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Hurricane and Storm Damage Reduction System Design Guidelines

INTERIM

New Orleans District Engineering Division
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of Engineers®

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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.

PART A:
DESIGN GUIDELINES

1.0 HYDRAULICS

1.1 Design Philosophy for Preliminary Design of Hurricane Protection System

This chapter presents the hydraulic design approach to determine protection system design elevations sufficient to provide protection from a hurricane event that would produce a 1% exceedence surge elevation and associated waves. This surge elevation has a one-percent chance of being equaled or exceeded during any year. The protection system design elevations, referenced in this document as the 1% exceedence design elevations, have been developed for two authorized hurricane protection projects in the New Orleans area: Lake Pontchartrain, LA & Vicinity; and West Bank & Vicinity (see Figure 1.1).

An extensive USACE/FEMA internal review and ASCE external review has been conducted on the approach during the period March through August 2007. The review documents can be found in USACE/FEMA South East Louisiana Joint Surge Study Independent Technical Review (Draft report 15 August 2007) and ASCE One percent Review Team (OPRT), Report Number 1 (31 May 2007) and 2 (30 July 2007).

Initial design elevations for Lake Pontchartrain, LA & Vicinity; and West Bank & Vicinity projects can be found in the report, “Elevations for Design of Hurricane Protection Levees and Structures,” dated September 2007. Hydraulic design and analysis associated with upcoming investigations will be documented in engineering analysis reports and also in addenda to the report. All hydraulic analyses associated with the two protection systems can be found in one comprehensive document.

To assure continuity of design methodology and provide close quality management, final design elevations utilized throughout the New Orleans area will be reviewed by the New Orleans District Engineering Division Chief of Hydraulics and Hydrologic Branch.

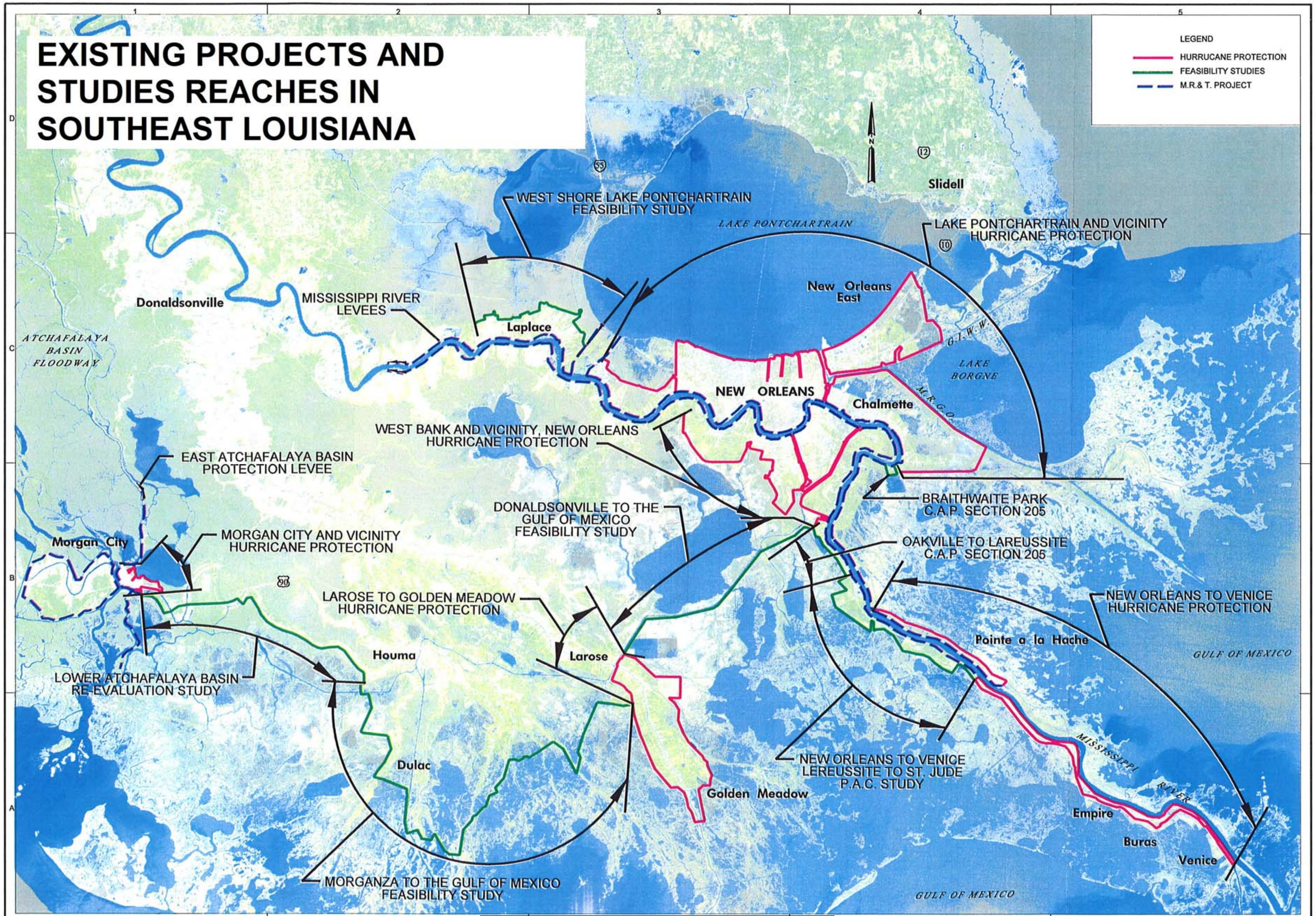


Figure 1.1 Map of existing projects and studies

1.2 Input Data and Methods for Design Approach

1.2.1 JPM-OS process

In 2006 and 2007, a team of Corps of Engineers, FEMA, NOAA, private sector, and academia developed a new process for estimating hurricane inundation probabilities, the Joint Probability Method with Optimal Sampling process (JPM-OS), see Resio (2007). This work was initiated for the Louisiana Coastal Protection and Restoration study (LACPR), but now is being applied to Corps work under the 4th supplemental appropriation, Interagency Performance Evaluation Team (IPET) risk analysis, and FEMA Base Flood Elevations for production of DFIRMs for coastal Louisiana and Texas. The Corps and FEMA work use the same model grids, the same model software, the same model input, such as wind fields, and the same method for estimating hurricane inundation probabilities. The JPM-OS process is shown in Figure 1.2. A more detailed description of the process and the modeling can be found in the White Paper, "Estimating Hurricane Inundation Probabilities" and documents prepared for FEMA for the coastal base flood elevation work.

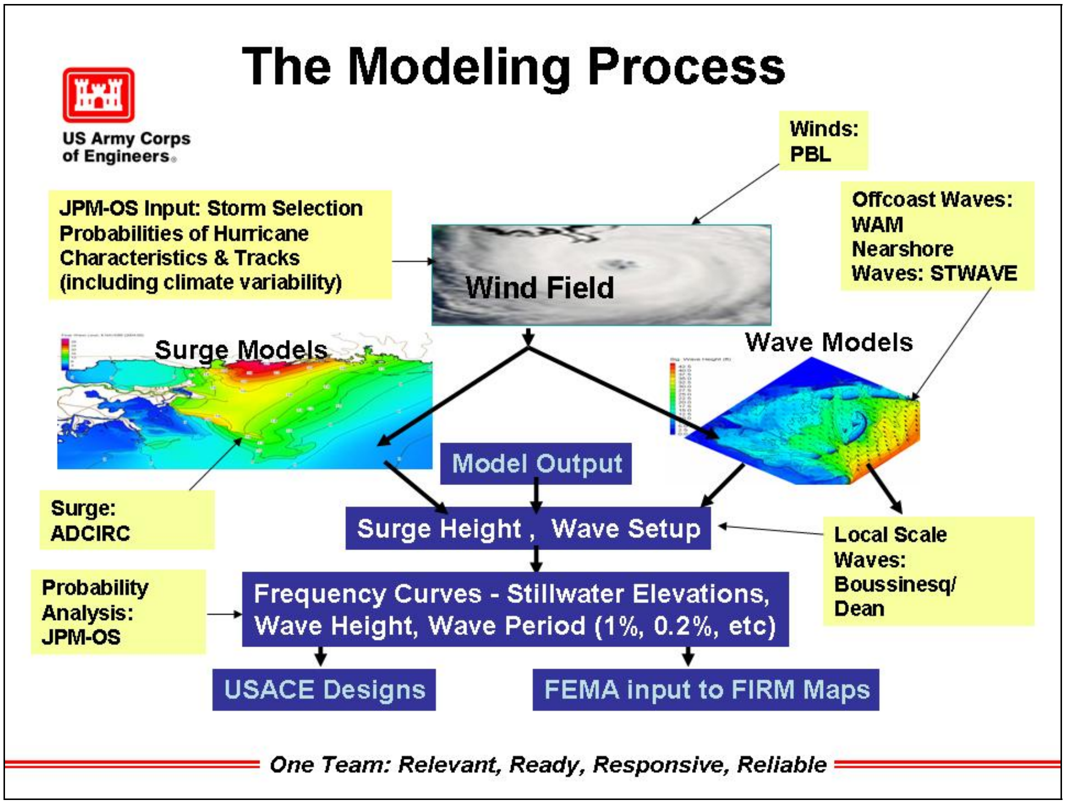


Figure 1.2 – The different components and their interaction in the JPM-OS Process

1.2.2 Modeling process

The following models are used in the JPM-OS process:

PBL – Planetary Boundary Layer Model. A marine planetary boundary layer model which links marine wind profiles to large scale pressure gradients and thermal properties has been developed by Oceanweather, Inc. Oceanweather, Inc is an internationally known company serving the international shipping, offshore industry and coastal engineering communities.

ADCIRC – Advanced Circulation Model. The ADCIRC model is used for the surge modeling. ADCIRC was developed by the ADCIRC Development Group which includes representatives from the University of North Carolina, the University of Oklahoma, the University of Notre Dame, and the University of Texas. The New Orleans District (MVN) is a development partner with the ADCIRC Development Group. The ADCIRC Model is a state-of-the-art model that solves the generalized wave-continuity equation on linear triangular elements. For the coastal Louisiana modeling, the finite element grid contains approximately 2.1 million horizontal nodes and 4.2 million elements.

WAM - The global ocean WAve prediction Model called WAM is a third generation wave model developed by the Corps of Engineers Coastal and Hydraulics Laboratory (CHL). WAM is used for offshore waves and boundary conditions for the nearshore wave modeling. WAM predicts directional spectra as well as wave properties such as significant wave height, mean wave direction and frequency, swell wave height and mean direction, and wind stress fields corrected by including the wave induced stress and the drag coefficient at each grid point at chosen output times.

STWAVE – Steady State Spectral Wave Model. STWAVE is a nearshore wave model developed by CHL. For the JPM-OS effort, STWAVE is used to generate the nearshore wave heights and wave periods using boundary conditions from the WAM modeling. The WAM-to-STWAVE procedure is applied for each storm. For the analyses completed to date, the STWAVE model did not include frictional effects.

The JPM-OS modeling process is as follows (see also Figure 1.2). The PBL model is used to generate the wind fields required in the JPM-OS process. For each storm, the PBL model is used to construct 15-minute snapshots of wind and pressure fields for driving the surge and wave models. ADCIRC, WAM, and STWAVE model runs are performed on high speed computers at the Corps of Engineers Engineering Research and Development Center (ERDC) in Vicksburg, MS, the Lonestar computer at University of Texas, and similar computers. With all major rivers already "spun up", the surge model ADCIRC is initiated assuming zero tide. The spectral deep water wave model WAM is run, in parallel with the initial ADCIRC run, to establish the directional wave spectra that serve as the boundary conditions for the near-coast wave model, STWAVE. The STWAVE model is used to produce the wave fields and estimated radiation stress fields. These stress fields, added to the PBL estimated wind stresses, are used in the ADCIRC model for the time period during which the radiation stress makes a significant contribution to the water levels.

Two conditions of the hurricane protection system have been modeled with ADCIRC/STWAVE for design purposes: 2007 condition and 2010 condition. The **2007 condition** considers the interim gates and closures at the three outfall canals and levees and floodwalls constructed to pre-Katrina authorized elevations. The **2010 condition** considers the permanent gates and closures at the three outfall canals, the gate on the GIWW/MRGO, and levees and floodwalls constructed to elevations at or greater than the preliminary 1% design elevations. For the 2010 runs, no gate is present at Seabrook.

For most Joint Probability Methods, several thousand events are evaluated. With the JPM-OS method, optimal sampling allows for a smaller number of events to be used. Based on optimized sampling, 152 hurricane events were modeled for the 2007 condition, and 56 hurricane events have been modeled for the 2010 condition. For the 2010 condition, the output from the 56 storms have been used with 96 storms from the 2007 condition to create a dataset of 152 storms required for the frequency analysis. A relationship has been determined from the two sets of conditions and applied to achieve a consistent dataset.

The 2007 results from ADCIRC and STWAVE have been used for Lake Pontchartrain Lakefront area and the West Bank. This area is not affected by the gates at GIWW/MRGO. The 2010 model results have been used for the analysis of the GIWW/MRGO gate were applied to the levee/floodwall sections starting from South Point to GIWW, the GIWW sections outside the gate and the St Bernard levee sections. In addition to that, the levee/floodwall sections of the GIWW and IHNC inside the gate with no Seabrook Gate have utilized the ADCIRC results.

1.2.3 Frequency Analysis

The output from the ADCIRC and STWAVE models used in the frequency analysis are the maximum surge elevation and maximum wave characteristics (significant wave height, peak period, and wave direction) at approximately 600 feet in front of the levee or floodwall. Typical parameters which are to be computed based on the surge level and the wave characteristics are the wave run-up and the overtopping rate. These parameters depend also on the levee geometry (i.e. levee height and levee slope). The determination of the wave overtopping will be discussed in Section 1.2.4.

An example of the model output at two locations within the hurricane protection system is shown in Figure 1.3. The wave characteristics along Lake Pontchartrain are typically wind-generated and depth-limited waves. There is a high correlation between the wave height and the wave period and between the surge level and wave height for this area. In contrast, the results at the MRGO are much more scattered. The relationship between the surge level and the wave height is less evident, and the wave period strongly varies as a function of the wave height. Long wave periods are observed for a few storm conditions. The long wave periods are related to swell waves from the ocean.

A probabilistic model is used to derive the surge elevation, wave height, and wave period frequency curves at specific points along the hurricane protection system using output from ADCIRC and STWAVE. This probabilistic model takes into account the joint probability of forward speed, size, central pressure, angle of approach and geographic distribution of the hurricanes. For more information, the reader is referred to Resio (2007).

Surge frequency curves are estimated from the ADCIRC output of the 152 storms for 2007 and 2010 conditions. There may be instances where there is no output from the 152 storms. In this case, estimates are to be made of the surge elevation for the missing output so that the frequency analysis continued to be based on 152 values. The resulting 1% surge levels are considered to be “best estimate” values. In addition to the best estimates, the probabilistic model also provides an error estimate of the 1% surge levels. Errors are generally in the order of 1 – 2 ft for the 1% surge levels.

The same methodology is also used to develop frequency curves for wave height and wave period. Examples of frequency curves can be found in Figure 1.4. The errors in the 1% wave height and wave period have been based on expert judgment (Smith, pers. comm.). The standard deviations of the 1% wave height and wave period are assumed to be 10% and 20% of the best estimate value, respectively.

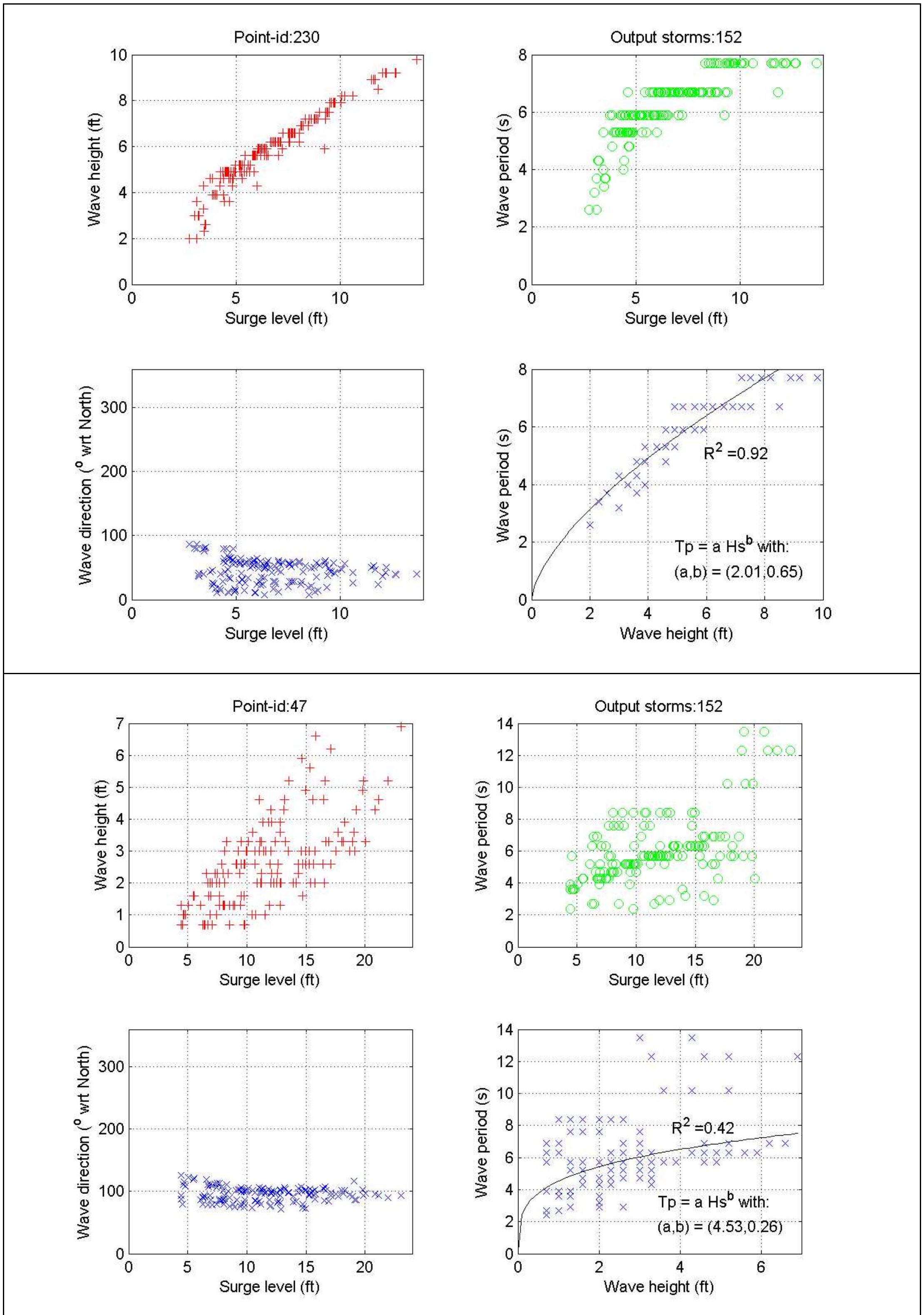


Figure 1.3 – Numerical results at Lake Pontchartrain (upper panel) and MRGO (lower panel) from ADCIRC and STWAVE.

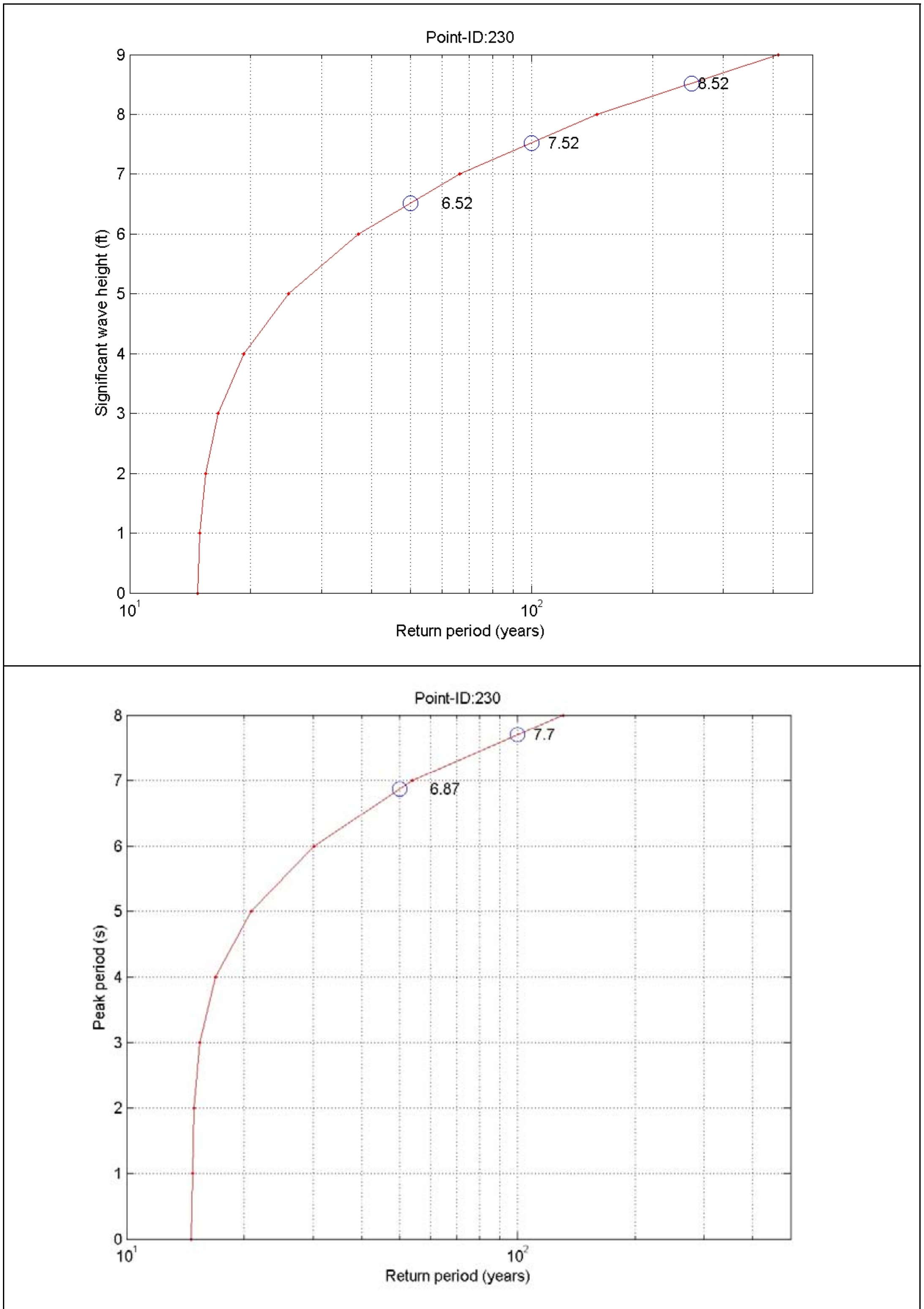


Figure 1.4 – Frequency curves of the wave height and wave period at Lake Pontchartrain (point 230) based on the STWAVE results and the JPM-OS method.

