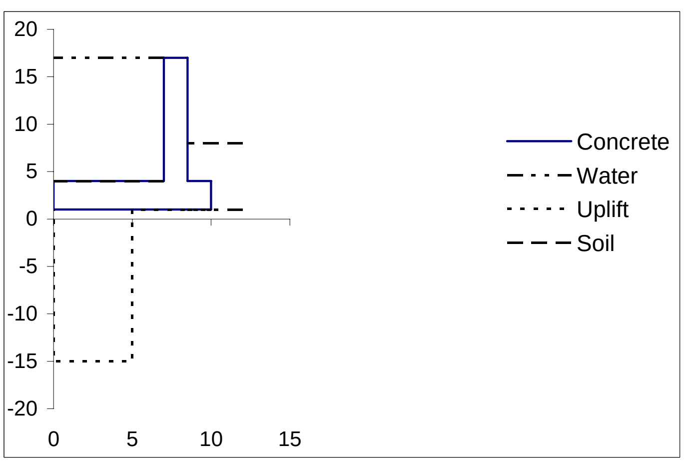
Input for CPGA pile analysis **Impervious Foundation Assumption**

Upstream Water Elevation	17	ft	Back Fill Soil Elevation	4	ft
Downstream Water Elevation	1	ft	Front Fill Soil Elevation	8	ft
Wall Top Elevation	17	ft	Gamma Water	0.0625	kcf
Structure Bottom Elevation	1	ft	Gamma Concrete	0.15	kcf
Base Width	10	ft	Gamma Soil	0.108	kcf
Toe Width	1.5	ft	Distance to Backfill Break	0.0	ft
Wall Thickness	1.5	ft	Slope of Back Fill	0.00	
Base Thickness	3	ft	Soil Elevation at Heel	4.00	ft

Vertical Forces								
Component	Height	x1	x2	Gamma	Force	Arm	Moment	
Stem Concrete	13	7	8.5	0.15	2.93	7.75	22.7	
Heel Concrete	3	0	8.5	0.15	3.83	4.25	16.3	
Toe Concrete	3	8.5	10	0.15	0.68	9.25	6.2	
Heel Water	13	0	7	0.0625	5.69	3.5	19.9	
Toe Water	0	8.5	10	0.0625	0.00	9.25	0.0	
Heel Soil	0	0	7	0.108	0.00	3.5	0.0	
-Triangle	0.00	0	7.0	-0.046	0.00	2.33	0.0	
Toe Soil	4	8.5	10	0.108	0.65	9.25	6.0	
Prot. Side Uplift	0	5	10	0.0625	0.00	7.5	0.0	
Flood Side Uplift	-16	0	5	0.0625	-5.00	2.5	-12.5	
Sum Vertical Forces					8.8	kip	58.6	ft-k

Horizontal Forces								
Component	H1	H2	Gamma	Lat. Coeff.	Force	Arm	Moment	
Driving Water	17	1	0.0625	1	8.00	5.33	42.67	
Resisting Water	1	1	0.0625	1	0.00	0.00	0.00	
Driving Soil	4	1	0.046	1	0.20	0.50	0.10	
Resisting Soil	8	1	0.108	1	-2.65	1.83	-4.85	
Sum Horizontal Forces					5.56	kip	37.92	ft-k

Total Structural Forces	Net Vert. Force	Arm	Moment	
About Heel	8.76	11.01	96.49	ft-k



Net Vertical Arm		
From Toe	-1.01	ft

Moment About Toe
8.9 ft-k

Model Width
5 ft

Calculation of Unbalanced Force

Unbalanced Force. F_{ub}	3,800	lb/ft	From UTexas Analysis
Elevation of Critical Surface	-22	ft	From UTexas Analysis
Length - Ground to Crit. Surface, Lu	26	ft	(assume failure surface is normal to pile)
Length - Base to Crit. Surface, Lp	23	ft	
Pile Moment of Inertia. I	904	in^4	HP14x73
Pile Modulus of Elasticity E	29,000,000	lb/in^2	
Soil Modulus of Subgrade Reaction, k	120	lb/in^2	
Soil Stiffness Parameter, R	122	in	$(EI / k)^{1/4}$
Equivalent Unbalanced Force	2,653	lb/ft	$F_{ub} * (L_u/2 + R) / (L_p + R)$