From the JPM-OS frequency analysis, 1% surge elevations, 1% wave heights, and 1% wave characteristics for existing conditions are applied in the wave run-up and overtopping calculations. These values do not consider any future changes due to factors such as subsidence and sea level rise. An additional analysis is performed representing conditions that may occur 50 years in the future and is discussed in Section 0. This future condition (year 2057) does consider changes in the surge levels and wave characteristics due to subsidence and sea level rise.

1.2.4 Wave Overtopping

Several methods are presently available for computing the wave overtopping rates. These methods can be divided into empirical methods (e.g. Van der Meer and Jansen, 1995 and Franco, 1999) and process-based methods (e.g. Lynett, 2002, 2004). Both methods are described briefly below:

> Empirical methods: Several empirical relationships are derived between the offshore hydraulic conditions (wave height, period and water level) and the levee geometry (levee height, slope) and the wave run-up and overtopping rate. These formulations are generally fitted against extensive sets of laboratory data. For levees, there are well-known relationships are formulated by Van der Meer and Jansen (1995) for wave run-up and overtopping. These relationships include the effect of berms, roughness, and wave incidence. These formulations have been incorporated in a software program (PC-Overslag) which is available on the internet at no cost (TAW, 2007)[1]. A second set of formulas developed by Franco&Franco (1999) were used to compute wave overtopping at a vertical wall. The equations were placed in an Excel spreadsheet.
>
> Process-based methods: In a process-based approach the run-up and overtopping rates are computed using the fundamental balance equations for mass and momentum of fluid motion. A Boussinesq model is presently the most appropriate model to compute these parameters within a reasonable time frame. The Boussinesq COULWAVE model from Texas AM was used for this report (e.g. Lynett, 2002, 2004).

Both methods have their advantages and disadvantages. The empirical methods are based on fitted curves through laboratory data, and their use is fairly straightforward. However, the disadvantage of the empirical methods is that these formulations cannot cope with very complex geometries. The basis of Boussinesq models is the governing equations of mass and momentum, and these models are able to handle more complex geometries. A drawback of these models is that they are still in an early stage of development, and the application is time-consuming. In addition, the Boussinesq model does not compute run-up and overtopping at vertical walls. As a design tool, the Boussinesq model lacks the capability to

[1] The reader is referred to the website: http://www.waterkeren.nl/download/pcoverslag.htm

execute in a production mode. Compound levee cross-sections cannot be modified iteratively in a straightforward and timely process.

It is concluded that both approaches give results within a factor of 2 - 3 if overtopping rates of 0.01 – 0.1 cfs/ft are considered. In terms of levee/flood wall heights, the differences in design elevations will be small (< 1ft).

1.2.5 Wave Forces

For floodwalls, pump station fronting protection, tie-in walls, and other vertical "hard" structures, the Goda formulation for computing wave forces is used (see e.g. USACE, 2001; part VI). A definition sketch is shown in Figure 1.5. Hydraulic inputs for these computations are the incoming wave height, wave period and the surge level. Moreover, the geometrical parameters of the structure (bottom elevation, top of wall, etc.) are inputs for this computation.

For submerged structures such as submerged breakwaters, ERDC has developed equations from measurements on a vertical wall in a straight flume physical model. There is the possibility of reflected waves in a confined basin, since his flumes tests did not consider wave amplification due to waves reflected from other vertical surfaces. Although refection would be possible under some conditions, the possibility of wave reflection was unlikely during a hurricane event when the seas were extremely disturbed. The reflected waves would need to be considered if forces during normal conditions are required.

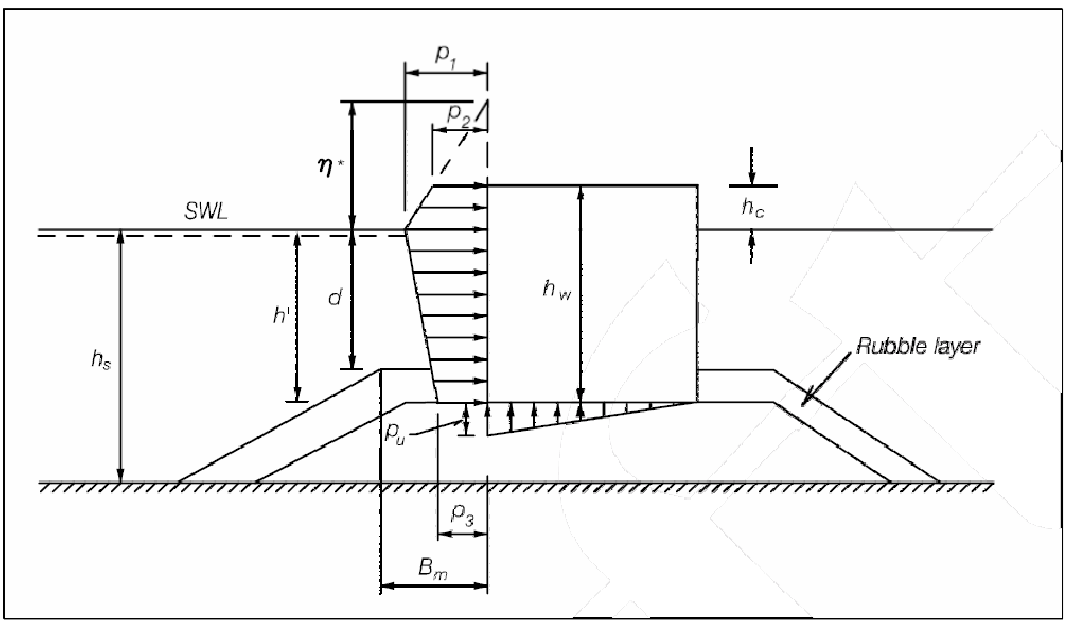


Figure 1.5 – Definition sketch of wave force calculations (source: Coastal Engineering Manual, 2001)

1.3 Step-wise Design Approach

The approach below gives a step-wise approach for determining design elevations and minimum cross sections of levees and design elevations for floodwalls. The step-wise approach is intended to be used for each section that is more or less uniform in terms of hydraulic boundary conditions (water levels, and wave characteristics) and geometry (levee, floodwall, structure). The hurricane protection reaches should be divided into segments with similar hydraulic boundary conditions, based on the JPM-OS frequency results for the water levels and wave characteristics.

Before giving an overview of the step-wise approach, several choices and assumptions in the design approach are discussed in detail. These items are:

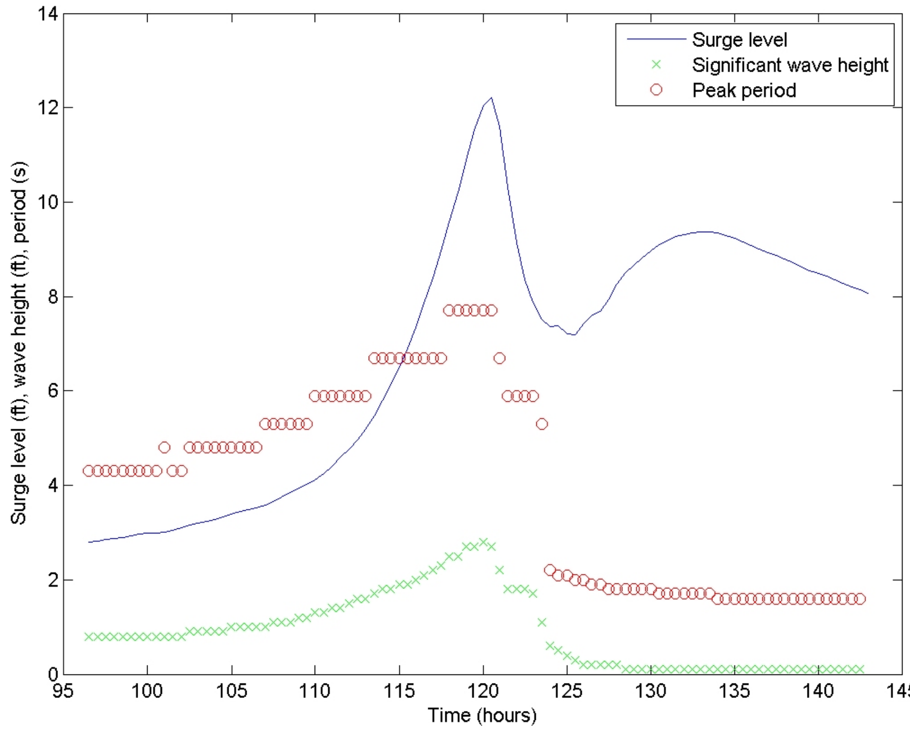
- Use of 1% values for surge levels and waves
- Simultaneous occurrence of maxima
- Breaker parameter
- Overtopping criteria
- Dealing with uncertainties

1.3.1 Use of 1% Values for Surge Elevations and Waves

The step-wise design approach below is probabilistic in the sense that it makes use of the derived 1% water elevations and 1% wave characteristics based on the JPM-OS method (see Resio et al., 2006). The procedure also includes an uncertainty analysis that accounts for uncertainties in the hydraulic parameters and the overtopping coefficients. However, the approach is not fully probabilistic because the correlation between the water elevation and the wave characteristics is not taken into account. This assumption is an important restriction of this approach. Because of this assumption the presented approach is conservative. The impact of this assumption may vary from location to location.

1.3.2 Simultaneous Occurrence of Maxima

Another assumption in the design approach is that the maximum water elevation and the maximum wave height occur simultaneously. Figure 1.6 shows time series of surge elevation and wave characteristics at two locations: Lake Pontchartrain and Lake Borgne. The plots show that the time lag between the peak of the surge elevation and the wave characteristics at both sites is small (< 1 hour). It should be noted that there are cases in which the time lag between surge and waves is a bit larger (say 1 – 2 hours). Although this assumption might be conservative for some locations, we feel that assuming a coincidence of maximum surge and maximum waves is reasonable for most of the levee and floodwall sections in our design approach.



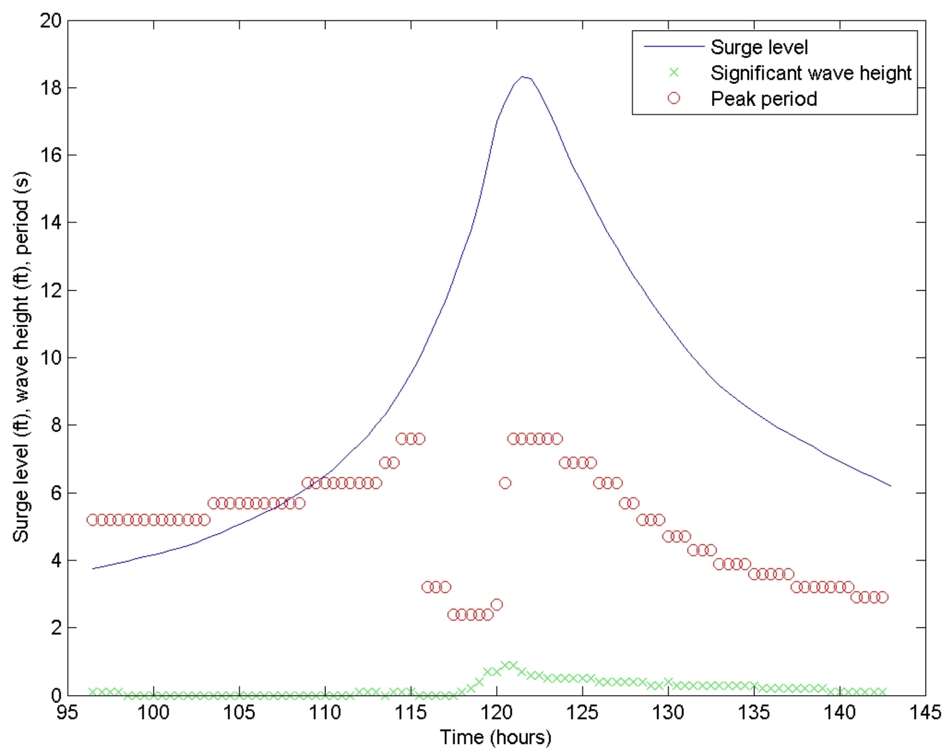


Figure 1.6 – Time histories of surge elevation and wave characteristics during storm 27 at Lake Pontchartrain (upper panel) and at Lake Borgne (lower panel).

1.3.3 Breaker Parameter

In the design approach, overtopping rates are computed using empirical formulations. One input is the wave height at the toe of the structure. This value must be estimated from the wave results from the STWAVE modeling at 600ft before the protection levee or structure. Because the foreshore is generally very shallow (same order as the wave height), wave breaking plays an important role in that 600ft. Hence, it is not likely that the wave height at 600ft in front of the levee or structure will be equal to the wave height at the toe of the levee or structure, but will be lower.

To account for breaking in front of the levee or structure, the wave height from STWAVE is reduced using a breaker parameter. The breaker parameter is the ratio between the significant wave height and the local water depth. In the literature, the breaker parameter is often a constant or it is expressed as a function of bottom slope or incident wave. A typical range for this parameter is between 0.5 – 0.78 in engineering purposes. These values are generally obtained for situations with a mild sloping bed.

Laboratory experiments (Resio, pers. comm.) and Boussinesq runs (Lynett, pers. comm.) suggest that the breaker parameter of 0.4 is a realistic choice for a relatively long shallow foreshore as it is the case for the levees and structures within the project area. Based on recommendations from ERDC, this value has been used in the entire design approach to translate the significant wave heights based on STWAVE model results in the significant wave height at the toe of the levee or structure. The peak period from STWAVE has been used without modification.

1.3.4 Overtopping Criteria

A literature survey has been carried out to underpin the value for the overtopping criterion for levees that must be used in this design approach. The survey shows that various numbers have been proposed. Experimental validation of these numbers is very limited. Typical values according to the Dutch guidelines are (see also TAW, 2002):

- 0.001 cfs/linear ft (cfs/ft) for sandy soil with a poor grass cover;
- 0.01 cfs/ft for clayey soil with a reasonably good grass cover;
- 0.1 cfs/ft for a clay covering and a grass cover according to the requirements for the outer slope or for an armored inner slope.

The literature review suggests that a 0.1 cfs/ft is an appropriate range for maximum allowable overtopping rates based on Dutch and Japanese research.

However, it is difficult to assess the adequacy of applying criteria for the New Orleans area without a good understanding of the overall quality of the levees following many different periods of construction and the effects of stresses of past hurricanes. The actual field evidence supporting these criteria is limited. After consultation with the ASCE External Review Panel, the following wave overtopping rates have been established for the New Orleans District hurricane protection system:

- For the 1% exceedence still water, wave height and wave period, the maximum allowable average wave overtopping of 0.1 cfs/ft at 90% level of assurance and 0.01 cfs/ft at 50% level of assurance for grass-covered levees;
- For the 1% exceedence still water, wave height and wave period, the maximum allowable average wave overtopping of 0.1 cfs/ft at 90% level of assurance and 0.03 cfs/ft at 50% level of assurance for floodwalls with appropriate protection on the back side.

1.3.5 Dealing with Uncertainties

The hydraulic and geometrical parameters in the design approach are uncertain. Hence, the uncertainty in these parameters should be taken into account in the design process to come up with a robust design. This section proposes a method that accounts for uncertainties in water elevations and waves, and computes the overtopping rate with state-of-the-art formulations. The objective of this method is to include the uncertainties check if the overtopping criteria are still met with a certain percentage of assurance.

The parameters that are included in the uncertainty analysis are the 1% water elevation, wave height and wave period. Uncertainties in the geometric parameters are not included; it is assumed that the proposed heights and slopes in this design document are minimum values that will be constructed. To determine the overtopping rate, the probabilistic overtopping formulations from Van der Meer are applied (see textbox below) but also the Boussinesq results could be incorporated in the method. Besides the geometric parameters (levee height and slope), hydraulic input parameters for determination of the overtopping rate in Eq. 1 and 2 are the water elevation (ζ), the significant wave height (H_s) and the peak period (T_p).

In the design process, we use the best estimate 1% values for these parameters from the JPM-OS method (Resio, 2007); uncertainty in these values exists. Resio (2007) has provided a method to derive the standard deviation in the 1% surge elevation. Standard deviation values of 10% of the average significant wave height and 20% of the peak period were used (Smith, pers. comm.). In absence of data, all uncertainties are assumed to normally distributed.

Van der Meer overtopping formulations
The overtopping formulation from Van der Meer reads (see TAW 2002):

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b\xi_0 \exp\left(-4.75\frac{R_c}{H_{m0}}\frac{1}{\xi_0\gamma_b\gamma_f\gamma_\beta\gamma_v}\right)$$

$$with \quad \max imum: \quad \frac{q}{\sqrt{gH_{m0}^3}} = 0.2\exp\left(-2.6\frac{R_c}{H_{m0}}\frac{1}{\gamma_f\gamma_\beta}\right) \qquad (1)$$

With:
q : overtopping rate [cfs/ft]
g : gravitational acceleration [ft/s^2]
H_{m0} : wave height at toe of the structure [ft]
ξ_0: surf similarity parameter [-]
α : slope [-]
R_c : freeboard [ft]
γ : coefficient for presence of berm (b), friction (f), wave incidence (β), vertical wall (v)

The coefficients -4.75 and -2.6 in Eq. 1 are the mean values. The standard deviations of these coefficients are equal to 0.5 and 0.35, respectively and these errors are normally distributed (see TAW document).

Eq. 1 is valid for $\xi_0 < 5$ and slopes steeper than 1:8. For values of $\xi_0 > 7$ the following equation is proposed for the overtopping rate:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 10^{-0.92}\exp\left(-\frac{R_c}{\gamma_f\gamma_\beta H_{m0}(0.33+0.022\xi_0)}\right) \qquad (2)$$

The overtopping rates for the range $5 < \xi_0 < 7$ are obtained by linear interpolation of eq. 1 and 2 using the logarithmic value of the overtopping rates. For slopes between 1:8 and 1:15, the solution should be found by iteration. If the slope is less than 1:15, it should be considered as a berm or a foreshore depending on the length of the section compared to the deep water wave length. The coefficients -0.92 is the mean value. The standard deviation of this coefficient is equal to 0.24 and the error is normally distributed (see TAW 2002).

The Monte Carlo Analysis is executed as follows:

1. Draw a random number between 0 and 1 to set the exceedence probability p.
2. Compute the water elevation from a normal distribution using the mean 1% surge elevation and standard deviation as parameters and with an exceedence probability p.
3. Draw a random number between 0 and 1 to set the exceedence probability p.
4. Compute the wave height and wave period from a normal distribution using the mean 1% wave height/wave period and the associated standard deviation and with an exceedence probability p.
5. Repeat step 3 and 4 for the three overtopping coefficients independently.
6. Compute the overtopping rate for these hydraulic parameters and overtopping coefficients determined in step 2, 4 and 5
7. Repeat the step 1 – 5 a large number of times (N)
8. Compute the 50% and 90% confidence limit of the overtopping rate (i.e. q_{50} and q_{90})

The procedure is implemented in the numerical software package MATLAB.

The Jefferson Lakefront levee section along Lake Pontchartrain has been taken as a reference herein to show one result of this uncertainty analysis. Table 1.1 shows the typical input needed for the Monte Carlo Analysis. It shows the input parameters for the coefficients of the overtopping formulation, the 1% hydraulic design characteristics, and the levee characteristics. Furthermore, the levee characteristics are listed such as the design height and the slope. Several test runs show that N should be +/- 10,000 to reach statistically stationary results for the 50% and 90% confidence limit value of the overtopping rate (Figure 1.7).

Figure 1.8 shows the result of the Monte Carlo analysis; overtopping rate is shown as a function of the exceedence probability. The red lines indicate the 50% and 90% confidence limit value of the overtopping rate for levees. The 50% and 90%-value of the actual overtopping rate for this specific levee section are also depicted in the plot. The result shows that the 90%-value for overtopping is below 0.1 cfs/ft and the 50%-value is below 0.01 cfs/ft, and this section meets the design criteria.

The computation of the overtopping rate in the present MATLAB routine is limited in the sense that it can only take into account an average slope for the entire cross-section. If a wave berm exists, this effect is included in a berm factor. The berm factor is adjusted in a realistic range so that the mean overtopping rate is estimated correctly compared with the result from PC-Overslag.

Table 1.1 -- Input for Monte Carlo Analysis.