CPGA Input

PX	-41.06	kips
PY		
PZ	43.80	kips
MX	0	
MY	44.41	kip-ft
MZ	0	

Group Input

2 Pile Rows Parallel to Wall Face

Unbalanced Loading on Piles for Group Analysis

Total	61	lb/in	F_{ub} * Model Width /L_u
50%	30	lb/in	For Pile on Protected Sied
50%	30	lb/in	

Note: Applied to length of pile from bottom of cap to top of critical surface. 23 ft

Unbalanced Loads on Wall for Group Analysis of Just Unbalanced Forces

Distance From Base to Ground Surface, Ds 3.00 ft

PX	0	lb	
PY	2,192	lb	F_{ub} * Model Width / L_u * Ds
PZ	0	lb	
MX	0		
MY	0		
MZ	-39,462	lb-in	-PZ * Ds/2

Total Loads for Group Analysis

PX	43,803	lb	
PY	29,986	lb	PYub + Sum Horizontal * Model Width
PZ	0	lb	
MX	0		
MY	0		
MZ	-572,384	lb-in	

Attachment 4 - Preliminary Analysis with CPGA

Input File

```
10 Gainard Woods T-wall, Example
15 3.0 ft slab, hp 14 x 89 piles, pinned head,
20 PROP 29000 326 904 26.1 0.5 0 all
30 SOIL ES 0.046 "TIP" 100 0 all
40 PIN all
50 ALLOW H 65.0 65.0 362.5  362.5  1108  3275 all
70 BATTER 2 1 2
80 ANGLE 180 1
180 PILE 1  1.2500 0.00 0.00
201 PILE 2  8.750 0.00 0.00
230 LOAD 1 -41.06  0.0  43.8  0.00  23.58
240 LOAD 2 -41.06  0.0  43.8  0.00  44.41
334 FOUT 1 2 3 4 5 6 7 GWex3.out
335 PFO ALL
```

Output

```
*********************************
 * CASE PROGRAM   #  X0080       *  CPGA - CASE PILE GROUP ANALYSIS PROGRAM
 * VERSION NUMBER # 1993/03/29   *  RUN DATE 27-JUL-2007   RUN TIME 12.58.29
 *********************************


 GAINARD WOODS T-WALL, EXAMPLE


 THERE ARE     2 PILES AND
               2 LOAD CASES IN THIS RUN.

 ALL PILE COORDINATES ARE CONTAINED WITHIN A BOX
                                    X          Y          Z
                                  -----      -----      -----
 WITH DIAGONAL COORDINATES = (      1.25 ,      .00 ,      .00 )
                             (      8.75 ,      .00 ,      .00 )


 ******************************************************************************

          PILE PROPERTIES AS INPUT


      E            I1            I2            A            C33            B66
     KSI          IN**4         IN**4         IN**2
  .29000E+05   .32600E+03   .90400E+03   .26100E+02   .50000E+00   .00000E+00

 THESE PILE PROPERTIES APPLY TO THE FOLLOWING PILES -

     ALL


 ******************************************************************************
```

SOIL DESCRIPTIONS AS INPUT

ES	ESOIL	LENGTH	L	LU
	K/IN**2		FT	FT
	.46000E-01	T	.10000E+03	.00000E+00

THIS SOIL DESCRIPTION APPLIES TO THE FOLLOWING PILES -

ALL

****

PILE GEOMETRY AS INPUT AND/OR GENERATED

NUM	X	Y	Z	BATTER	ANGLE	LENGTH	FIXITY
	FT	FT	FT			FT	
1	1.25	.00	.00	2.00	180.00	111.80	P
2	8.75	.00	.00	2.00	.00	111.80	P

						223.61	

****

APPLIED LOADS

LOAD CASE	PX K	PY K	PZ K	MX FT-K	MY FT-K	MZ FT-K
1	-41.1	.0	43.8	.0	23.6	.0
2	-41.1	.0	43.8	.0	44.4	.0

****

ORIGINAL PILE GROUP STIFFNESS MATRIX

.12087E+03	.49431E-05	.41211E-12	.00000E+00	-.99740E+04	.74146E-04
.49431E-05	.77891E+01	-.96883E-05	.00000E+00	.14533E-03	.46735E+03
.41211E-12	-.96883E-05	.45334E+03	.00000E+00	-.27200E+05	-.14533E-03
.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.00000E+00
-.99740E+04	.14533E-03	-.27200E+05	.00000E+00	.25500E+07	.21799E-02
.74146E-04	.46735E+03	-.14533E-03	.00000E+00	.21799E-02	.43814E+05

S(4,4)=0. PROBLEM WILL BE TREATED AS TWO DIMENSIONAL IN THE X-Z PLANE.

LOAD CASE 1. NUMBER OF FAILURES = 0. NUMBER OF PILES IN TENSION = 1.

LOAD CASE 2. NUMBER OF FAILURES = 0. NUMBER OF PILES IN TENSION = 1.

****

```
          PILE CAP DISPLACEMENTS

LOAD
CASE        DX           DZ           R
            IN           IN          RAD

   1    -.7541E+00  -.2047E+00  -.5023E-02
   2    -.5370E+00  -.4687E-01  -.2391E-02

*******************************************************************************
               ELASTIC CENTER INFORMATION


ELASTIC CENTER IN PLANE X-Z          X             Z
                                    FT            FT
                                   5.00         -6.88

LOAD     MOMENT IN
CASE     X-Z PLANE
   1   .76399E+04
   2   .30736E+05

*******************************************************************************

          PILE FORCES IN LOCAL GEOMETRY

              M1 & M2 NOT AT PILE HEAD FOR PINNED PILES
              * INDICATES PILE FAILURE
              # INDICATES CBF BASED ON MOMENTS DUE TO
                          (F3*EMIN) FOR CONCRETE PILES
              B INDICATES BUCKLING CONTROLS


LOAD CASE -     1

PILE     F1       F2       F3         M1         M2         M3   ALF   CBF
          K        K        K        IN-K       IN-K       IN-K

   1      3.7       .0     62.5        .0     -259.0        .0   .96   .25
   2     -4.1       .0    -13.7        .0      289.5        .0   .21   .13


LOAD CASE -     2

PILE     F1       F2       F3         M1         M2         M3   ALF   CBF
          K        K        K        IN-K       IN-K       IN-K

   1      2.4       .0     65.0        .0     -171.6        .0  1.00   .23
   2     -2.9       .0    -16.2        .0      202.1        .0   .25   .11

*******************************************************************************
```

PILE FORCES IN GLOBAL GEOMETRY

LOAD CASE - 1

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	-31.2	.0	54.2	.0	.0	.0
2	-9.8	.0	-10.4	.0	.0	.0