Parameter	Mean	Standard Deviation	Unit	Remarks
Coefficient overtopping formula in Eq. 1	-4.75	0.5	-	Mean and standard deviation follow from TAW manual (TAW, 2002)
Coefficient overtopping formula in Eq. 1	-2.6	0.35	-	See above
Coefficient overtopping formula in Eq. 2	-0.92	0.24	-	See above
1% water elevation	9.0	0.6	ft	Values follow from JPM-OS analysis (see Resio, 2007)
1% wave height	3.6	0.4	ft	Mean value from JPM-OS analysis, standard deviation 10% of mean value based on expert judgment
1% wave period	7.7	1.54	s	Mean value from JPM-OS analysis, standard deviation 20% of mean value based on expert judgment
Levee height	16.5	-	ft	
Slope	1V:4H	-	-	
Berm factor	0.6	-	-	
Number of runs	10,000	-	-	

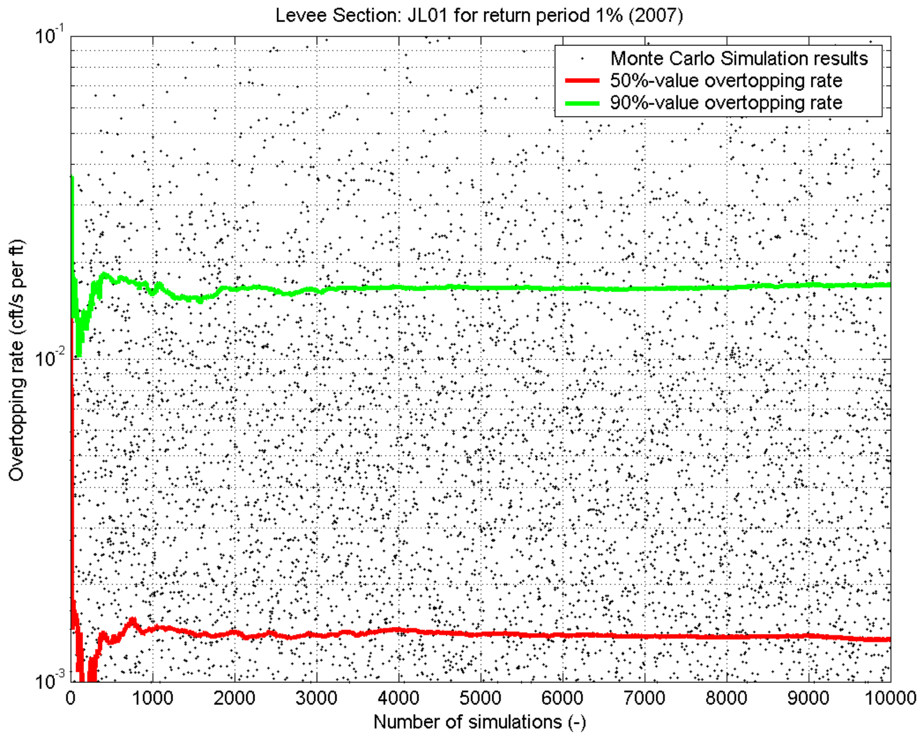


Figure 1.7 – The 50% and 90% confidence limit value of the overtopping rate as a function of the number of simulations during the Monte Carlo Analysis. The dots represent the actual results from the Monte Carlo Simulation, whereas the red and green lines represent the moving value over the number of simulations.

Notice that the uncertainty analysis described above is also implemented to compute the wave forces with different confidence levels. It makes use of exactly the same procedure, but computes the wave forces based on the Goda formulation. A Monte Carlo Simulation is performed with the water level, wave height and wave period, and the associated uncertainty, to compute the 50% and 90% assurance wave forces. Dependency between the errors in the wave height and wave period is maintained, whereas the error in the surge level and the wave characteristics are to be treated independently.

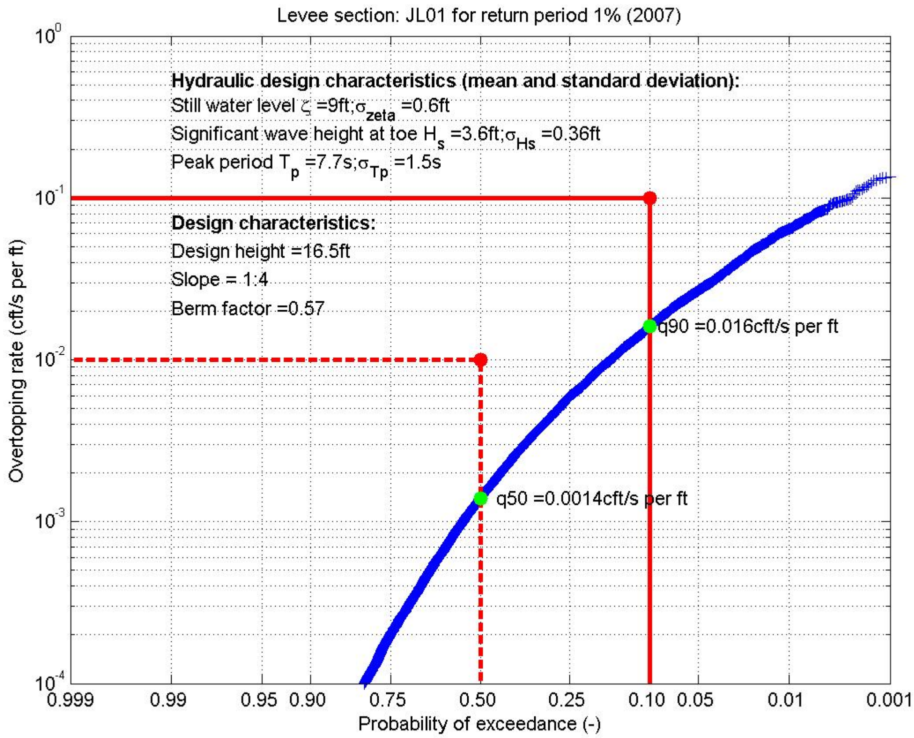


Figure 1.8 – Result of Monte Carlo Analysis for Jefferson Lakefront levee (existing conditions).

1.3.6 Step-Wise Approach

The proposed step-wise approach for design is as follows:

<u>Step 1: Water elevation</u>

1.1 Examine the 1% surge elevation from the surge frequency plots at all output points along the reach under consideration. The 1% surge elevations are the results based on the 152 storm combinations and using the probabilistic tool (JPM-OS method).

1.2 Determine the maximum 1% surge elevation for a design reach and use this number for the entire reach. The maximum is chosen to meet the design criterion at the most critical point in the section.

<u>Step 2: Wave characteristics</u>

2.1 Examine the 1% significant wave height and peak period from the frequency plots at all output points along the reach. The 1% wave heights and peak periods are the results based on the 152 storm combinations and using the probabilistic tool based on the JPM-OS method.

2.2 Determine the maximum 1% significant wave height and peak period for the reach and use these numbers for the entire reach. The maximum wave height and wave period are chosen to meet the design criterion at the most critical point in the section under consideration.

2.3 Determine if the foreshore in front of the structure is shallow. The foreshore is shallow if the ratio between the significant wave height (H_s) and the water depth (h) is small ($H_s/h > 1/3$) and if the foreshore length (L) is longer than one deep water wave length L0 (thus: $L > L_o$ with $L_o = gT_p^2/(2\pi)$). If so, the wave height at the toe of the structure should be reduced according to $H_{smax} = 0.4$ h. This reduction should only be applied if an empirical method is applied for determining the overtopping rate (e.g. PC-Overslag). The breaking effect is automatically included in the Boussinesq runs.

<u>Step 3: Overtopping rate</u>

3.1 Apply PC-Overslag with Van der Meer formulations (see also CEM) to determine the overtopping rates. If a wall is present, the empirical formulation of Franco&Franco (1999) will be applied. For specific complicated cross-sections, the Boussinesq lookup tables may be applied as well to compute the overtopping rate.

3.2 Determine the overtopping rate based on the 1% (average) values for the surge elevation, the significant wave height and the peak period. Use the reduced wave height in case of a shallow foreshore in the empirical approach only (e.g. PC-Overslag).

<u>Step 4: Dealing with uncertainties</u>

4.1 Apply a Monte Carlo Simulation to compute the chance of exceedence of the overtopping rate given the design elevation and slope from step 3. This method takes into account the uncertainties in the 1% water elevation, the 1% wave height and the 1% wave period. The approach is explained in detail in the next section.

4.2 Check if the overtopping rate will not exceed the design thresholds for overtopping. If yes, the design process is finished from a hydraulic point of view. If not adapt the levee or floodwall height or slope in such a way that this criterion is reached.

<u>Step 5: Resiliency</u>

For the design analysis, the overtopping rate for the 0.2% exceedence event is evaluated and both the 50% and 90% confidence limits of the overtopping rates are computed given the 1% designs. This information will be used in the entire design process to evaluate the resilience and check if armoring or other measures are necessary. This approach is still under review, and no final decisions have been made as to the use of the 0.2% event information.

1.4 Design Conditions

Two design conditions are considered in this report: existing conditions and future conditions. Both conditions are discussed below.

1.4.1 Existing Conditions

Design elevations for this scenario are considered to reflect conditions that are likely to exist in the year 2007 or year 2010. It is assumed that all levee and floodwall repairs have been made, and the interim or permanent closures and pumping stations at 17th St., Orleans Avenue and London Avenue outfall Canals are in place. The gates on the MRGO/GIWW are in place.

For most of the analysis, the existing surge elevations are based on the ADCIRC results of the 152 storm conditions for the 2007 case in conjunction with the JPM-OS method. The existing wave conditions are derived based on the STWAVE results, and are derived in a similar way. Model results from the 2010 condition were used for the analysis of the area that is affected by the MRGO/GIWW gate.

1.4.2 Future Conditions

Design elevations for this scenario are considered to reflect conditions that are likely to exist in the year 2057. Changes in surge elevations will occur in the future due to subsidence and sea level rise. Historical subsidence, projections of sea level rise, and previous studies were used to estimate future changes in surge elevations. Natural subsidence rates, including sea level rise, have been mapped by MVN for the LCA effort. Figure 1.9 shows the combined natural subsidence/eustatic sea level rise for the hurricane protection project area. The values presented in Figure 1.9 are geologic rates and do not consider any factors such as pumped drainage, which can influence regional subsidence. A relative sea level rise of 1ft over 50 years was used in the design analysis to represent future conditions in the entire area.

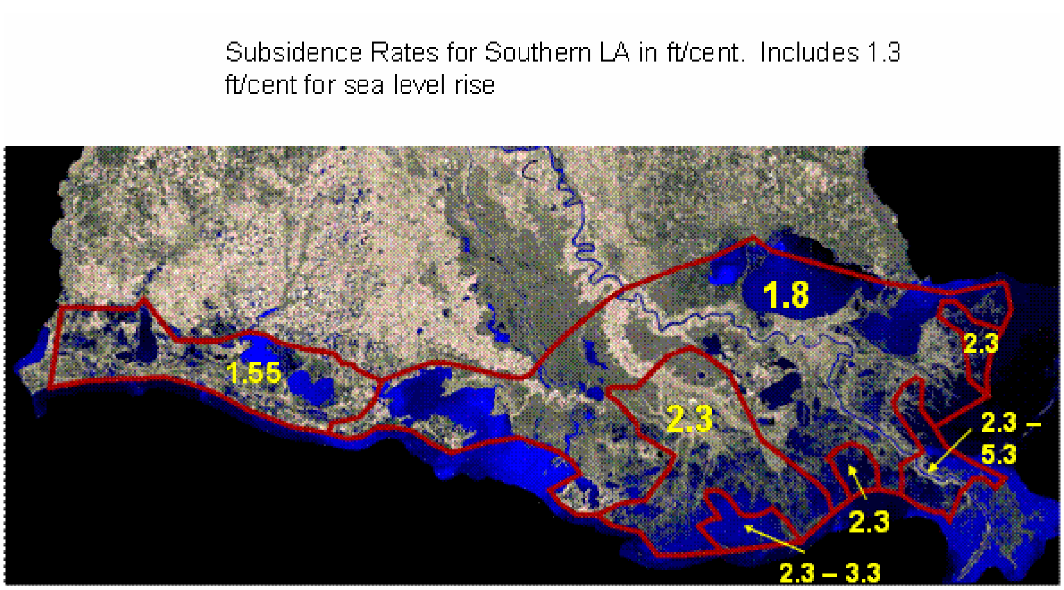


Figure 1.9 Estimated relative sea level rise during 100 year (subsidence + sea level rise)

Several ADCIRC and STWAVE model runs were performed to investigate the effect of the increasing sea level rise on surge levels and wave characteristics. These results show that:

- The surge levels increase more than proportional to increasing sea level rise (factor 1.5 to 2). A factor 1.5 implies that 1 ft sea level rise results in 1.5 ft increase of the surge level etc.
- The wave heights increase due to sea level rise. The relative effect on the wave heights is about 0.3 to 0.6 which means that 1 ft surge level results in 0.3 to 0.6 ft increment of wave height.
- The effects are not uniform in the entire area but depend on the local water depth, and geometry of the area of interest.

Based on these, the future conditions are summarized below (Table 1.2):

Table 1.2 - Future conditions for surge level and wave characteristics

Future conditions	**Surge level h_{surge}**		**Significant wave height H_s**		**Peak period T_p**
	Δh_{surge}/ $\Delta h_{sealevel}$ (-)	**Δh_{surge} (ft)**	**ΔH/ Δh_{surge} (-)**	**ΔH (ft)**	**ΔT_p (s)**
Lake Pontchartrain, New Orleans East, IHNC and GIWW, St Bernard	1.5	+1.5ft	0.5	+0.75ft	Increase by assuming unchanged wave steepness (H/T^2)
Caernarvon, West Bank	2.0	+2ft	0.5	+1ft	Increase by unchanged wave steepness (H/T^2)

Because the future condition surge elevations are derived from the surge elevations for existing conditions, uncertainty in the data and methodologies has been included. No additional value was added to address uncertainty in the increment representing subsidence, land loss, and sea level rise. The future condition surge elevation was used in wave computations, wave loads on walls and other "hard" structures, and to determine design elevations.

1.5 Design Elevations and Loads

In the design analysis, two types of flood protection are considered: soft structures (levees) and hard structures (floodwalls and other structures like pumping stations).

Levees. The design elevations are computed for both the present and the future conditions. The design elevations presented in this report only consider (relative) sea level rise for future conditions, but do not consider settlement or other structural adjustments. The design elevation recommended for levee construction at this time is the existing elevation. The levees are expected to be adapted several times during its lifetime due to settlement and changes in the hydraulic conditions should be taken into account as well.

Floodwalls and Other Structures. The recommended design elevation for floodwalls and other "hard" structures is the future conditions elevation. The recommended design elevation for floodwalls and other "hard" structures should be no less than the future condition design elevation of adjacent levees. Floodwalls and other "hard" structures will require extensive reconstruction in the future; incorporating future changes into the design of these structures now is a prudent design consideration.

The design elevations of floodwalls sometimes do include structural superiority. Structural superiority is incorporated in the design elevation for those structures that would be very difficult to rebuild, if damaged, because of disruption in services. Examples are major highway and railroad gates that require detours, pumping station fronting protection that requires reductions to pumping capacity, sector gated structures, etc. These structures are to be constructed to the 2057 levels plus 2 ft. for structural superiority. Floodwalls that can be rebuilt in areas with little or no disruption of services are to be constructed to the 2057 level.

The wave forces have been computed for the floodwalls and submerged breakwaters. These forces are evaluated for future conditions (2057). Wave forces are evaluated for two confidence levels (50% and 90%) to present the uncertainty in these numbers. At this moment, there has not been made a final decision at MVN which of these results will be used in the structural design.

1.6 Armoring

1.6.1 Introduction

Damage sustained to the levee system during Hurricane Katrina occurred primarily: (1) at transitions between earthen levees and vertical floodwall structures, (2) on the protected-side slopes of earthen levees, and (3) near the protected side base of vertical floodwalls. In May 2006, US Army Engineer Research and Development Center (ERDC), Vicksburg, MS completed an evaluation of armoring for the US Army Engineer District, New Orleans (MVN) and for Task Force Guardian (TFG). The purpose of this evaluation was to overview levee and floodwall failure modes, characterize the hydrodynamic forces that protection systems must withstand, establish initial performance criteria for protection systems, and provide an initial assessment of available armoring and protection systems.

There are four major topics relating to armoring for which guidance is required – protected side fortification of levees to minimize the effects of overtopping, frontside protection of levees from wave attack, protected side protection of walls and levee/wall transition areas, and the use of engineering solutions such as breakwaters and soil modification to modify or reduce overtopping effects.

Scour protection details and guidance used for TFG have been included in the Structrural section of this document; it is included as reference only. Proper engineering must be accomplished to ensure the best solution. There are many factors that must be considered, such as scour materials, overtopping hydraulics, and the effects of water that has overtopped on interior drainage and infrastructure.

Different materials are available for armoring. They include: Riprap; Gabions or other wire baskets filled with stone; Rock-filled wire or geogrid mattresses; Articulated concrete mattresses of interlocking blocks or blocks connected by cables; Cast-in-place, concrete-filled geosynthetic mattresses or tubes; Soil stabilizing devices designed to retain the soil within the structure such as geocells; Mattresses designed to hold vegetation in place such as “Turf Reinforcement Mats” (TRMs); and paving with asphalt or concrete. Soil reinforcement and the use of best construction materials and techniques may improve the levee’s ability to withstand erosion.

1.6.2 Levee Armoring

Two essential items are needed in order to design armoring. First, it is essential to know the anticipated extreme loading for which armoring is required, and, second, it is essential to know the limits of applicability of various armoring protection systems and the upper limits of the extreme loading for which protection is desired. When both of these are known, the engineer will select the appropriate armoring that has a resistance equal or greater than the anticipated extreme loading.

The current design philosophy entails limiting the overtopping of protections that occur in the 1% event to a quantity that can be carried by typical turf covering. The more critical design condition is to provide armoring for overtopping of protections that occur in the 0.2% event. The hydraulic engineer will provide the design overtopping rates for this event. It is important to note that overflow of the system, i.e., free flow at the still water level, is not allowed for the 1% or 0.2% events. Armoring will be designed to protect from wave and over splash only.

The use of existing guidelines for stone as an armoring material clearly demonstrates the problem of lack of testing and lack of guidance on hydraulic issues related to overtopping; one such problem is the thickness of the stone vs the depth of wave runup or overtopping. For stone to withstand the magnitude of the velocities experienced during Hurricane Katrina computed by IPET on the MRGO levee, the thickness calculated using traditional methods contained in EM 1110-2-1601 is considerably larger than the depth of water. Will the overtopping continue to flow on top of the rock or be absorbed within the rock thickness? How are the velocities altered?

Revetment is presently being tested at ERDC as a potential armoring material along the MRGO levee. Anchoring the revetment is a critical issue. ERDC tests show the possibility of the revetment at the toe of the floodside slope to roll up; at the toe of the backside slope, the revetment was lifted each time a wave of water reached it.

In addition to armoring protection for all forms of overtopping, armoring protection may be needed for wave attack. Overtopping protection is for the crest and the back, or protected, side of the levee, and wave protection is for the floodside of the levee. The floodside protection for wave attack is much better documented than is the protection for overtopping. Armor stone size and riprap gradations can be obtained from the interactive version of the Coastal Engineering Manual.

ERDC found that few (if any) armoring or slope protection products have been tested at large scale for effectiveness when subjected to wave overtopping. The periodic nature of wave overtopping makes a difference between wave overtopping and steady flow overtopping. As each wave overtops, it has a forward velocity across the levee crest that likely exceeds the crest velocity of surge overtopping. Thus, unprotected soil on the levee crest that is stable for surge overtopping may erode if waves overtop. However, this flow condition is unsteady and peak velocities are sustained for only a brief time. In addition, the unsteady discharge over the crest results in a limited overtopping volume. Consequently, any erosion on the backside slope due to wave overtopping is intermittent, and probably does not progress at rates as high as what can occur for steady surge overtopping.

Without a doubt, turf is the most economical revetment material in terms of installation and maintenance. However, there are situations where turf is not strong enough to resist the erosive forces due to design conditions. The more preferable alternative is to use turf reinforcement since it has distinct advantages in terms of cost, weight, ease of installation and maintenance over other systems of armoring. When the potential erosion forces are deemed to be greater than the resistance capacity of reinforced turf, other systems such as rip-rap, articulated mats, interlocking blocks, gabions, concrete paving, etc. will be required.

However, before designing armoring for wave attack it is important to recognize how well the turf on the New Orleans Lakefront levees (LPV project) withstood wave attack. Waves of 2.5 to 3 meters were measured on the south shore of the lake in the vicinity of the new Coast Guard station just west of the 17th Street Canal. To the east, the levee is protected by the Orleans seawall but to the west in Jefferson Parish there is little protection for the levee. Along the entire Lakefront levee, there was no reported wave erosion.

The Dutch have published a technical report on the erosion resistance of grass as levee (dike) covering (TAW, 1997). In the Netherlands, waves against the outer banks of sea and lake dikes can reach heights of more than 1.5 meters. The Dutch found that very good grass mats, on a bank of slope 1:3 to 1:4 and on erosion-resistant undersoil, can withstand waves up to 1.0 meters with no serious damage after more than one day. The damage free period for waves of slightly more than 1.0 meters was shorter, but still long enough to cope with the Dutch storm flood. The underlayer was found to be important; it should always consist of adequate erosion-resistant clay, which must be at least 1 to 1.5 meters thick. Grass mats above the still water level were found to resist waves higher than grass mats in the wave breaking zone.