LOAD CASE - 2

PILE	PX K	PY K	PZ K	MX IN-K	MY IN-K	MZ IN-K
1	-31.2	.0	57.0	.0	.0	.0
2	-9.8	.0	-13.2	.0	.0	.0

Attachment 5 – Group 7 Summary Output for Pervious Condition

```
===============================================================================

                GROUP for Windows, Version 7.0.7

                 Analysis of A Group of Piles
              Subjected to Axial and Lateral Loading

               (c) Copyright ENSOFT, Inc., 1987-2006
                     All Rights Reserved

===============================================================================


This program is licensed to:

k
c

Path to file locations:      C:\KDH\New Orleans\T-walls\Group\Adeles\
Name of input data file:     GW Example Perv 3.gpd
Name of output file:         GW Example Perv 3.gpo
Name of plot output file:    GW Example Perv 3.gpp
Name of runtime file:        GW Example Perv 3.gpr
Name of output summary file: GW Example Perv 3.gpt


-------------------------------------------------------------------------------
                          Time and Date of Analysis
-------------------------------------------------------------------------------

               Date:  July  9, 2007     Time:  16:21:51
 PILE GROUP ANALYSIS PROGRAM-GROUP
 PC VERSION 6.0 (C) COPYRIGHT ENSOFT,INC. 2000

 THE PROGRAM WAS COMPILED USING MICROSOFT FORTRAN
 POWERSTATION 4.0 (C) COPYRIGHT MICROSOFT CORPORATION, 1996.


     Gainard Woods:  F.S. 17.0, P.S. 1, Pervious


                *****     INPUT INFORMATION     *****



     * TABLE C *   LOAD AND CONTROL PARAMETERS


     UNITS--

          V LOAD,LBS      H LOAD,LBS     MOMENT,LBS-IN
```

```
     0.4380E+05      0.2999E+05     -0.5724E+06

  GROUP NO. 1


               DISTRIBUTED LOAD CURVE           2 POINTS

                          X,IN       LOAD,LBS/IN
                          0.00         0.310E+02
                        308.00         0.310E+02

  GROUP NO. 2


               DISTRIBUTED LOAD CURVE           2 POINTS

                          X,IN       LOAD,LBS/IN
                          0.00         0.310E+02
                        308.00         0.310E+02


* THE LOADING IS STATIC *


    KPYOP =  0      (CODE TO GENERATE P-Y CURVES)

    ( KPYOP = 1 IF P-Y YES; = 0 IF P-Y NO; = -1 IF P-Y ONLY )


* CONTROL PARAMETERS *
    TOLERANCE ON CONVERGENCE OF FOUNDATION REACTION      =  0.100E-04 IN
    TOLERANCE ON DETERMINATION OF DEFLECTIONS            =  0.100E-04 IN
    MAX NO OF ITERATIONS ALLOWED FOR FOUNDATION ANALYSIS =     100
    MAXIMUM NO. OF ITERATIONS ALLOWED FOR PILE ANALYSIS  =     100



* TABLE D *   ARRANGEMENT OF PILE GROUPS

  GROUP  CONNECT  NO OF PILE PILE NO  L-S CURVE  P-Y CURVE
    1      PIN         1        1         1           0
    2      PIN         1        1         1           0

  GROUP         VERT,IN     HOR,IN    SLOPE,IN/IN  GROUND,IN SPRING,LBS-IN



* TABLE E *   PILE GEOMETRY AND PROPERTIES
             PILE TYPE = 1 - DRIVEN PILE
                       = 2 - DRILLED SHAFT

  PILE  SEC  INC       LENGTH, IN     E  ,LBS/IN**2 PILE TYPE
    1    1   100       0.1006E+04     0.2900E+08        1

  PILE   FROM,IN      TO,IN      DIAM,IN   AREA,IN**2    I,IN**4
```

```
    1  0.0000E+00  0.1006E+04  0.1400E+02  0.2610E+02  0.9040E+03

      * THE PILE ABOVE IS OF LINEARLY ELASTIC MATERIAL *


* TABLE F *   AXIAL LOAD VS SETTLEMENT

(THE LOAD-SETTLEMENT CURVE OF SINGLE PILE IS GENERATED INTERNALLY)

  NUM OF CURVES  1

   CURVE  1          NUM OF POINTS = 19

       POINT        AXIAL LOAD,LBS      SETTLEMENT, IN
         1          -0.8554E+05         -0.2075E+01
         2          -0.8546E+05         -0.1075E+01
         3          -0.8542E+05         -0.5748E+00
         4          -0.8888E+05         -0.1784E+00
         5          -0.8583E+05         -0.1246E+00
         6          -0.2191E+05         -0.2768E-01
         7          -0.1092E+05         -0.1377E-01
         8          -0.2183E+04         -0.2753E-02
         9          -0.2183E+03         -0.2753E-03
        10           0.0000E+00          0.0000E+00
        11           0.2185E+03          0.2755E-03
        12           0.2185E+04          0.2755E-02
        13           0.1093E+05          0.1377E-01
        14           0.2193E+05          0.2769E-01
        15           0.8589E+05          0.1247E+00
        16           0.8897E+05          0.1785E+00
        17           0.8576E+05          0.5753E+00
        18           0.8595E+05          0.1075E+01
        19           0.8624E+05          0.2076E+01


* TABLE H *   SOIL DATA FOR AUTO P-Y CURVES


SOILS INFORMATION

     AT THE GROUND SURFACE          =     -36.00 IN

    8 LAYER(S) OF SOIL
    THE SOIL IS A SOFT CLAY
    X AT THE TOP OF THE LAYER         =     -36.00 IN
    X AT THE BOTTOM OF THE LAYER      =     -12.00 IN
    MODULUS OF SUBGRADE REACTION      =  0.300E+02 LBS/IN**3
    THE SOIL IS A SOFT CLAY
    X AT THE TOP OF THE LAYER         =     -12.00 IN
    X AT THE BOTTOM OF THE LAYER      =      96.00 IN
    MODULUS OF SUBGRADE REACTION      =  0.300E+02 LBS/IN**3
    THE SOIL IS A SOFT CLAY
    X AT THE TOP OF THE LAYER         =      96.00 IN
    X AT THE BOTTOM OF THE LAYER      =     132.00 IN
```

```
MODULUS OF SUBGRADE REACTION     =  0.300E+02 LBS/IN**3
THE SOIL IS A SOFT CLAY
X AT THE TOP OF THE LAYER        =     132.00 IN
X AT THE BOTTOM OF THE LAYER     =     276.00 IN
MODULUS OF SUBGRADE REACTION     =  0.300E+02 LBS/IN**3
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER        =     276.00 IN
X AT THE BOTTOM OF THE LAYER     =     336.00 IN
MODULUS OF SUBGRADE REACTION     =  0.300E+02 LBS/IN**3
THE SOIL IS A SOFT CLAY
X AT THE TOP OF THE LAYER        =     336.00 IN
X AT THE BOTTOM OF THE LAYER     =     492.00 IN
MODULUS OF SUBGRADE REACTION     =  0.300E+02 LBS/IN**3
THE SOIL IS A SOFT CLAY
X AT THE TOP OF THE LAYER        =     492.00 IN
X AT THE BOTTOM OF THE LAYER     =     552.00 IN
MODULUS OF SUBGRADE REACTION     =  0.300E+02 LBS/IN**3
THE SOIL IS A STIFF CLAY BELOW THE WATER TABLE
X AT THE TOP OF THE LAYER        =     552.00 IN
X AT THE BOTTOM OF THE LAYER     =    1440.00 IN
MODULUS OF SUBGRADE REACTION     =  0.300E+02 LBS/IN**3
```

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
12 POINTS

X,IN	WEIGHT,LBS/IN**3
-36.0000	0.2600E-01
-12.0000	0.2600E-01
-12.0000	0.1400E-01
96.0000	0.1400E-01
96.0000	0.2000E-01
132.0000	0.2000E-01
132.0000	0.2200E-01
276.0000	0.2200E-01
276.0000	0.3300E-01
336.0000	0.3300E-01
336.0000	0.2200E-01
1440.0000	0.2200E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
16 POINTS

X IN	C LBS/IN**2	PHI,DEGREES	E50	FMAX LBS/IN**2	TIPMAX LBS/IN**2
-36.00	0.1890E+01	0.000	0.2000E-01	0.1000E+00	0.0000E+00
-12.00	0.1890E+01	0.000	0.2000E-01	0.1000E+00	0.0000E+00
-12.00	0.1420E+01	0.000	0.2000E-01	0.1000E+00	0.0000E+00
96.00	0.1420E+01	0.000	0.2000E-01	0.1000E+00	0.0000E+00
96.00	0.1420E+01	0.000	0.2000E-01	0.1000E+00	0.0000E+00
132.00	0.1420E+01	0.000	0.2000E-01	0.1000E+00	0.0000E+00
132.00	0.1420E+01	0.000	0.2000E-01	0.1000E+00	0.0000E+00
276.00	0.1420E+01	0.000	0.2000E-01	0.1000E+00	0.0000E+00
276.00	0.0000E+00	30.000	0.0000E+00	0.1500E+01	0.0000E+00
336.00	0.0000E+00	30.000	0.0000E+00	0.1700E+01	0.0000E+00
336.00	0.2220E+01	0.000	0.2000E-01	0.2220E+01	0.0000E+00