1.6.2.1 Turf Design

Both the Dutch and the Danes have done extensive testing of existing turf on dikes. The resistance to erosion increases with the density of root mass. The critical parameter is the dry root mass per unit area. They have also determined the best practices to increase the root mass of the turf. All of the mechanisms that are expounded by the Dutch and the Danes appear counter-intuitive at first but upon reflection make perfect sense. For example, non-fertilized turf has better erosion resistance than fertilized turf. This is because the amount of roots is the most important factor. Fertilization will produce lush greenery, but the greenery does not contribute to erosion resistance. It merely shears off in any high energy environment. Fertilization allows the roots to uptake lots of nutrients without having to extend the root mass in search of nutrients. For the same reason soils with low nutrient content produce better erosion resistant turf, since the roots have to grow and search for nutrients. A large variety of species will produce a better turf since there will be competition among the plants. The Danes categorize a turf in terms of the number of species per 25 square meters. A good dike turf will have over 20 species per 25 square meters.

Land use will influence the quality of the turf. Grazing of livestock (equivalent to our frequent mowing) does not produce the same root mass as haying. Allowing the grass to grow tall before cutting encourages deeper roots to support the taller grass. Of course the grass should be removed (as is done in making hay) for two reasons, one so that the cut grass does not suffocate the grass plants and two so that the cut grass does not compost and produce nutrients in the upper layer and thus impeding root growth.

The geotechnical lab at ERDC produced a scope of work and a cost estimate to investigate the strength of the turf on the hurricane levees in the New Orleans District. The scope included parameterizing the depth and density of the roots for various levee turfs. When this investigation gets funded, it will help District engineers to understand the limits of turf protection. This investigation will also have help to answer questions MVN-ED-H engineers have about the testing of reinforced turf mats at the Colorado State steep gradient flume facility.

In the past very little attention has been given to the production of quality turf. It is essential that the Corps begin to look at turf as the important revetment material that it is and start to implement a program along with the local sponsors to produce the best quality turf and turf management practices.

1.6.2.2 Turf Reinforcement

Turf reinforcement has four distinct advantages over any other system of levee armoring. Foremost, the turf reinforcement does not contribute any significant weight that will induce settlement or stability issues. The cost is much less than rock, or any other heavy material. Turf reinforcement can be more quickly installed than any other system. Turf reinforcement is easily maintained, it just needs to be mowed the same as turf. Riprap and gabions will eventually have trees and shrubs growing in them and properly removing them is a serious negative consideration.

For the reasons listed above turf reinforcement mats (TRM) should be given serious consideration in the effort to armor the hurricane protection levees. The only question is to determine the limits of the applicability of TRM protection. Only vigorous research can provide this much needed answer.

1.6.3 Walls and Levee Transitions

Floodwalls that may be overtopped by rising water should be designed with erosion protection on the protected side capable of resisting the force of the free-falling water jet. Equations are available to compute the location where the free-falling water jets hits the ground on the backside of the wall. This location is dependent on the height of the wall and the surge height above the wall. ERDC found that these equations may under estimate the distance. The protection coverage must extend away from the wall beyond this location to account for the hydraulic jump that will form when the flow changes from supercritical to subcritical as well as uncertainty in the computation. Where overtopping is from waves only, the unsteady discharge will be a function of wave height, wave period, and surge elevation relative to the wall. Erosion of unprotected soil will occur as the waves cascade over the wall, but the unsteadiness of the process, coupled with the variation of impact point due to irregular waves, makes scour estimation difficult, if not impossible.

For transition areas, as indicated in the ERDC report, simple analytical methods for estimating the increased flow velocities that occur at transitions are lacking, and most likely either physical modeling or sophisticated numerical simulations will be required to establish flow velocities due to surge overtopping in the vicinity of levee/floodwall transitions. However, some insight into the overtopping problem can be gleaned by looking at results obtained from two-dimensional inviscid jet theory. Based on discharge contours, the flow velocity along the outer edge of the jet is about 1.64 times the flow velocity through the middle of the gap. Therefore, it is easy to see that the region immediately adjacent to the vertical wall experiences the largest flow velocity. The addition of waves propagating on top of the overtopping surge compounds the complexity of the flow situation, and no simple procedures are available to address this case. Laboratory testing will be the best tool for examining the stability of armoring alternatives subjected to water and wave overtopping at levee transitions.

1.6.4 On-going Studies

ERDC Coastal and Hydraulics Laboratory has completed field study of the effects of the 2005 hurricanes on the hurricane and storm damage reduction system. Their findings are summarized in the report, "Protection Alternatives for Levees and Floodwalls in Southeast Louisiana: Phase One Evaluation." Although the document is still a draft, Chapter 4, "Protection for Overtopped Floodwalls," is included as an appendix to these guidelines for information only.

Phase Two of the study, which is to provide physical modeling and recommendations for design of overtopping and scour protection, has not been completed. That information will be incorporated into these guidelines as soon as it is available.

Task Force Hope has commissioned an Armoring Team to provide guidance on the use of existing technologies for armoring and to more rigorously investigate armoring design and methods for future use. Engineering Division Hydraulics Branch has also chartered a team to investigate ways to provide resiliency for levees and walls that are overtopped by events exceeding design conditions. This effort includes plans to perform a field test of a levee subjected to overtopping forces. Input from these two teams will guide future design work and design guidance.

2.0 RELOCATIONS

2.1 Facility Relocations

There are numerous facilities within the limits of work that will/may be affected by the proposed work and will/may require relocation. Relocation documents, which include ROW drawings with identified existing facilities, completed questionnaire forms, and as-built drawings, are required at 35% Review. These documents are for the relocating of all roads, railroads or utilities, or any feature impacted by the proposed work.

Existing facilities impacted shall be identified on plan and profile and rights-of-way drawings. Designers shall verify all relocation items within the ROW and identify any additional relocation items that lie within the ROW and are not shown on these drawings. Relocation plans shall include overhead and underground electrical lines, overhead and underground cable and telephone lines, underground pipelines, sewer lines, and any other utilities as well as any roads or rail lines that will be affected by the proposed work.

The Corps of Engineers drawing H-8-29027 “Pipeline Crossing Over Levees and Floodwalls” provides approved design guidelines for a variety of pipeline crossing situations. This drawing is included in Section 12.

In addition, if the Construction Contractor must cross any buried pipeline during construction, coordination with the owner shall be required to determine if pipeline protection is required. If an owner determines that protection is required, designers shall obtain a pipeline protection plan from the owner. Designers shall meet with the facility owners and provide them with the Government-furnished Utility Relocations Questionnaires to gather all information regarding the types of facilities crossing the proposed flood protection. Designers shall request any as-built drawings for and shall also discuss the proposed relocations of the facilities crossing the flood protection within the project limits. This meeting shall take place during the preparation of the right-of-way maps.

If this work is provided by an A-E, the A-E may include a Government and local representative at these meetings. However, the A-E shall keep the Government informed of all coordination meetings held with the facility owners. The Government alone shall determine compensability. Therefore, the A-E shall have no discussions with facility owners regarding the compensability of affected facilities.

Designers shall coordinate facility relocations and schedules as required with the facility owner. Plans for concurrent relocations, if necessary, shall be developed in conjunction with the facility owner. The proposed facility relocations shall be acceptable to the owners and the levee district and shall comply with the Government’s standards for crossings of levees and floodwalls.

If this work is provided by an A-E, each facility relocation plan must be submitted to the Government for approval. Upon approval of the plan by the Government, the A-E shall instruct the facility owner to apply to the levee district for a permit to accomplish the relocation.

No facility can be considered abandoned unless the designer acquires a letter of abandonment from the owner.

Designers shall present the relocation information on the ROW drawings, the contract plans and in the contract specifications. A-Es shall provide copies of all completed Utility Relocations Questionnaires and as-built drawings obtained from the facility owners to the Government. Designers shall also list points of contact for all relocation items in the contract specifications. The list shall include all pipelines for which the owner will provide pipeline protection. Each facility shall be tabulated on both the ROW drawings and the contract plans with the following information: Baseline Station, Owner, Description, and one of the following dispositions:

(1) DO NOT DISTURB
(2) DO NOT DISTURB-ACCESS ONLY
(3) TO BE RELOCATED BY OWNER PRIOR TO CONSTRUCTION
(4) TO BE RELOCATED BY OWNER CONCURRENT
WITH CONSTRUCTION
(5) TO BE RELOCATED BY CONTRACTOR DURING CONSTRUCTION.

2.2 Deliverables And Project Schedule

If an A-E provides these services, deliverables shall include:

(1) As-Built Drawings. The A-E shall provide the Government copies of the owners' as-built drawings.

(2) Relocation Plans. The A-E shall provide the Government copies of the owner's engineering drawings that detail the relocation plan and protection plan where necessary for approval.

(3) Relocations Correspondence. The A-E shall provide the Government a copy of all correspondence with facility owners including the questionnaires completed by the owners and letters in which owners declare facilities abandoned.

2.3 Utility Relocations Questionnaires

Sample questionnaires to be used to collect information from owners of affected facilities are included in Part B of this document.

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.

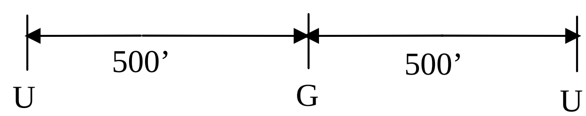


Figure 3.1 Boring spacing

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.

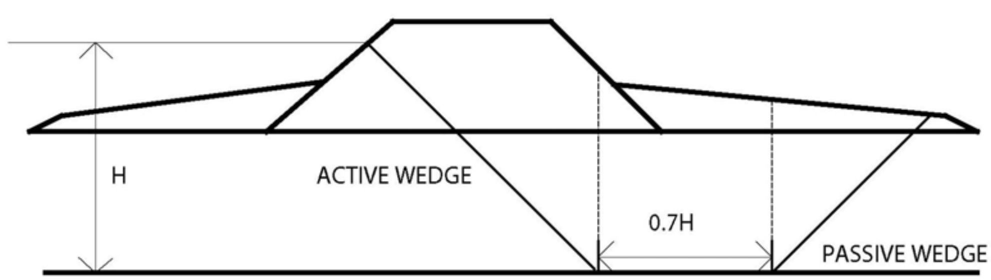


Figure 3.2 Minimum distance between active and passive wedges (embankments)

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.

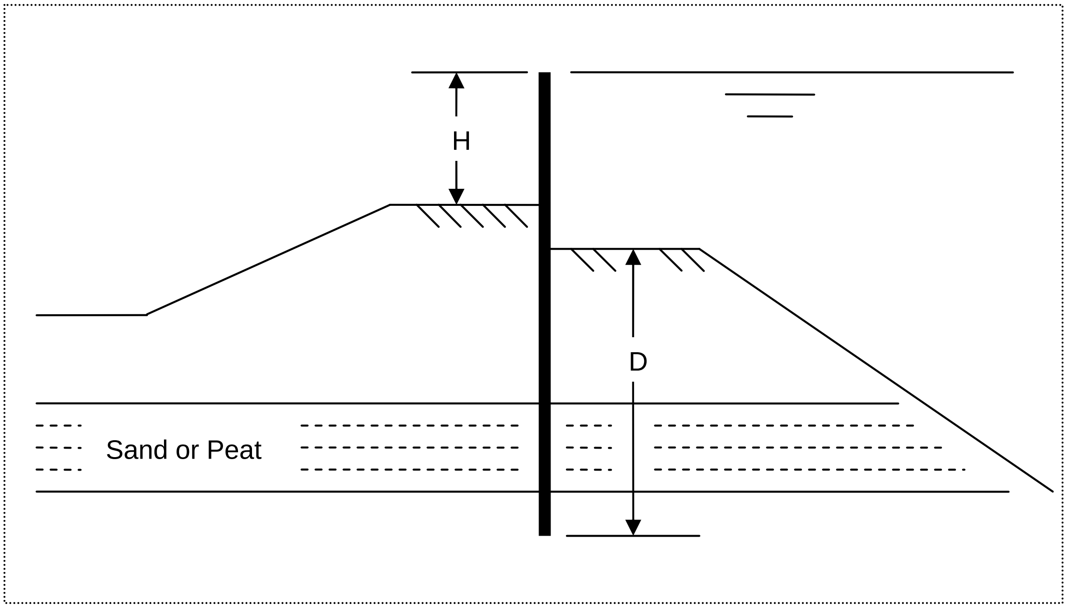


Figure 3.3 Minimum tip penetration depth

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.

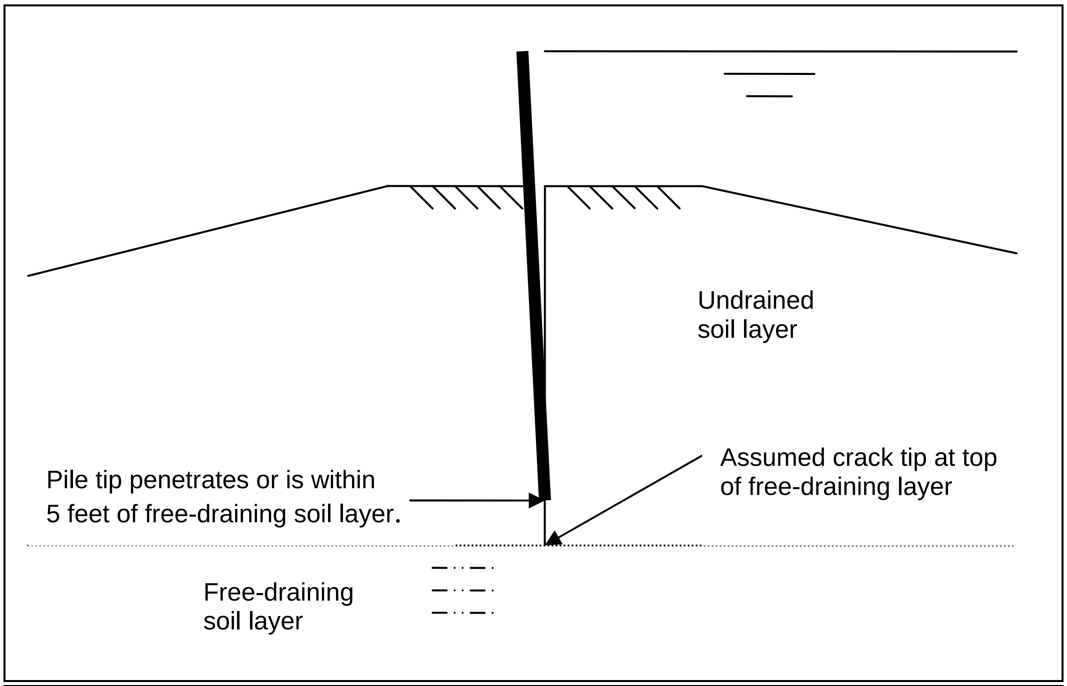


Figure 3.4 Computed crack depth near an aquifer

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

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

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

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

Stability Analysis Results:

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

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

3.4.2.2 T-Wall Sheet Piling Cut-off Tip Penetration

Sheet pile tip elevations shall meet criteria for seepage control and at a minimum, shall extend 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

Piping (cutoff-wall tip elevation). T-walls and L-Walls must be designed for piping erosion along the base of the pile founded wall. Analysis shall be based on water to the top of the wall. This analysis can be performed by various methods such as Lane's weighted creep ratio, flow nets, Harr's method of fragments, or finite element methods. A design procedure used for evaluating piping erosion for clays, silts, and sands directly beneath pile-founded L-walls and T-walls for hurricane protection is to use Lane's weighted creep ratio for a seepage path along the sheet pile wall. Engineering judgment should be exercised in selecting appropriate weighted creep ratio values for this analysis and using the weighted creep length based on flow path through the different foundation materials.

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

3.4.2.6 Heave Analysis

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

3.4.3 T-Wall Design Procedure

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.

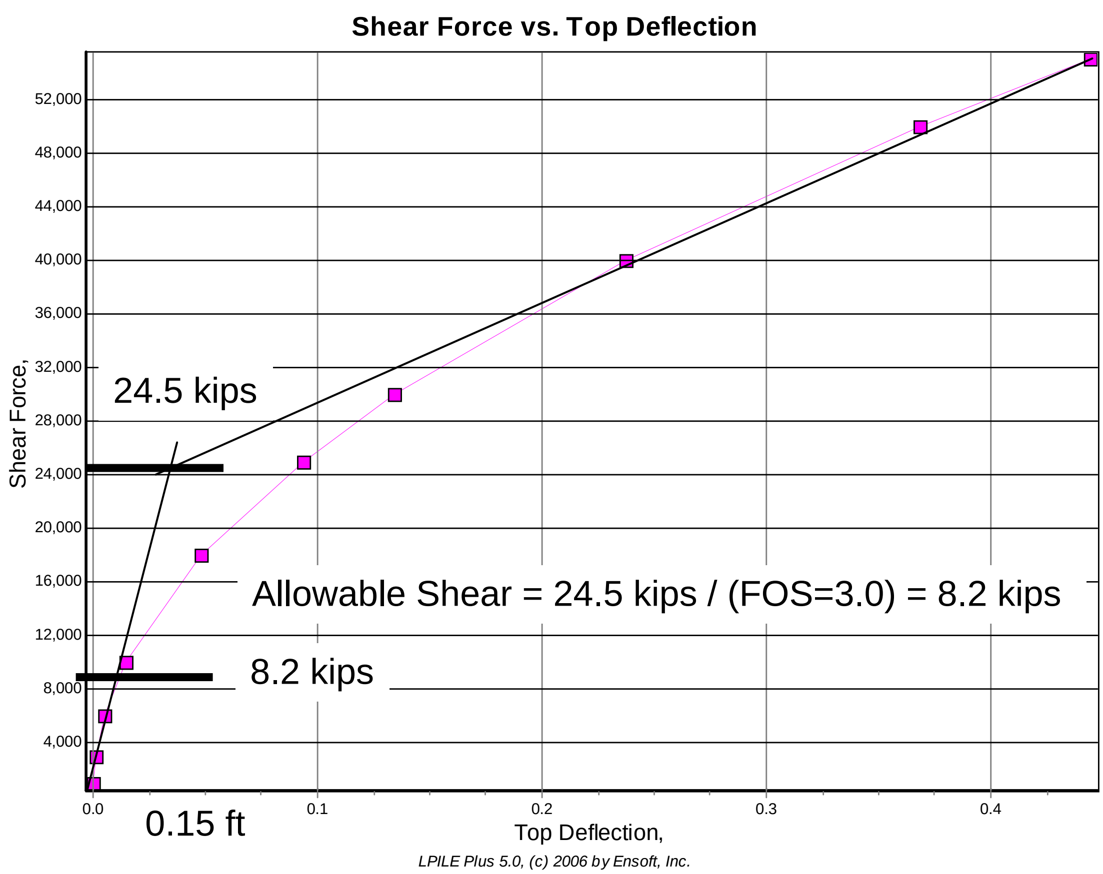


Figure 3.5 Example of computation of ultimate shear load in the pile from a load vs. deflection curve developed using LPILE. FOS varies depending on load case.

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:

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

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)

$$R = \sqrt[4]{\frac{EI}{Es}} \qquad (2)$$

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.

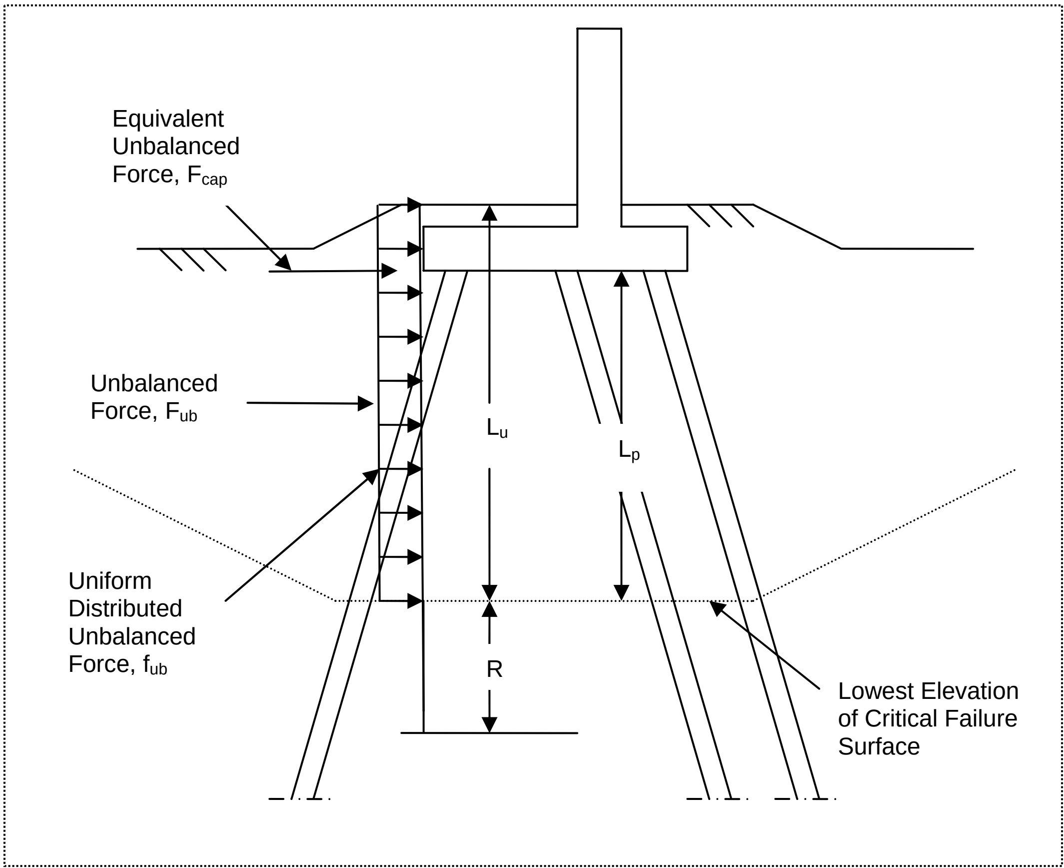


Figure 3.6 Unbalanced Forces.