492.00	0.2220E+01	0.000	0.2000E-01	0.2220E+01	0.0000E+00
492.00	0.3130E+01	0.000	0.2000E-01	0.3130E+01	0.0000E+00
552.00	0.3130E+01	0.000	0.2000E-01	0.3130E+01	0.0000E+00
552.00	0.3130E+01	0.000	0.2000E-01	0.3130E+01	0.0000E+00
1440.00	0.3130E+01	0.000	0.2000E-01	0.3130E+01	0.0000E+00

REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS

GROUP NO	P-FACTOR	Y-FACTOR
1	1.00	1.00
2	0.97	1.00

Gainard Woods: F.S. 17.0, P.S. 1, Pervious

***** COMPUTATION RESULTS *****

VERT. LOAD, LBS	HORI. LOAD, LBS	MOMENT,IN-LBS
0.4380E+05	0.2999E+05	-0.5724E+06

DISPLACEMENT OF GROUPED PILE FOUNDATION

VERTICAL,IN	HORIZONTAL,IN	ROTATION,RAD
0.1133E+00	0.9562E-02	-0.1230E-02

NUMBER OF ITERATIONS = 5

* TABLE I * COMPUTATION ON INDIVIDUAL PILE

* PILE GROUP * 1

PILE TOP DISPLACEMENTS AND REACTIONS

THE GLOBAL STRUCTURE COORDINATE SYSTEM

XDISPL,IN	YDISPL,IN	SLOPE	AXIAL,LBS	LAT,LBS	BM,LBS-IN	STRESS,LBS/IN**2
0.949E-01	0.956E-02	0.600E-03	0.575E+05	0.246E+05	0.000E+00	0.239E+04

```
        THE LOCAL MEMBER COORDINATE SYSTEM
        ------------------------------------

 XDISPL,IN    YDISPL,IN    SLOPE    AXIAL,LBS   LAT,LBS   BM,LBS-IN   STRESS,LBS/IN**2

 0.891E-01 -0.339E-01 0.600E-03 0.624E+05-0.368E+04 0.000E+00   0.239E+04


        * PILE GROUP *   2


 PILE TOP DISPLACEMENTS AND REACTIONS


        THE GLOBAL STRUCTURE COORDINATE SYSTEM
        --------------------------------------

 XDISPL,IN    YDISPL,IN    SLOPE    AXIAL,LBS   LAT,LBS   BM,LBS-IN   STRESS,LBS/IN**2

-0.158E-01  0.956E-02 0.211E-03-0.137E+05 0.534E+04 0.000E+00   0.560E+03

        THE LOCAL MEMBER COORDINATE SYSTEM
        ------------------------------------

 XDISPL,IN    YDISPL,IN    SLOPE    AXIAL,LBS   LAT,LBS   BM,LBS-IN   STRESS,LBS/IN**2

-0.184E-01  0.147E-02 0.211E-03-0.146E+05-0.134E+04 0.000E+00   0.560E+03
```