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:

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

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

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

S_u = soil shear strength

b = pile width

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

For leading (flood side) piles:

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

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

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

Where:

s = spacing *between* piles parallel to loading

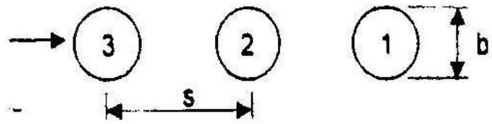


Figure 3.7 Spacing between piles

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}:

$$F_p = w f_{ub} L_p \tag{7}$$

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:

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

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

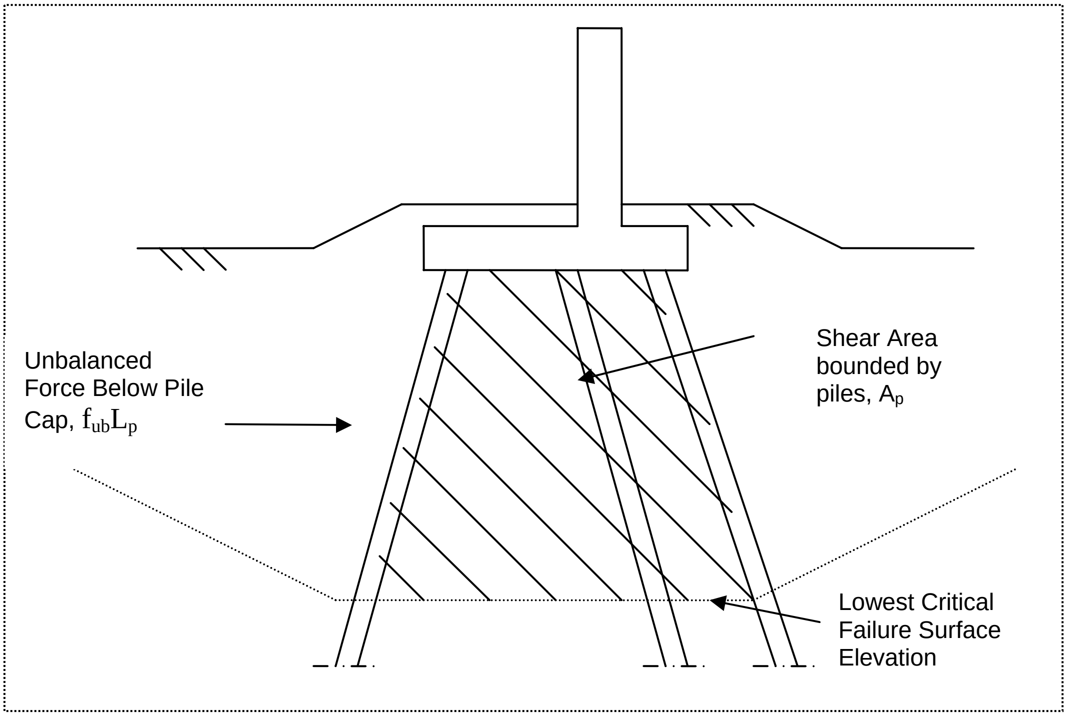


Figure 3.8 Area for soil flow-through shear check.

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.

<u>Step 5. Pile Group Analysis (all loads)</u>

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.)

$$P_f = \sigma_3 + s_u \tag{1}$$

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).

$$FOS = (\sigma_3 + s_u)/P_m \tag{2}$$

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.

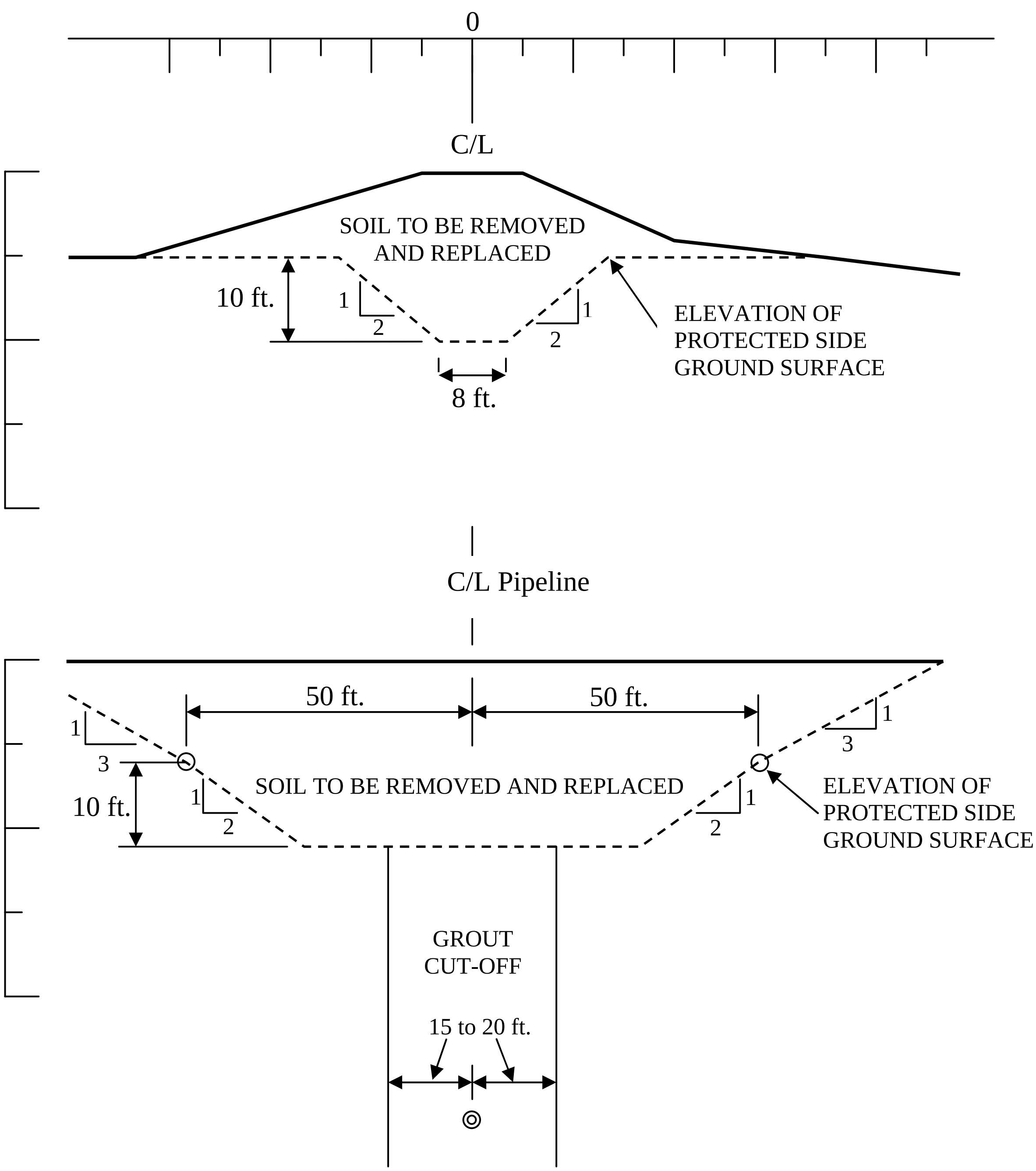


Figure 3.9 Sample detail of repair of directional drilling damage to levee

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.

4.0 LEVEES

4.1 Sampling of References

- EM 1110-2-1913, Design and Construction of Levees, 30 Apr 00
- LADOTD Standard Specifications for Roads and Bridges, Louisiana Department of Transportation and Development

In addition, Section 12 of this document includes typical details applicable to levee design and construction.

4.2 Preliminary Work

4.2.1 Develop Project Delivery Schedule

Develop project delivery schedule in advance of anticipated start date. A standard timeline based on complexity of project shall be utilized as a basis for developing the schedule. Accelerated design contracts shall be adjusted accordingly to meet project deadlines for project delivery completion dates. The Product Delivery Team (PDT) members shall be consulted to provide time frames for incorporation into the schedule.

4.2.2 Initial Project Site Visit

Visit the site of work with the PDT. The site visit is commenced after becoming familiar with the area through the office study. Walking the proposed project and potential borrow areas shall be performed to gather physical information. Physical features to be observed are inventoried by detailed notes, supplemented by photographs. Local persons, the local sponsor and/or organizations having knowledge of existing conditions and facilities in the area should be interviewed to gather information concerning subsurface utilities, historical problematic conditions, etc. A site inspection report shall be prepared for permanent files summarizing the findings with prints of significant photos.

4.2.3 Preliminary Requests to PDT

Request right of entry (ROE) for surveys, borings, HTRW, cultural resource and environmental investigations encompassing the entire project area and potential borrow areas as determined during the initial project site visit.

4.3 Project Delivery Work

4.3.1 Request for Initial Engineering Input from PDT

Request initial utility locations/ownership determination, field surveys, design borings, hydraulics and initiation of preliminary HTRW, cultural resource and environmental investigations. A sample survey request form for internal MVN use is shown in Figure 4.1. Note in 4.3.2 below that further information is to be provided to the Environmental Team with additional detailed input for the investigations.

4.3.2 Construction Solicitation Documents Preparation

4.3.2.1 Initiate Final Requests for Engineering Input into Construction Solicitation Documents

Upon receipt of field surveys, verify that they have been performed as requested and are complete and include all requested deliverables. Upon receiving the soils report for the project from the Geotechnical Team, read and understand the report and required construction items to be included in the construction documents.

Investigate the impacts of construction using multiple lifts in coordination with the Geotechnical Engineer and the rest of the design team. Seek a plan that will provide high levels of protection quickly while minimizing costs. Consider the need for future lifts due to long-term settlement, sea level rise, etc., in order to maintain authorized Level of Protection throughout project design life.

Based on required embankment design from the soils report, determine initial fill quantities and consult with the Geotechnical Team to determine most suitable borrow area(s) to take detailed surveys and borrow borings. Request ROE for borrow area surveys and borrow borings. Drainage impacts of the required embankment sections shall be investigated and the Hydraulics Team shall be consulted to determine adjustments to existing drainage features.

Request Environmental Team to begin all HTRW, cultural resource and environmental clearances of project and borrow area. The request shall include all information and drawings as stipulated in the appendix to this section describing Engineering Input.

4.3.2.2 Right of Entry for Construction

Prepare request for ROE into right of way (ROW) from Real Estate Team.

Prepare ROW drawings showing limits of project and existing and new ROW (if needed), required construction easements, required limits of construction within existing ROW and all temporary access easements. The required design section shall be applied to the existing surveys to determine extents of work to be constructed outside of existing ROW. A meeting with the Geotechnical Team shall be held to determine if there are any alternative design sections to keep the design section within existing ROW (i.e. structural solutions, reinforcement geotextile to reduce berm section, etc.). A cost comparison shall be investigated to determine most feasible solution (i.e., acquire new ROW vs. cost of I-Wall, T-Wall or a geotextile reinforced section).

Determine impacts to required new right of way of the authorized grade level of protection and 50 year future level of protection design section (to be provided from Geotechnical Team with final soils report). Meet with the Project Manager to evaluate the 50 year future levee footprint and potential to construct 50 year future design section versus authorized grade levee. Consideration will also be given to acquiring a minimum 15 feet beyond the toe of the levee to enhance access for maintenance and to keep trees and adjacent construction well clear of the design section.

Send request for ROE for construction ROW (with ROW drawings as prepared above) to Real Estate Team.

4.3.2.3 Construction Solicitation Documents

Using the design input from all PDT members, prepare detailed plans for construction of the flood protection project. Include all necessary details for construction of the flood protection project. Prepare specifications including all required technical specification sections and a bid schedule to include all biddable items. Calculate all quantities.

Conduct Independent Technical Review (ITR). Upon completion incorporate all changes to the construction solicitation documents as a result of the ITR. Obtain ITR certification.

Conduct BCOE Review. Construction solicitation documents shall be sent to all PDT members, other offices required, local sponsor, utility owners, and all local, state, and Federal agencies as required to review the P&S.

A plan-in-hand site inspection shall be conducted during BCOE review period. The PDT, Construction Division representatives, local sponsor and other persons as required based on project shall take part. Any changes to existing site conditions, potential design changes, etc shall be documented and photographed. A brief report of the plan-in-hand inspection indicating significant findings shall be prepared and disseminated to the PDT.

Evaluate all comments from the BCOE review.

Note that all reviews shall be conducted in DrChecks, and are considered complete when all comments are closed. The comments and comment evaluations must be thoroughly reviewed and checked prior to final input into the DrChecks review system.

A BCOE comment resolution meeting shall be held with all commentors to evaluate comment responses and resolve any and all comments not adequately evaluated. A brief report shall be prepared for the files transcribing discussions during the meeting. All DrChecks comments shall be closed out for BCOE completion.

All changes as a result of BCOE review shall be incorporated into the construction solicitation documents. All quantity calculations shall be verified and the documents checked thoroughly prior to the 100% complete documents being delivered to begin advertisement.

4.4 Engineering Input for NEPA

4.4.1 Description of Work

Provide a general description of work including the purpose and need for the work and alternatives considered. The description must include the following: method and duration of construction, time of construction (season, daytime only 24 hr. etc), equipment used, description of site preparation (grubbing etc), types of equipment used, description of construction access routes to include haul roads, residential routes and flotation channels etc., borrow needs and location. If any borrow material is utilized note the source, location, deposition area and whether the pit is existing and permitted or new.

4.4.2 Maps and Drawings

Provide an electronic copy (Jpeg or PDF file) of project vicinity map vicinity map.

Provide an electronic copy (Jpeg or PDF file) of the project footprint and construction area as an overlay on the most current aerial photography of the site. The electronic site map should also include latitude and longitude, north arrow, and identifier place name.

Provide engineering drawings (jpeg or PDF) of levee sections excavation etc.

Line work should include acreage of footprint affected by the project, limits of work, construction right of way, no work zones, stockpile area, staging areas, wash racks, fuel containment areas etc.

Where the footprint will exceed the original levee footprint the new area of impact should be clearly indicated on the drawing.

Required borrow areas should be identified on a vicinity map, current aerials with north arrow, latitude and longitude as well as acreages of the pit delineated.

Drawings of canal work or channel improvements denoting dredged depth and changes in configuration.

Drawings shall note all areas such as commercial storage, abandoned gas/fuel stations, etc., which could contain obvious potential HTRW or environmental issues. The designer shall consult with the Environmental Team Leader on such areas to determine applicability of engineering information to be provided.

4.4.3 Borrow Material

Prepare a map of borrow areas as noted above.

Note cubic yards of material used.

Provide soil type (as noted in borings i.e. sandy loam, clays w/ organics etc.).

Provide containment analysis if applicable.

Note where material is deposited (either stockpiled, used or both).

CEMVN-ED-L

MEMORANDUM FOR C/Design Services Branch Date:

SUBJECT: Survey Request Form Job No.:

1. Job Title:

2. Job Location:
   Levee District:
   Nearest Town:

3. Type of Survey: (Check as Applicable)
   a. [ ] Cross-Sections; Approx Number:
   b. [ ] Profile(s); Estimated Length:
   c. [ ] Hydrographic
      Referenced to [ ] C/L [ ] B/L
   d. [ ] B/L Traverse; Estimated Length:
      [ ] New [ ] Re-establish [ ] Recover [ ] Offsets Allowed
   e. [ ] C/L Traverse; Estimated Length:
      [ ] New [ ] Re-establish [ ] Recover [ ] Offsets Allowed
   f. [ ] Reference Off-sets
   g. [ ] Other: ______________________________
      Station Limits   Cross-Section Interval   Shot Int.

4. Control: Vertical / Horizontal
   Enclosed: [ ] Yes [ ] No / [ ] Yes [ ] No
   Datum: [ ] NAVD88 / [ ] NAD-1927
   [ ] NGVD-1929 (MSL) / [ ] NAD-1983
   [ ] CAIRO / [ ] Pre-83
   Epoch: [ ] 2004.65
   [ ] Post-83 (Latest)
   [ ] Other Pre -83
   Accuracy Required:
   [ ] 3rd / [ ] 3rd

5. Description of work to be performed:

6. Field Books Required: [ ] Yes [ ] No

7. Right of Entry Available: [ ] Yes [ ] No
   Available by:
   Requested on:

8. Please Provide: [ ] Cost Estimate
   [ ] Time Schedule
   [ ] Resume of Negotiations

Figure 4.1 Sample Survey Request Form (continued on next page)

9. Cost Account #'s:
   Contract:
   In-House:
   S & I:

10. Date Completed Survey Required:

11. Copy of Plans, Maps, Drawings, Etc. Enclosed: [ ] Yes [ ] No

12. Point of Contact:

LOUIS E. DANFLOUS, P.E.
Chief, Civil Branch

Encls

Figure 4.1 Sample Survey Request Form (continued from prior page)

5.0 STRUCTURES

5.1 In General

This guidance applies to structures whose primary function is flood protection in the New Orleans area, which includes T, L & I-walls, sluice gates, fronting protection and flood gates. Sector gates and other navigable waterway structures shall have all design criteria approved prior to design.

The Corps of Engineers is governed by engineering regulations (ER's), engineering manuals (EM's), engineering technical letters (TL's) and engineering circulars (EC's). These Corps publications are available on line at the following web site: http://www.usace.army.mil/inet/usace-docs. The designer is responsible for compliance with all civil works engineering regulations, circulars, technical letters and manuals (Corps publications). For convenience, this document highlights certain Corps publications that engineers should be aware of. Also, specific design criteria are identified in the following sections that may not agree with the Corps publications; in this case, the more conservative criteria shall be applied. Industry standards shall apply when Corps criteria is not applicable.

5.1.1 Sampling of References

USACE Publications

- EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures, June 92 (Including Change 1, Aug 03)
- EM 1110-2-2105, Design of Hydraulic Steel Structures (including Change 1), May 94
- EM 1110-2-2502, Retaining and Flood Walls, Sept. 89
- EM 1110-2-2906, Design of Pile Foundations, Jan. 91
- EM 1110-2-2503, Design of Sheet Pile Cellular Structures Cofferdams & Retaining Structures, Sept. 89
- EM 1110-2-2504, Design of Sheet Pile Walls, Mar. 94
- EM 1110-2-2705, Structural Design of Closure Structures for Local Flood Protection Projects, Mar. 94
- EM 1110-2-1901, Seepage Analysis and Control for Dams, Apr 93
- EM 1110-2-2100, Stability Analysis of Concrete Hydraulic Structures, Dec 05

Technical Publications

- American Concrete Institute, Building Code and Commentary, ACI 318-02
- American Institute of Steel construction, Manual of Steel Construction (9th Ed.)
- American Welding Society, AWS D1.1 (2006)

- American Welding Society, AWS D1.5 (2002)
- ASCE 7, Minimum Design Loads for Buildings and Other Structures

Computer Software

- CE Pile Group Analysis Program, “CPGA”
- CE Structural Analysis Program, “C-Frame”
- CE Strength Analysis of Concrete Structural Elements, “CGSI”
- CE Sheet Pile Wall Design/Analysis Program, “CWALSHT”
- Structural Analysis and Design Software, “STAAD”
- Ensoft, “Group 7.0”
- Additional approved USACE programs

5.1.2 Survey Criteria

Surveys shall conform to “USACE New Orleans District Guide for Minimum Survey Standards” (see Section 9) and the following guidance at a minimum. A typical scope of services for surveys in support of structural designs is included in Section 9.4.

5.1.3 General Design Criteria

Walls shall be constructed using the latest datum from Permanent Benchmarks certified by NGS - NAVD88 (2004.65).

The following is a summary of protection heights for various wall systems:

- I-Walls – 4 ft. maximum height
- T-Walls – No height limit; Typically 4 ft. and greater
- L-Walls / Kicker Pile Walls – 8 ft. maximum

Structural Superiority – All structures that are difficult to construct due to their nature, such as railroad and highway gates, pump station fronting protection, sector gates, utility crossings, etc., shall have a minimum 2 ft. overbuild. The overbuild is only required for new structures.

All I-walls shall have 6 in. minimum overbuild.

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. Global stability, as it affects T-wall foundation design, is addressed in Section 3.4.3 T-Wall Design Procedure.

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. L-walls shall resist boat impact where applicable.

Typically, I-walls shall **not** be used on navigable waterways or where there is the potential for barge/boat impact loading unless measures (such as berms for grounding vessels or separate pile fender systems) are taken to protect the wall. However, I-walls are acceptable as tie-ins to levee embankments. Site and soil conditions will dictate their use in these applications.

Lengths of L-Wall or T-wall monoliths should generally be 40 to 60 feet between expansion joints. I-wall monoliths should generally be 30 to 40 feet. At PI Corners, walls shall extend monolithically past the corner a minimum of 5 feet, but not less than two full sheet pilings and at least one row of bearing piles.

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.

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.

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.

5.2 T-wall & L-wall Design Criteria

T-walls, whose primary function in the New Orleans area is flood protection, are pile founded structures that consist of a reinforced concrete wall and base with steel sheet pile cut-off. Steel or prestressed concrete piles are battered towards the protected and flood sides and are the main components that support the concrete wall and base. The primary purpose of the steel sheet piling is to provide a seepage cutoff beneath the wall.

Previous experience has shown T-walls to perform well; even in situations where the floodwall was overtopped and experienced loadings beyond their intended design. T-walls are typically considered for a floodwall system in cases where there is a potential for barge or boat impact or there is a potential of foundation instability due to hydraulic loading.

L-walls are similar to T-walls except that the steel sheet pile replaces the flood side pile row.

Revised T-wall design procedures are included in Section 3.4.

5.2.1 Loading Conditions

1) Load Cases. See Section "5.7 General Load Case Tables."

2) Impact Cases. See Section "5.9 Boat/Barge Impact Loading Tables & Maps."