Attachment 6 – UTexas analysis with piles as reinforcement (Figure 12).

```
HEADING
    T-wall Deep Seated Analysis
    Step 7 Check with Group 7 Pile Forces

PROFILE LINES
         1    5 Profile 5
                .00       3.30
             130.00       3.30
             170.00       4.00
             180.00       4.00

         3    1 T-wall
             180.00       4.00
             186.50       4.00
             186.51      17.00
             188.50      17.00
             188.51       4.00
             190.00       4.00

         2    5 Profile 5 PS
             190.00       8.00
             195.00       8.00
             198.00       7.00
             210.00       5.80
             216.20       4.00
             219.50       3.03
             219.60       3.00
             223.00       2.00

         6    6 Profile 6 - FS
                .00       2.00
             180.00       2.00

         7    6 Profile 6 - Under Wall
             180.00       1.00
             190.00       1.00

         8    6 Profile 6 - PS
             190.00       2.00
             223.00       2.00
             225.00       1.47
             241.00      -2.80
             271.00      -6.00
             281.00      -7.00

         9    7 Profile 7
                .00      -7.00
             281.00      -7.00
             295.00      -9.00
             305.00      -9.00
             311.00     -10.00

        10    8 Profile Line 8
                .00     -10.00
```

```
           311.00    -10.00
           324.00    -11.37
           330.00    -12.00
           337.50    -11.50
           345.00    -11.00
           351.00    -10.50
           358.00     -9.30
           400.00     -9.30

     11     9 Profile Line 9
              .00    -22.00
           400.00    -22.00

     12    10 Profile Line 10
              .00    -27.00
           400.00    -27.00

     13    12 Profile Line 12
              .00    -40.00
           400.00    -40.00

     14    13 Profile Line 13
              .00    -45.00
           400.00    -45.00


MATERIAL PROPERTIES
     1 T-wall
          0.00 Unit Weight
          Very Strong
     5 Material 5
          108.00 Unit Weight
          Conventional Shear
              400.00        .00
          No Pore Pressure
     6 Material 6
          86.00 Unit Weight
          Interpolate Strengths
              150.00     300.00
          No Pore Pressure
     7 Material 7
          98.00 Unit Weight
          Interpolate Strengths
              150.00     300.00
          No Pore Pressure
     8 Material 8
          100.00 Unit Weight
          Interpolate Strengths
              150.00     300.00
          No Pore Pressure
     9 Material 9
          120.00 Unit Weight
          Conventional Shear
                 .00      30.00
          Piezometric Line
          1
     10 Material 10
```

```
          100.00 Unit Weight
          Conventional Shear
              320.00       .00
          Piezometric Line
          1
     12 Material 12
          100.00 Unit Weight
          Interpolate Strengths
              320.00    450.00
          No Pore Pressure
     13 Material 13
          100.00 Unit Weight
          Conventional Shear
                 .00    450.00
          No Pore Pressure

PIEZOMETRIC LINES
         1      62.40 Water Level
                 .00     17.00
              180.00     17.00
              180.00      1.00
              190.00      1.00
              190.00      8.00
              195.00      8.00
              198.00      7.00
              210.00      5.80
              223.00      2.00
              241.00     -2.80
              281.00     -7.00
              400.00     -7.00


DISTRIBUTED LOADS
      1
REINFORCEMENT LINES
      1        .00          2
140.50 -80.00  292.   2020.
181.00    1.00  292.  2020.

      2         .00         2
189.00    1.00 -78.  1840.
229.50 -80.00  -78.  1840.

      3         .00         2
5.00       1.00  0.   0.
5.00    -10.50   0.   0.

INTERPOLATION DATA
Su - Undrained Shear Strength
            .00      2.00    300.00         6
            .00     -7.00    300.00         6
         185.00      2.00    300.00         6
         185.00     -7.00    300.00         6
         225.00      2.00    150.00         6
         225.00     -7.00    150.00         6
         400.00      2.00    150.00         6
         400.00     -7.00    150.00         6
```

```
            .00      -7.00    300.00         7
            .00     -10.00    300.00         7
         185.00      -7.00    300.00         7
         185.00     -10.00    300.00         7
         225.00      -7.00    150.00         7
         225.00     -10.00    150.00         7
         400.00      -7.00    150.00         7
         400.00     -10.00    150.00         7
            .00      -40.00    320.00         12
            .00     -45.00    450.00         12
         185.00      -40.00    320.00         12
         185.00     -45.00    450.00         12
         225.00      -40.00    320.00         12
         225.00     -45.00    450.00         12
         400.00      -40.00    320.00         12
         400.00     -45.00    450.00         12
            .00      -10.00    300.00         8
            .00     -22.00    300.00         8
         185.00      -10.00    300.00         8
         185.00     -22.00    300.00         8
         225.00      -10.00    150.00         8
         225.00     -22.00    270.00         8
         400.00      -10.00    150.00         8
         400.00     -22.00    270.00         8


ANALYSIS/COMPUTATION
     Noncircular
         143.39      3.53
         150.64     -2.36
         164.69    -13.63
         189.61    -18.28
         205.04    -21.72
         234.03    -21.59
         261.62    -17.99
         280.42    -13.65
         301.55     -9.10
         301.65     -9.00

SINgle-stage Computations
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
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