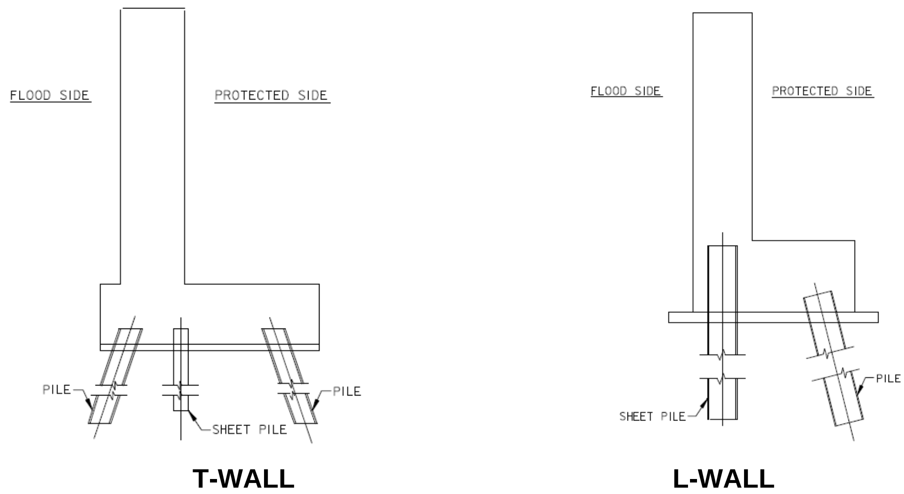


Figure 5.1 Typical T-Wall and L-Wall configuration

5.2.2 Pile Design – Precast-Prestress Concrete, Steel H and Pipe

The factors of safety with no overstress for all MVN projects are:

	With Pile Load Test	W/O Pile Load Test
Q-Case	2.0*	3.0
S-Case	---	1.5

* FOS = 2.5 must be used with a PDA test for the Q-case (for compression piles only)

Actual unfactored service loads shall be used in any pile analysis. See Sections 5.7 and 5.9 for further details on required FOS with various overstress conditions. Unless considered in the pile load test, the increased friction capacity due to the added length of a battered pile versus the vertical component shall be ignored.

Piles battered at a slope steeper than 1H on 8V shall be analyzed as vertical piles.

Weight of piles may be neglected in pile design.

Maximum structural deflections at pile heads:

Normal case, no overstress allowed
Vertical – 0.50” or less
Horizontal – 0.75” or less

Case with 16⅔ % overstress allowed
Vertical – 0.583” or less
Horizontal – 0.875” or less

Case with 33⅓ % overstress allowed
Vertical – 0.67” or less
Horizontal – 1.0” or less

Larger deflections may be allowed for design checks if stresses in the structure and piles are not excessive. Larger deflections are limited to values that remain in the elastic state of the soil.

Reductions for pile spacing shall be investigated with input from the MVN Geotechnical Branch. Final design reductions shall only be allowed with approval of MVN Geotechnical Branch.

A minimum pile embedment of 9” is required. For this depth of embedment, the connection is assumed to be pinned. A pile embedment length equal to or greater than twice the pile depth or diameter is required to develop full fixity for a pile embedded in the base of the structure. Any embedment depth between these two options must be researched to determine the applicable connection. CERL Technical Report M-339, dated Feb 1984 and entitled “Fixity of Members Embedded in Concrete, is a recommended information source.

The embedded portion of a pile consists of the solid concrete or steel section and does not included the tension hooks. See Figure 5.2.

When only 2 rows of piles are present, tension connectors shall be used on all piles.

When 3 or more rows of piles are present, tension connectors are required on all tension piles. Tension connectors are not required on compression piles unless any load case for a particular pile induces a compressive load in the pile less than 15% of the maximum compressive load in that pile.

Splices are prohibited in the upper half of the pile. Handling holes are permitted in the embedded depth of the pile and in the lower half of the pile. The total hole area shall not exceed 15% of the flange area. Holes are prohibited when driving stresses exceed 90% Fy.

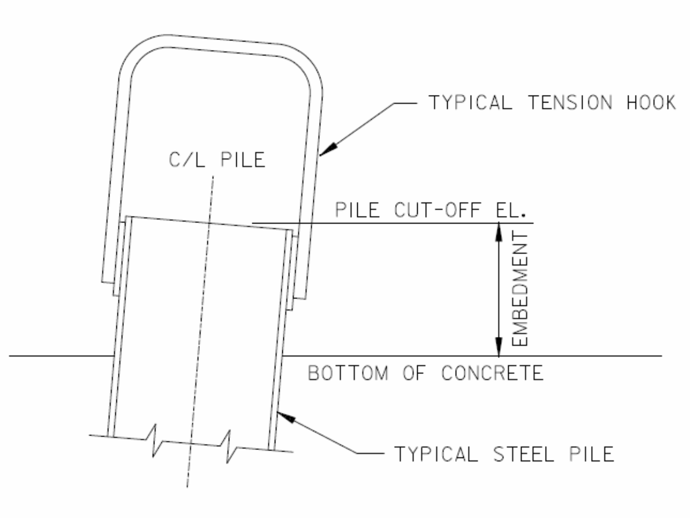


Figure 5.2 Depth of pile embedment

5.2.3 T-wall Sheet Piling Section

The primary purpose of the steel sheet piling is a pile acting to control seepage. Piping and Seepage Analysis methods are described in Section 3.4.2.5.

If unbalanced forces exist, design the steel sheet piling cut-off to extend to the critical failure plane plus embedment into the stable layer below. Embedment minimum is 5 feet.

If no unbalanced forces exist, a minimum PZ-22 hot rolled sheet piling shall be utilized for seepage cut-off.

The sheet pile shall be adequately anchored into the base slab to resist pull out. This can be achieved by passing U-bars through existing handling holes or burning holes in the sheet pile, if necessary.

5.2.4 L-wall Sheet Piling Section

The steel sheet piling is a pile acting to control seepage and provide support to the structure.

The sheet pile shall be designed to take the tension loads resulting from an inverted T-Wall analysis (CPGA) for the listed loading conditions. In addition, the sheet pile shall be designed as a compression member for the dead load case.

The minimum sheet piling section shall be a hot rolled PZ–27.

Due to the embedment of the sheet pile, approximately 2.75 to 3.0 feet into the base slab, the sheet pile should be assumed to be a fixed pile in the CPGA program.

The sheet pile properties should be assumed to be the summation of the pile properties for the kicker pile spacing.

The sheet pile shall be adequately anchored into the base slab to resist tension loads. This can be achieved by the use of welded studs or welded tension connectors.

5.2.5 Sheet Piling Tip Penetration

See the Geotechnical Section of this document for sheet pile tip penetration requirements for T-walls & L-walls.

5.2.6 Reinforced Concrete Section

Reinforced concrete hydraulic structures must follow Corps criteria (EM 1110-2-2104).

ACI factored loads are typically multiplied by an additional hydraulic factor, Hf = 1.3. The hydraulic factor is used to improve crack control in hydraulic concrete structures by increasing reinforcement requirements, thus reducing steel stresses.

The moment from the piles transferred into the base slab must be considered when designing the concrete reinforcement. Care must be taken to ensure proper moment orientation. A pile moment which is beneficial to the design shall be neglected.

5.3 I-wall Design Criteria

5.3.1 Loading Conditions

(1) Load Cases. See Section “5.7 General Load Case Tables.”

(2) Impact Cases. See Section “5.9 Boat/Barge Impact Loading Tables & Maps.”

5.3.2 I-wall Sheet Piling Section

The steel sheet piling is a pile acting to control seepage and provide support to the structure.

Design the steel sheet piling using the factored moments and shears obtained from the geotechnical design for tip penetration.

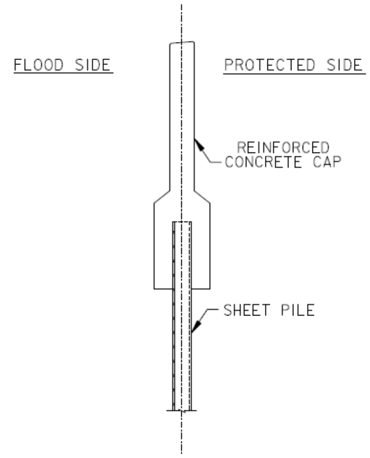


Figure 5.3 Typical configuration

The minimum sheet piling type shall be hot rolled PZ–27. However, I-walls **within** the levee tie-ins may have as a minimum a hot rolled PZ-22.

The sheet pile shall be adequately anchored into the concrete stem to resist pull out. A minimum embedment of 2’-9” shall be used on PZ-35 or smaller sheet pile. Bond development shall be checked for larger sheets. The projected area of the sheet piling shall be sufficiently embedded to develop bond between the piling and concrete cap adequate to resist the moment couple force. Additionally, U-bars shall be passed through existing handling holes or by burning holes in the sheet pile.

I-wall sheet pile shall be designed such that settlement is limited to an acceptable amount and differential settlement is negligible. Settlement of the cap should be less than 6 inches. Deviations shall be approved in advance by the USACE engineer of record. Concrete capping of walls shall be delayed in levees with anticipated settlement until movement has subsided. In the interim, the sheet piling shall be extended to the project Design Grade.

Maximum horizontal displacement shall be determined by USACE structural engineer of record.

5.3.3 I-wall Sheet Piling Tip Penetration

See the Geotechnical Section of this document for sheet pile tip penetration requirements for I-walls.

5.3.4 Reinforced Concrete Section

Reinforced concrete hydraulic structures must follow Corps criteria (EM 1110-2-2104).

ACI factored loads are typically multiplied by an additional hydraulic factor, H_f = 1.3. The hydraulic factor is used to improve crack control in hydraulic concrete structures by increasing reinforcement requirements, thus reducing steel stresses.

It is recommended that all I-walls shall be at least 2 ft. thick. There shall be a minimum 6" of concrete clear over the sheet piling section.

5.4 Temporary Retaining Structure (TRS) Design Criteria

A TRS is used for braced excavation construction purposes. The TRS design is the responsibility of the contractor but shall be submitted for approval. Where applicable, construction live loads shall be considered in the TRS design; a common minimum is 200 pounds per square foot. For braced excavations constructed in water, only hot-rolled piling shall be permitted. Boat impact shall be applied where applicable unless protective marine fenders are included in the TRS design.

5.4.1 General Notes (Flood Protection)

For TRS walls that serve as or impact flood protection, the post-Katrina hurricane protection design criteria shall be applied.

Areas below the required flood protection elevation will be considered breaches in the protection. Contractors will be permitted to allow an area in the existing flood protection to fall below the required elevation provided that area can be closed with steel sheet piling in a maximum of twenty-four (24) continuous hours. The sheet pile materials for closing such breaches shall be stockpiled at the site. Plans for closing breaches in the floodwall shall be updated periodically to reflect the status of construction progress. The Contractor shall develop and submit for approval, plans, including methods, equipment, materials and actions to close breaches in the event that an impending storm or high water event threatens the area. Prior to removing any existing flood protection, the Contractor shall have the plan of interim protection approved.

5.4.2 Sheet Piling Section (for Non-Flood Protection)

Design the steel sheet piling, using the moments and shears obtained from the geotechnical design for tip penetration, with allowable steel stresses, $F_b = 0.65\ F_y$ and $F_v = 0.40\ F_y$.

If archweb "U" piles are used, then the design shall account for and include calculations for shear transfer across their interlocks. Arch web piles or piles with interlocks at or near their center of gravity tend to slip under loading when the shear transfer cannot be achieved across their interlocks. Arch web piles shall be designed in accordance with the recommendations set forth in the standard CUR 166 published in 1993 in Holland by the Center for Execution, Investigations and Standardization in Civil Engineering (CUR), available from New Orleans District, Corps of Engineers, ED-T. Anti-slipping connections such as welding or crimping of the interlocks can be employed to help prevent displacement of the interlocks. The design calculations shall include all assumptions and shall consider the type(s) of soil, the effects of water, type of wall (i.e. cantilevered versus braced and shall include the location and number of wales, struts, etc), whether the piles are driven singly, in pairs, triple, etc., effects of phased excavation, treatment of the interlocks (i.e. how shear transfer is accomplished through welding or crimping), references cited, and any other considerations.

5.4.3 General Notes (for Non-Flood Protection)

The option or requirement to flood the excavation during a potential flood event may be used.

Design single steel struts or tie rods, using the maximum anchor forces obtained from the geotechnical design, using the latest AISC industry standards.

Design multiple steel struts, using the maximum un-factored forces obtained from the geotechnical design, using the latest AISC industry standards.

Design the steel wales, using the maximum anchor forces obtained from the geotechnical design, using the latest AISC industry standards.

Design the anchors and deadmen, using the maximum anchor forces obtained from the geotechnical design, using the latest AISC and ACI industry standards.

5.4.4 References

- "Steel Sheet Piling Design Manual", United States Steel Corporation
- "Steel Sheet Pile Design Manual", Pile Buck Inc.
- "Engineering Manual for Sheet Pile Walls", Virginia Tech Department of Civil Engineering
- "Design of Sheet Pile Walls", USACE Engineering Manual EM 1110-2-2504
- "CUR 166", published in 1993 in Holland by the Center for Execution, Investigations and Standardization in Civil Engineering (CUR) ('Dammwandconstructies' Civieltechnisch Centrum Uitvoering Research en Regelgeving, Holland

5.5 Reinforced Concrete Design Criteria

5.5.1 Structural Concrete

f_c' = 4000 psi minimum – 28 day strength (except concrete piles) or 90 days if pozzolans are used to replace cement. (3000 psi for incidental structures).

f_c' = 5000 psi minimum (prestressed concrete piles).

Thermal considerations: Slab and wall components that are greater than 4 feet thick shall require a thermal analysis. A simplified Level 1 analysis, as specified in ETL 1110-2-542 (dated 30 May 97), will suffice. A low-heat mix shall be included in the project specifications when analysis proves thermal stresses are elevated. A low-heat mix can be achieved by replacing the chirt aggregate with limestone; the larger the aggregate size the better. Additionally, replace the cement content with as much pozzalan as possible. Not all flyash and slags reduce heat. The most benefical are Class F flyash and Grade 120 ground granulated blast-furnace slag.

5.5.2 Steel Reinforcing

Steel reinforcing shall be ASTM A615 Gr. 60 with f_y = 60 ksi
(Designs utilizing f_y > 60 ksi are not allowed)

Steel reinforcing for prestress concrete shall be:

Prestressing strands, Grade 250	250,000 psi
Prestressing strands, Grade 270	270,000 psi

5.5.3 Load Factors

Single Load Factor of 1.7 for dead and live loads shall be used in addition to a Hydraulic Factor.

Hydraulic Factor of 1.3 shall be applied to both shear and moment.

Hydraulic Factor of 1.65 shall be used for member in direct tension. This includes base sections that have a net lateral tensile reaction from loads and piles in tension.

Strength reduction factor for bending shall be 0.9

Strength reduction factor for shear shall be 0.85

5.5.4 Steel Requirements

Maximum Flexural Reinforcement
$0.25\ p_b$ (Recommended)
$0.375\ p_b$ (Permitted w/o special studies)

Minimum Flexural Reinforcement
ACI Code

Temperature Reinforcement
$0.0028A_g$ (1/2 in each face)

5.5.5 Concrete Requirements

Clear Cover (except for channel lining) (Also see Section 12.0 – Typical Drawings):

- 2” min. for concrete sections equal to or less than 12” in thickness.
- 3” min. for concrete sections greater than 12” and less than 24” in thickness.
- 4” min. for concrete sections equal to or greater than 24” in thickness and when concrete is placed directly in contact with the ground.

Minimum Wall Thickness:

- T-walls = 24” minimum
- L-walls and I-walls = the width of the sheet piling plus 12”

Tapered walls are not recommended.

5.5.6 Lap Splices

See typical drawings and details in Section 12.0 for Lap Splice charts and notes.

Splices shall be staggered whenever possible. Otherwise, the ACI code shall be adhered to.

Mechanical Splices
1) Mechanical Connectors
2) Thermit Welding (Cadweld) (Only use when necessary)
3) Welding (Never to be used)

When using mechanical splicers, do not add the coupling device to a short bar (usually equal to the lap length) that in turn laps to a long length. This creates two lap splices at the same location. Lap splices should be held to a minimum.

5.5.7 General Notes

Any prestress concrete (except piles) shall be approved in advance by the USACE engineer of record.

In a base slab where 3 or more pile rows are present, it is recommended that primary and secondary reinforcing steel be placed above piles when possible.

When primary steel is placed above embedded piles, temperature steel shall be placed in the depth of concrete below the primary steel (typically 12 inches). The temperature steel requirement is based on the depth of concrete below the primary steel, not the total depth of concrete.

5.6 Miscellaneous

5.6.1 Material Unit Weights

MATERIAL	UNIT WT (lb/ft^3)
Water	64
Concrete	150
Steel	490
Rip rap	132
Semi-Compacted Granular Fill	120
Fully-Compacted Granular Fill, Wet	120
Fully-Compacted Granular Fill, Effective	58
90% -Compacted Clay Fill, Wet	110
90% -Compacted Clay Fill, Effective	48

5.6.2 Loading Considerations

1) Concrete
- Unit weight of monolith
- Neglect weight of stabilization and tremie slab when beneficial to the foundation loading (i.e. uplift)

2) Water
- SWL Elev. (Hydrostatic pressure)
- Wave Loading (exclude the water weight due to the wave weight above the SWL when designing the foundation)

3) Soil
- Vertical - Use Unit Weight
- Horizontal - Use Unit Weight and K at rest values

  K_o = 0.8 for clay

  K_o = 0.5 for granular materials

  K_o = 0.5 for rip rap

4) Wind
- Use ASCE 7 to determine max wind force
- 50 psf minimum

5) Uplift
- Impervious sheet pile cut-off, 100% effective
- Pervious sheet pile cut-off, slopes uniformly along base from flood side uplift at flood side edge of base to protected side uplift at protected side edge of base
- See Section "5.8 Examples of Uplift Cases."

5.6.3 Structural Steel Design

Minimum steel thickness = 5/16" (corrosion control)

Allowable stress = 5/6 of AISC allowable stress

The ASD method shall be used. The LRFD design method may **not** be used for structural steel design.

The American Welding Society, AWS D1.5 (2002) code shall be used for fracture critical members.

Welded structures should be welded all around (seal welded). Welds shall be designed and not simply made full penetration as the cost and residual stresses imparted by unequal cooling are detrimental. Weld inspection and NDT shall be made part of the contract requirements.

5.6.4 Steel Sheet Pile Design

For unbalanced load cases:

$F_b = 0.75\ f_y$
$F_v = 0.45\ f_y$

For all other cases:

$F_b = 0.5\ f_y$
$F_v = 0.33\ f_y$
F_a = 5/6 AISC allowable

Thickness = 0.375 in. minimum

Only hot-rolled sections are allowed.

5.6.5 Gate Design

5.6.5.1 Concrete Monolith

For the foundation design of most of the gate monoliths in our flood protection system, a rule of thumb for the pile layout is to use battered piles to resist the horizontal loads at the columns and use vertical piles to resist vehicular and railway loads in the center of the monolith. Engineering judgment shall be used to determine the zone of influence to resist the horizontal loads in respect to battered pile placement. Where unbalanced loads are present in the foundation design, battered piles may also be required in the center. Low unbalanced loads may also be transferred to the end walls where battered piles are concentrated.

5.6.5.2 Steel Gates

Gates 12 feet tall or less may utilize a two girder system. The gates are considered low head and need not comply with Fracture Critical Requirements. Girder splices are not recommended, but when approved the splice shall be NDT tested along 100% of the length. Stress levels and deflections shall limit the girder capacity. Stress levels shall be kept below 0.5 Fy and stresses about both axis maintained below 75% of unity.

Gates taller than 12 ft. to 16 ft. tall may also utilize a two girder system, but must meet all fracture critical criteria for a hydraulic steel structure. Fracture critical requirements are specified in ER 1110-2-8157. Non-redundant tension members shall comply with AWS D1.5 and 100% of welded tension connections shall be NDT tested, including all plates and stiffeners welded to the tension flange of both girders. Splices to the critical horizontal girders are prohibited.

Gates taller than 16 ft. shall utilize at least three girders. At the hinge column, the third girder shall transfer the load to the column through an additional hinge. For welded connections, AWS D1.1 is adequate. Splices to the critical horizontal girders are prohibited.

Roller Type Gates. Consideration should be given to the design of the gate in respect to rolling the gate into placement. New gates may be very large and will pose concerns when the gate is moved into position. Roller gates shall be used when the clearance requirements within the closure swing cannot be guaranteed.

Swing Type Gates. The use of three hinges or extension of columns and tension supports should be considered for gates that are very large in height. The top hinge tends to bind when moving gates that are very heavy. Adjustable bottom seals shall be added where slight variations in sill height could occur (i.e. road pavement topping improvements).

Overhead Roller Type Gates. The use of this type of gate shall be of last resort. If there are no problems with swing tolerances, then we recommend using a swing gate.

Miter Type Gates. The latching of the gates after placed into the closed position is very critical for the proper function of the miter gate. A latching system should be investigated if miter gates are being considered. The latch shall resist all applicable design hurricane protection design cases.

5.6.6 General Design Considerations

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.

Where non-displacement piles are required and corrosion is not a controlling factor, consider H-piles or pipe piles; otherwise, investigate the use of prestressed concrete piles which are typically more cost effective. Steel piles are required in foundations that experience an unbalanced load.

5.6.7 Utility Crossings

For a structural alternative on utility crossings, the utility shall only be allowed to pass through a pile founded L-Wall or T-Wall. Utilities should pass through a properly sealed pipe sleeve in the cut-off sheet piling. On case-by-case bases, utilities may pass through the concrete wall and in general, should not be attached. See Section 12.0 for typical examples. A typical drawing specifying utility clearances is included in Section 12.

All Utility Crossings shall approved by the USACE engineer of record.

5.6.8 Painting

Only coal tar epoxy shall be used.

Steel sheet, H and Pipe pilings that will be installed in new fill, disturbed materials or fluctuating water tables shall be painted with a coal tar epoxy system. The H-piles and sheet piling shall be painted 3 inches above the stabilization slab and to a 5 ft. minimum below new fill material, disturbed soil or the lowest elevation of fluctuating water tables. Use engineering judgment for final painting requirements.

5.6.9 Levee Tie-ins, Transitions and Scour Protection

Typical scour protection details can be found in Appendix C. These drawings show work typical to date and are provided for information only. Future ERDC and IPET reports shall be used for guidance.

ERDC Overtopping Protection can be found in Appendix D. This is for reference and not to be considered guidance.

Proper engineering judgment and settlement considerations shall be used to determine the proper level of scour protection. Scour protection materials and details should be properly engineered and suitable for the specific site location. Scour protection on the flood side should be considered on a case-by-case basis, especially if hurricane wave loading exists.

95% compaction of the scour protection sub-base shall be considered to minimize settlement.

Scour protection is required on the protected side of all I-walls. Scour protection shall transition a minimum of 10’ into any adjacent T-wall or L-wall sections then curve inward at a radius equal to that of the protection width.

Proper earthen cover and scour protection are mandatory. Future settlement should be accounted for in detailing scour protection over the sheeting piling.

Typical MVN details should be used for transitions from L-Wall or T-wall to T-wall, L-Wall or T-wall to I-wall and L-Wall or T-wall to uncapped sheet piling (slip joint). See Section 12.0 for typical drawings.

The tie-in details for T-Walls, L-Walls and I-walls that terminate into a levee section must follow the latest guidance. As a minimum, the uncapped cut-off sheet piling must extend horizontally 30 feet into the full levee section. Tip penetration in the transition zone shall continue at the full depth of the adjacent sheetpile unless a reduction in depth is supported by a seepage analysis showing that the transition would not be flanked.

A minimum hot rolled PZ-22 shall be used at all levee tie-ins.

5.7 General Load Case Tables

Following are general load case tables. It is important to note that these tables are not inclusive of all possible scenarios.

Table 5.1 General Load Cases

LC No.	Overstress Allowed		Load Case Name	Description
	Fdn.	Wall		
LC 1a	16⅔ %	16⅔ %	Construction	Dead load 200 psf equipment surcharge No uplift No wind
LC 1b	33⅓ %	33⅓ %	Construction plus Wind	Dead load No unbalanced load No uplift Wind from protected side
LC 2a	0	0	Water to SWL (impervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off No wind[2] No boat/barge impact
LC 2b	0	0	Water to SWL (pervious)	Dead load Unbalanced load (if present) Pervious sheet pile cut-off No wind[2] No boat/barge impact
LC 2c	33⅓%	50%	Water to SWL plus Barge Impact (impervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off No wind See “Boat/Barge Impact Loading Tables & Maps”
LC 2d	33⅓%	50%	Water to SWL plus Barge Impact (pervious)	Dead load Unbalanced load (if present) Pervious sheet pile cut-off No wind See “Boat/Barge Impact Loading Tables & Maps”
LC 3a	33⅓%	33⅓%	Water to SWL plus Wave Load (impervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off No wind Wave load applied
LC 3b	33⅓%	33⅓%	Water at SWL plus Wave Load (pervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off No wind Wave load applied

LC No.	Overstress Allowed		Load Case Name	Description
	Fdn.	Wall		
LC 4a[1]	50%	75%	Water to SWL plus Wave Load plus Barge Impact (impervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off No wind Wave load applied See "Boat/Barge Impact Loading Tables & Maps"
LC 4b[1]	50%	75%	Water to SWL plus Wave Load plus Barge Impact (pervious)	Dead load Unbalanced load (if present) Pervious sheet pile cut-off No wind Wave load applied See "Boat/Barge Impact Loading Tables & Maps"
LC 5a[1]	0	0	Water to Reverse Head plus Wind (impervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off No boat/barge impact Wave load applied
LC 5b[1]	0	0	Water to Reverse Head plus Wind (pervious)	Dead load Unbalanced load (if present) Pervious sheet pile cut-off No boat/barge impact Wave load applied
DC A	33⅓%	50%	Water to Top of Wall (pervious or impervious)	Dead load No unbalanced load Pervious or impervious sheet pile cut-off No wave load No wind load No boat/barge impact load
DC B	50%	50%	Water to Top of Wall (pervious or impervious)	Dead load With unbalanced load Pervious or impervious sheet pile cut-off No wave load No wind load No boat/barge impact load

LC No.	Overstress Allowed		Load Case Name	Description
	Fdn.	Wall		
DC C[1]	67%	75%	Water to Top of Wall plus Barge Impact (impervious)	Dead load Unbalanced load (if present) Impervious sheet pile cut-off No wave load No wind load See "Boat/Barge Impact Loading Tables & Maps"
DC D[1]	67%	75%	Water to Top of Wall plus Barge Impact (pervious)	Dead load Unbalanced load (if present) Pervious sheet pile cut-off No wave load No wind load See "Boat/Barge Impact Loading Tables & Maps"

NOTES:
1. If applicable; i.e. not all structures will be subject to barge impact.
2. If wind is applied, a 33⅓% overstress is allowed.
3. Boat impact shall be assumed to be concentrically placed when designing the monolith foundation. Eccentric impacts shall be checked

5.8 Examples of Uplift Cases

Following are examples of uplift cases.

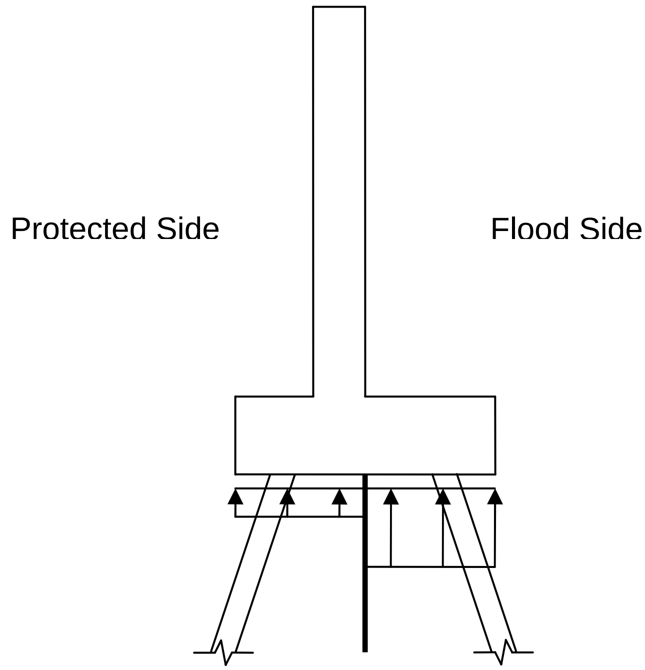


Figure 5.4 Impervious Sheet Pile Cut-off

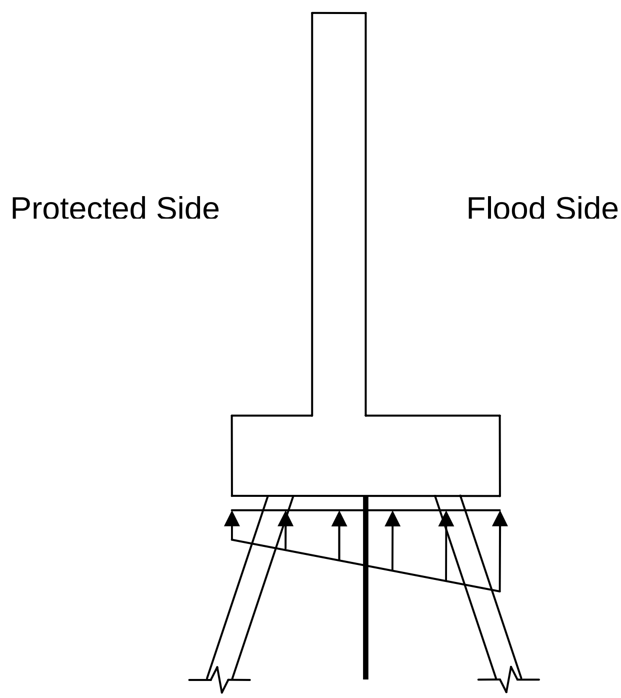


Figure 5.5 Pervious Sheet Pile Cut-off

5.9 Boat/Barge Impact Loading Tables & Maps

Impact loads for boats and barges shall be considered as shown in the following tables and Figures 5.6 through 5.9 at a minimum.

Table 5.2

HURRICANE PROTECTION - BASIC LOAD CASE COMBINATIONS (01-12-07)								
LOAD CASE	% ALLOWABLE OVERSTRESS		PILE LOAD - FACTORS OF SAFETY (FOS)					
	WALL	FOUNDATION	STATIC LOAD TEST		PDA LOAD TEST		NO LOAD TEST	
			C	T	C	T	C	T
I. CONSTRUCTION	16⅔	16⅔	1.70	1.70	2.15	2.60	2.60	2.60
II. CONSTRUCTION + WIND	33⅓	33⅓	1.50	1.50	1.90	2.25	2.25	2.25
III. STILL WATER LEVEL (SWL)	0	0	2.00	2.00	2.50	3.00	3.00	3.00
IV. SWL + WIND	33⅓	33⅓	1.50	1.50	1.90	2.25	2.25	2.25
V. SWL + WAVE	33⅓	33⅓	1.50	1.50	1.90	2.25	2.25	2.25
VI. SWL + ** BOAT IMPACT (BI)	50	33⅓	1.50	1.50	1.90	2.25	2.25	2.25
VII. SWL + WAVE + **BI	75	50	1.33	1.33	1.67	2.00	2.00	2.00
VIII. SWL + UNBALANCED LOAD	0	0	2.00	2.00	2.50	3.00	3.00	3.00
IX. REVERSE HEAD	0	0	2.00	2.00	2.50	3.00	3.00	3.00

HURRICANE PROTECTION - DESIGN CHECKS (10-07-06)								
LOAD CASE	% ALLOWABLE OVERSTRESS		PILE LOAD - FACTORS OF SAFETY (FOS)					
	WALL	FOUNDATION	STATIC LOAD TEST		PDA LOAD TEST		NO LOAD TEST	
			C	T	C	T	C	T
I. WATER TO TOP OF WALL, NO UNBALANCED LOAD + NO WAVE LOAD	50	33⅓	1.50	1.50	1.90	2.25	2.25	2.25
II. WATER TO TOP OF WALL, UNBALANCED LOAD + NO WAVE LOAD	50	50	1.33	1.33	1.67	2.00	2.00	2.00
III. WATER TO TOP OF WALL, W/ OR W/O UNBALANCED LOAD + ** BOAT IMPACT (BI)	75	67	1.20	1.20	1.50	1.80	1.80	1.80

<u>* GENERAL NOTES:</u>
1. If unbalanced load is present for the SWL load case, it shall be included in all SWL load case combinations.
2. Actual unfactored service loads shall be used in any pile analysis program (CPGA).
3. An increase in allowable deflections will be allowed for overstress conditions. Sound engineering judgement shall be utilized in deciding the appropriate overstress.

<u> NOTES ON BOAT IMPACT:</u>**
1. For SWL cases, apply (BI) 3-ft above SWL.
2. For water to top of wall, apply (BI) at top of wall.
3. Design assuming a 100 kip load where barge impact can occur now or in the future, or a 50 kip load for other vessels such as pleasure craft or work boats. A minimum boat impact load of 0.5 kips/ft shall be applied where applicable. Current obstructions that are marginal and have a high probability of not lasting the project life shall be assumed non-existent.
4. Wall load distribution. The load shall be distributed over a 5 foot width plus the width gained along a 45-degree angle.
5. Foundation load distribution. The load shall be distributed over the full width of the monolith foundation. Effects of any eccentric loading caused by impact at one end of the monolith (moment about the vertical axis) should be based on sound engineering judgement.
6. Gate load distribution. The loads shall be distributed over a 5 foot width on the upper girder. No load is assumed on the lower girder(s). A 33⅓% overstress is permitted in this case.
7. Minimum thickness for walls subject to boat impact shall be 24 inches.

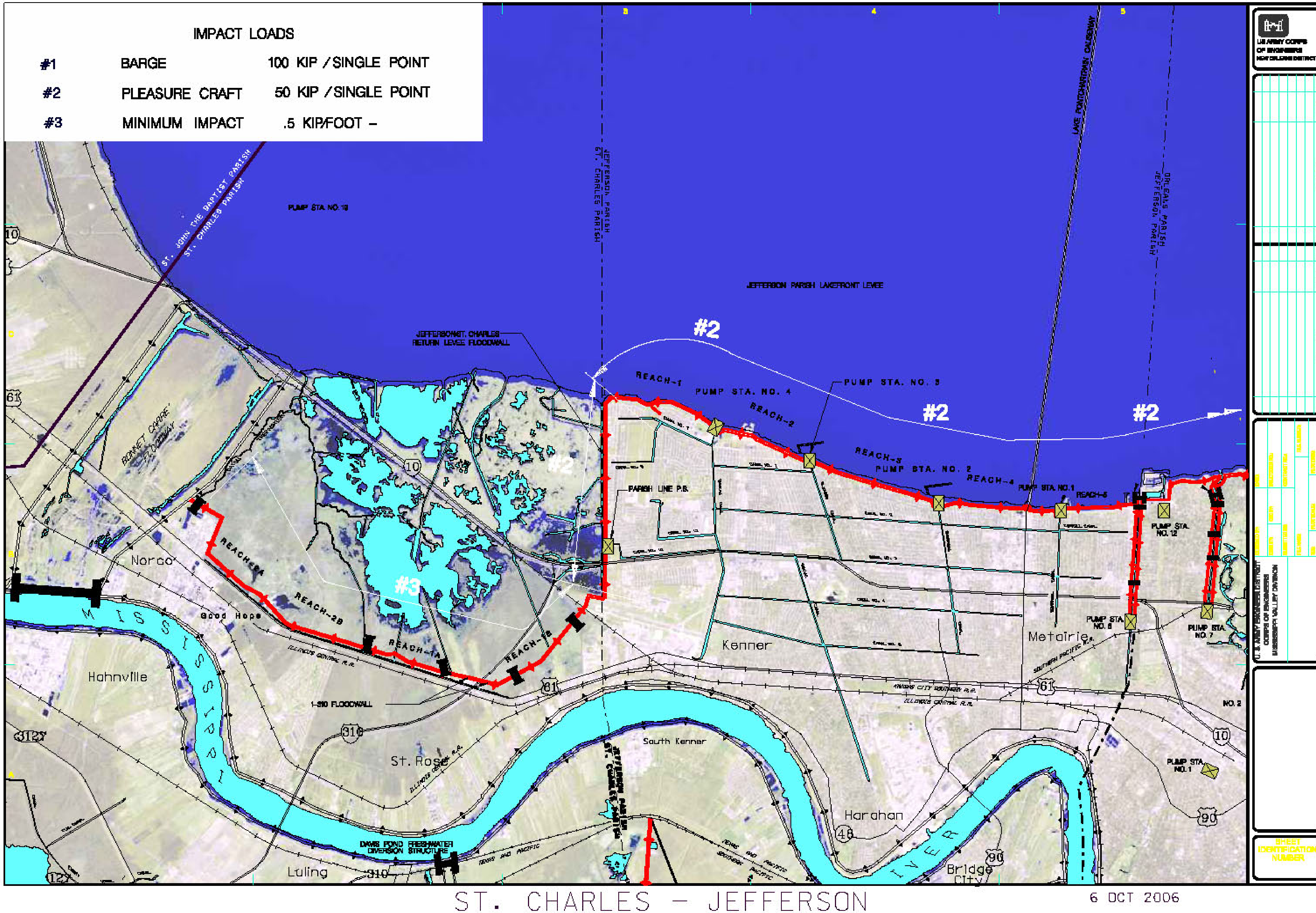


Figure 5.6 Boat/Barge Impact Map

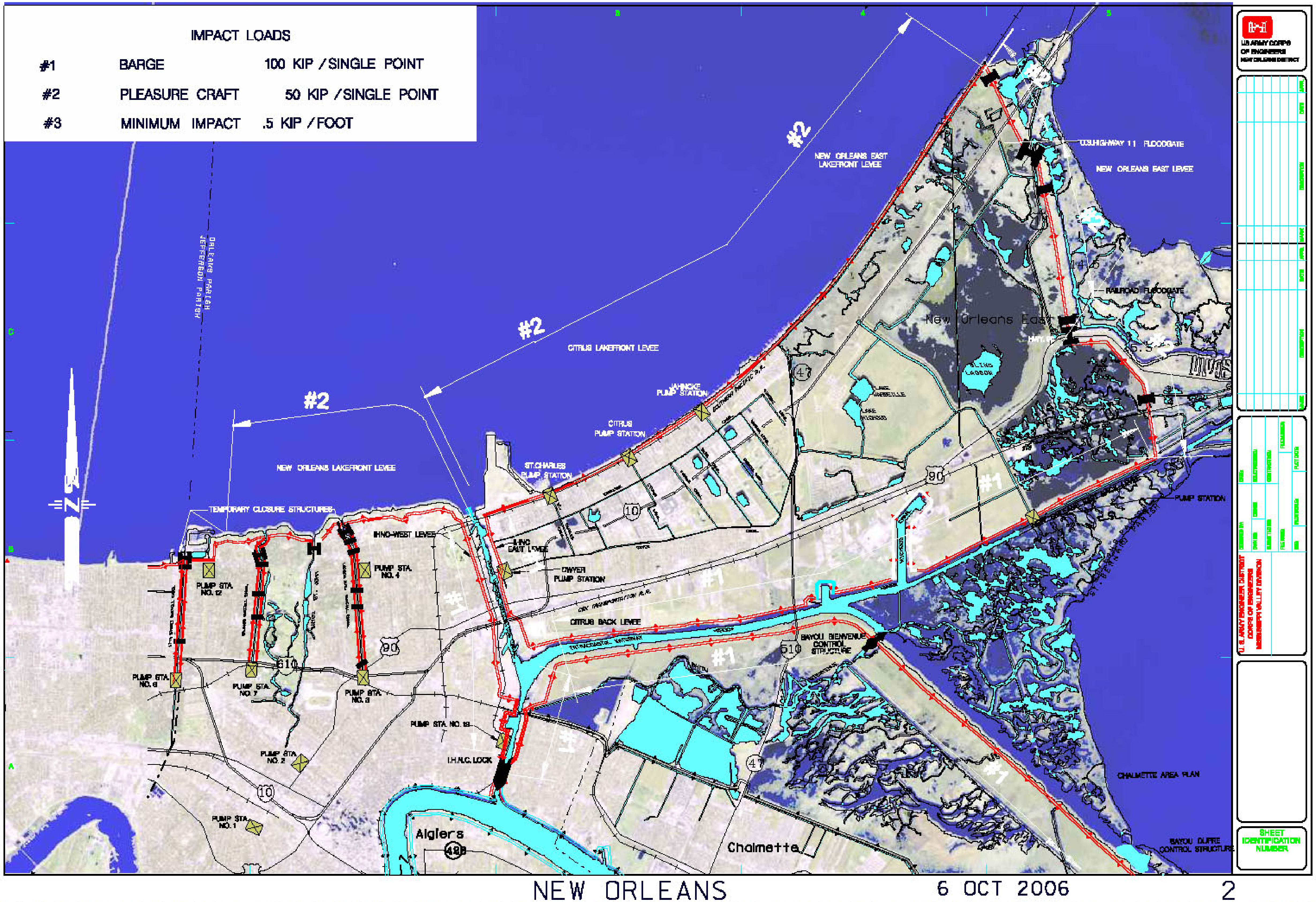


Figure 5.7 Boat/Barge Impact Map

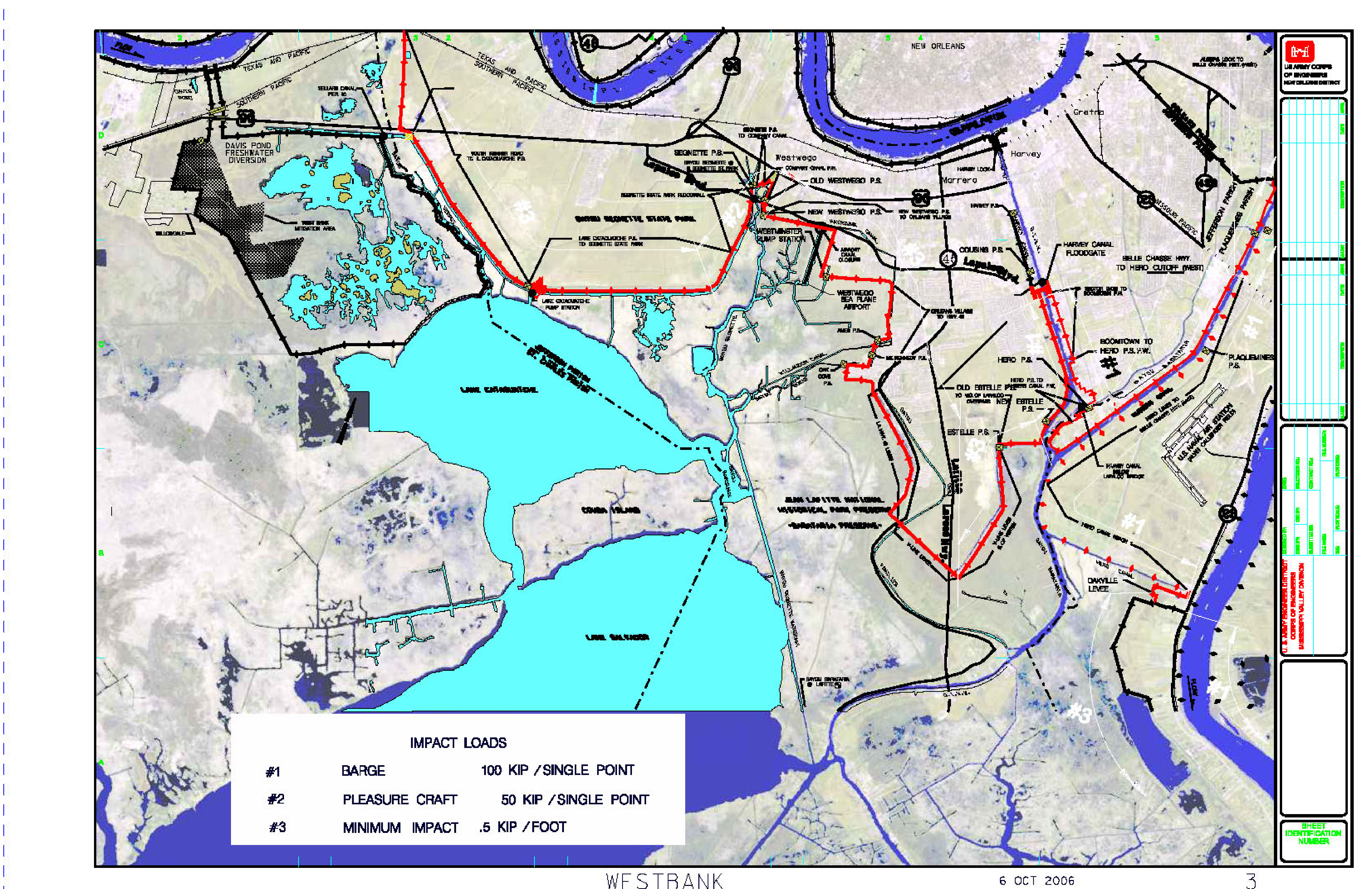
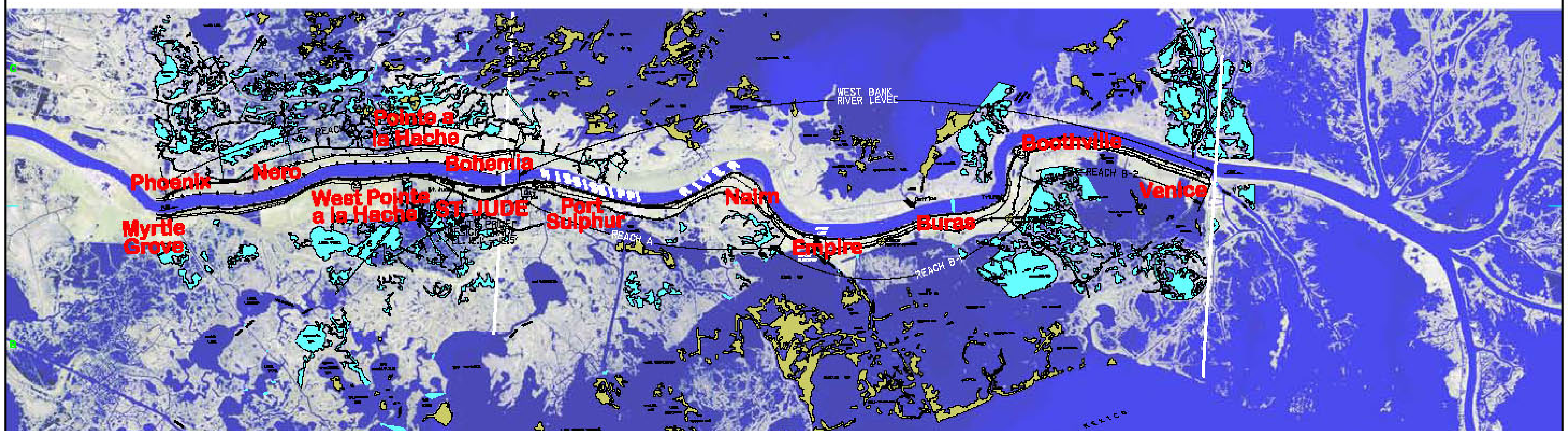


Figure 5.8 Boat/Barge Impact Map

IMPACT LOADS

1	BARGE	100 KIP / SINGLE POINT
2	PLEASURE CRAFT	50 KIP / SINGLE POINT
3	MINIMUM IMPACT	.5 KIP / FOOT



MISSISSIPPI RIVER WALLS / STRUCTURES – #1

WEST BANK BACK LEVEE – #3

EAST BANK BACK LEVEE – #3

EXCEPTION – MARINA AREAS – #2

PLAQUEMINES PARISH 6 OCT 2006 4

Figure 5.9 Boat/Barge Impact Map

6.0 MECHANICAL & ELECTRICAL

6.1 Sampling of References

- EM 1110-2-3102, General Principles of Pumping Station Design & Layout
- EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations
- UFGS 221000.0010, Vertical Pumps Axial Flow & Mixed Flow Impeller Type
- UFGS 334500.0010, Speed Reducer for Storm Water Pumps
- UFGS 416510.0010, Diesel & Natural Gas Fueled Engine Pump Drives
- NFPA 37, Standard for the Installation and Use of Stationary Combustion Engines and Gas Turbines

6.2 Mechanical

Mechanical systems should conform to established USACE criteria and standards with attention to the following suggested guidelines.

1. Wherever possible use vertical pumps, with form suction intakes (FSI).

2. Locate operating floor above maximum expected flood elevation.

3. Provide redundant flood protection by installing shut off gates at the pumps discharge.

4. Use aluminum pipe for combustion air intake ducts.

5. Design control room to be a “safe room” for continued operation during hurricane conditions. Safe rooms should have the capability to start, stop and monitor pump units, and control discharge gates and the trash rake. They also should have living accommodations for personnel during and after the storm. Redundant communication systems with backups should be provided also.

6. Provide diesel-driven generators for backup power supply. If pumping station is to be located in an area rich in underground natural gas distribution lines, specify natural gas engines in lieu of diesel engines for: (a) reliability (no storage tanks, transfer pumps, piping, level controls), (b) simplicity of design and (c) vandalism protection.

7. Other equipment or components should be elevated above the maximum expected flood elevation as much as practicable, including the fuel storage area, access roads to the pump station, the fuel distribution system. Attention shall be given to the elevation of combustion air filter/silencers, trash screen cleaners motors and controls.

8. When clean water is required for bearing lubrication, provide a local water well source as a backup for municipal water.

9. Provide event recorders which also record water levels. Recorders should be automated in both operation and reporting.

6.3 Electrical

All Electrical Systems shall confirm to the established USACE criteria and standards with attention to the following suggested guidelines:

1. Locate all electrical equipment including back-up generators, electrical controls and external electrical connections above maximum expected flood elevation.

2. Back-up power should be sized and designed for operation during storms to provide adequate power for station ventilation, lights, HVAC, fuel transfer pumps, trash rake cleaners, automatic pump lubricators, air compressors and all other critical systems.

PART B: STANDARDS

UPDATED 04 OCT 07

7.0 UTILITY RELOCATIONS QUESTIONNAIRES

Following are sample questionnaires to be used to collect information from owners of affected facilities.

7.1 Company Information

(revised 9/04)

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
RELOCATIONS SECTION
COMPANY INFORMATION

____Project, __________________, LA

1. Official Name of Facility/Utility Owner, as reflected in the records of the Louisiana Secretary of State:

____

2. Type of Business Entity (check one):

___ Limited Liability Company (LLC)

___ Corporation

___ Partnership

___ Other (define):

3. Provide name of state of incorporation:

____

4. If the state of incorporation is not Louisiana, has the corporation registered with the Louisiana Secretary of State as a foreign corporation?

____ YES

____ NO

5. Provide information about nature of work or corporate purpose:

____

____

____

____

6. Provide name, address, telephone number and e-mail address of person available for contact by Corps of Engineers:

Right-of-Way Department ____

____

Legal Department ____

____

Other ____

____

7. Provide information about real property upon which facilities are located. Is it owned in fee, servitude, or leased?

____

____

8. If facility owner has written recorded rights-of-way and/or lease, provide a copy of rights-of-way document and/or lease, and if the document is recorded, provide the recordation information.

9. Please explain any and all predecessor(s) in interest:

____

____

____

____

____

10. Indicate width of right-of-way.

____

11. If facility is a pipeline, is it a common carrier?

____

12. If facility was placed pursuant to a permit, provide the name of agency that issued permit (including, but not limited to, permits for Section 10 of The Rivers and Harbors Act of 1899 and permits from municipalities or local governments), the permit number, and the date on which the permit was issued. Please attach a copy of the permit or the Corps of Engineers letter explaining that no permit was needed, if the company had applied for such a permit.

____

____

____

____

____

13. The date the facility was first installed: ____________________________

7.2 Communication Lines

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
RELOCATIONS SECTION
DESCRIPTIVE INFORMATION FOR COMMUNICATION LINES

1. Company Name:

2. Description (trunk, primary, etc.):

3. Size (pair, gauge, etc.):

4. Type (aerial, buried, submerged, etc.):

5. Location

USACE Project Baseline Station:

Longitude, Latitude Coordinates:

6. Function Served:

7. Date Installed:

8. Design Life:

9. Total Length of Facility:

10. Current Status of Facility (active, inactive, abandoned, etc.):

11. Clearance (height from lowest line crossing over project to top elevation of project):

12. Other Pertinent Data (manholes, towers, etc.):

7.3 Highway Bridges

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
RELOCATIONS SECTION
DESCRIPTIVE INFORMATION FOR HIGHWAY BRIDGES

1. Company Name:

2. Location (city, street, road, highway served, etc.):

3. Type of Bridge (concrete, steel, timber, etc.):

4. Design Load:

5. Description (bents, piers, decking, foundation, piling, etc.):

6. Embankment Slope Protection in Channel (type, thickness, etc.):

7. Number of Bridges and Lanes with Clear Width Dimension:

8. Class of Road Served (primary, secondary, class-I, 2, A, B, etc.):

9. Traffic Information (daily traffic count, type of traffic, etc.):

10. Provide Drawings (showing profile, overall length, spans, decks, pile penetration and elevation of high water on bridge):

7.4 Navigation Lights

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
RELOCATIONS SECTION
DESCRIPTIVE INFORMATION FOR NAVGATION LIGHTS

COMPANY ____

1. Description. (size, type facility, etc.) ________________________________

____

2. Number of Hours in Service ______________________________________

____

3. Width of Existing R-O-W ____

____

4. Location (See Note) ____

5. Latitude/Longitude ____

5. Functions Served ____

6. Date Installation ____

____

7. Design Life ____

____

8. Other Pertinent Data ____

Note: Question #4, if facility crosses project, give project stationing.

7.5 Conveyor Shafts

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
RELOCATIONS SECTION
DESCRIPTIVE INFORMATION FOR CONVEYOR SHAFTS

COMPANY ____

1. Description. (size, type facility, etc.) ________________________________

____

2. Number of Hours in Service ____

____

3. Width of Existing R-O-W ____

____

4. Location (See Note) ____

5. Latitude/Longitude ____

5. Functions Served ____

6. Date Installation ____

____

7. Design Life ____

____

8. Other Pertinent Data ____

Note: Question #4, if facility crosses project, give project stationing.

7.6 Pipelines

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
RELOCATIONS SECTION
DESCRIPTIVE INFORMATION FOR PIPELINES

1. Company Name:

2. Size (Diameter) and Type of Facility:

3. Type of Construction (Steel, cast iron, etc):

4. Function Served (oil, gas, water, etc):

5. Location

USACE Project Baseline Station:

Longitude, Latitude Coordinates:

6. Date Installed:

7. Design Life:

8. Total Length of Facility:

10. Current Status of Facility (active, inactive, abandoned, etc.)

11. Other Pertinent Data (Manholes, Valves, etc):

12. Depth of pipeline beneath levee or channel

7.7 Powerlines

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
RELOCATIONS SECTION
DESCRIPTIVE INFORMATION FOR POWERLINES

1. Company Name:

2. Description (transmission, primary, distribution, etc.):

3. Size (voltage, gauge, etc.):

4. Type (aerial, buried, submerged, etc.):

5. Location of utility pole(s) supporting powerline.

USACE Project Baseline Station

Offset from levee centerline

5. Location (where line crosses levee centerline)

USACE Project Baseline Station:

Longitude, Latitude Coordinates:

6. Function Served:

7. Date Installed:

8. Design Life:

9. Total Length of Facility:

10. Current Status of Facility (active, inactive, abandoned, etc.):

11. Clearance (height from lowest line crossing over project to top elevation of project):

12. Other Pertinent Data (manholes, towers, etc):

7.8 Railroad Bridges

U.S. ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT
RELOCATIONS SECTION
DESCRIPTIVE INFORMATION FOR RAILROAD BRIDGES

1. Company Name:

2. Type of Bridge (timber, steel, concrete, etc.):

3. Number of Tracks:

4. Description of Superstructure:

5. Width of Right of Way:

6. Designed Load:

7. Quantity and Type of Trains Scheduled Daily:

8. Provide Drawings (show profile, overall length, spans, pile penetration, and elevation of high water on bridge):

9. Other Pertinent Data:

8.0 GEOTECHNICAL INVESTIGATIONS

Throughout this section, reference to the "Contractor" simply means the entity responsible for the subject work. The same procedures and requirements generally apply to anyone providing geotechnical investigation services, whether the work is done in-house or by other USACE districts.

8.1 Contractor Requirements

Each work unit shall consist of personnel duly qualified and experienced to perform the type of required services. The Contractor shall use professional judgment in determining what equipment and/or supplies are needed to complete each delivery order assignment. The Government reserves the right to inspect and to monitor the activities of the A-E's work in determining that the A-E is performing the required services in accordance with Government standards procedures. The Contractor shall submit a time and cost estimate for each proposed assignment. The Contractor shall also submit detailed plans for performance of the work. The Contractor shall perform soil borings, testing, logging, reporting and plotting in the Corps of Engineers, New Orleans Districts (Government) format. The Corps of Engineers (or the Designer of Record) will pick the type of soil borings, boring sample size and length, boring locations, type and location of required soil lab tests.

8.1.1 Field Assignments

The Contractor will be responsible for locating, clearing, determining ground surface elevations and water tables, retrieving soil borings (including 1-7/8" I.D. Splitspoon, 3" general type and undisturbed type and 5" Undisturbed soil borings), sealing boreholes, and acquiring other equipment as necessary to complete the field assignments. Borings may include work in marsh areas, and/or work over water.

8.1.2 Office and Laboratory Assignments

The Contractor will be responsible for classifying and testing soil samples and computing, compiling and furnishing plotted boring logs of the resulting field and laboratory data.

8.1.3 Quality Assurance

The Contractor shall discuss each proposed assignment to develop a mutual understanding of:

(1) Type of work to be done
(2) Received Soil Boring Locations
(3) End result expected by the COR

(4) Methods to be used by the Contractor
(5) Format of computations and/or drawings
(6) Completion data required by a date to be determined by the Government.

A Government representative shall be present during boring retrieval and sample testing at various times to be determined by the Government in order to perform quality assurance and to verify boring sample quality and soils testing accuracy.

8.1.4 Government Furnished Materials

The Contractors automated computations shall follow the same format as that used by the Government. The necessary, locally derived, MS-DOS/Windows programs can be made available if required by the Contractor. Due to copyright laws commercially available off-the-shelf programs, such as CADD-type programs, will not be available from the Corps.

Point of Contact is Denis J. Beer, P.E. at Denis.J.Beer@usace.army.mil.

8.2 Subsurface Investigations

8.2.1 Locating and Setting-Up for Borings

Normally the Government will obtain right of entry to take soil borings and inform the local sponsor that clearing of small trees and underbrush may be required. The Contractor shall locate borings; cut brush and/or timber to provide access to the site; obtain latitude, longitude, ground surface elevations and water table elevations; and set-up soil boring drill rigs in the field. The Government (or Designer of Record) will furnish soil boring locations. The Government (or Designer of Record) will supply a map showing the soil boring locations. The locations will be either tied to a baseline with a station, distance and azimuth to each boring location, if one exists, or lat/longs or X-Y coordinates are provided for each location. Vertical control for use in determining the ground surface elevations of the borings will be furnished by the Contractor. The Contractor should contact the Government for benchmark information.

8.2.2 Sampling of Borings

The Contractor shall use a fixed-piston type sampling method (Hvorslev fixed-piston or equivalent) for (CH), (CL) and (ML) type soils and be capable of providing undisturbed sampling to depths of 300 ft. Bentonite based drilling mud shall be used throughout the sampling process to improve sample recovery and minimize sample disturbance. The sampler for (SM) and (SP) type soils shall be standard Splitspoon sampler (1-7/8 inch I.D., 2 inch O.D.). The driving resistance in blows per foot shall be determined for (SM) and (SP) type soil with a 140 pound driving hammer having a 30" drop. The Driller shall state the type of hammer used for the SPT, such as automatic or two rope-wraps around cathead.

The driller will measure the hammer energy delivered to the drill rods from the sampler for each drill rig. The hammer and how the hammer energy was obtained will be placed on the boring log. The Government or the Designer of Record should determine if a correction factor is applied to the SPT results. The Contractor should be aware that a number of borings will be taken from marshy environments that may require special equipment, such as marsh buggies.

8.2.2.1 Shelby Tube Sampling

The general type piston sampler shall utilize a minimum of 3 inch Shelby Tubes (3” O.D., approx. 2-7/8” I.D.) that are a minimum of 46 inches in length with sealing caps. An undisturbed type piston sampler shall utilize a minimum of 5 inch thin-wall Shelby Tubes (5” O.D., approx. 4-3/4” I.D.) that are a minimum of 54 inches in length with sealing caps. During sampling with the fixed-piston type sampler, the piston should be locked at the bottom of the sampling tube until it is seated on the bottom of the borehole. The piston shall be released, piston rod held in place, and the tube shall be pushed in one or two pushes to obtain sample. The sample tube is then removed hydraulically through a vacuum then with hoists and cables. The Contractor shall use the Government field boring log form as a record of soil stratification and soil sampling (see Figure 8.3). A copy of the original field boring logs shall be supplied to the Government. The soil samples will be preserved in airtight containers to prevent loss of moisture.

8.2.2.2 Sample Storage, Extrusion and Shipment

Once samples have been removed from the boreholes for undisturbed soil borings, they must be sealed within the sampling tube with end caps and ends taped prior to shipment to the laboratory for extrusion, classification and testing. Hydraulically activated sample jacks shall be used to extrude the samples from the tubes. Mechanical and pneumatically activated sample jacks shall not be used to extrude the samples. All tubes shall be identified and labeled immediately to ensure correct orientation and to accurately identify the samples. ENG Form 1742 and/or ENG Form 1743, as shown in Figure 8.2 (or equivalent), should be completed and securely fastened to each sample. Sample tubes shall be shipped to the testing laboratory such that they are not allowed to roll around in the shipping vehicle, nor should they be dropped or otherwise roughly handled. Samples shall be protected from extreme temperatures and exposure to moisture. Samples shall be extruded from the tube within 5 days after retrieval and shall be kept in air-tight container. Any samples that will be tested more than seven (7) days after extrusion shall be waxed. Waxed samples shall be stored in a humid room. All storage, extrusion and shipment procedures shall be done in accordance with EM 1110-1-1804, Chapter F, paragraphs 6-5 through 6-7. Samples remaining after testing will remain at the Contractor's office until the Government requests their disposal or collected by the Government.

8.2.2.3 Backfilling of Borehole

Upon completion of the borings, the borehole shall be grouted full depth in accordance with State of Louisiana regulations. Grout mix should consist of 2 part cement and 1 part bentonite and shall be tremie grouted from the bottom of the hole within three feet of the ground surface. The top three feet will be backfilled with native soil.

8.3 Laboratory Soil Testing

8.3.1 Laboratory Facilities

A laboratory preferably should be on a ground floor or basement with a solid floor and should be free of traffic and machinery vibrations. Separate areas should be designated for dust producing activities such as sieve analyses and sample processing. Temperature control of the entire laboratory is to be preferred. If the temperature-controlled space is limited, this space should be used for triaxial compression, consolidation, and permeability testing. A humid room large enough to permit the storage of samples and the preparation of test specimens should be available. The Contractor shall, at its own expense, obtain validation as an approved testing laboratory by the Materials Testing Center (MTC) of the Engineering Research and Development Center (ERDC). This shall be done in accordance with ER 1110-1-8100 and ER 1110-1-261. Depending upon the workload by the Government inspecting agency, acceptance or rejection of the Contractor proposed testing laboratory is usually done approximately 60 to 120 days after notification is received from the Contractor. The certification is typically valid for three years.

8.3.2 Soil Classification

The Contractor shall classify, record and plot soil data within 7 days of obtaining the samples from the field. A water content determination shall be made and recorded on all samples classified as (CH), (CL), and (ML). The Unified Soil Classification System and the “Guide for Moisture Contents adapted to CEMVN-ED-F Soils” shall be used in classifying the soils (see Figures 8.4 and 8.5). All data recorded during the classification process (including but not limited to strata elevations, soil type, moisture content, consistency, color and modifiers) shall be recorded and furnished on LMN form 721, Nov 69 (see Figure 8.1), as well as in a computer file format specified by the Government.

The soil borings logs shall also be plotted and supplied to the Government using computer software available from the government. Request should be made for the General Boring Log Program (FG002) and Undisturbed Boring Log Program (FS008). Note: This software will only execute under Micro Station SE or J. The location, number and type of soils testing shall be furnished to the Contractor within 3 days of the receipt of the soil classification and boring log data.

8.3.3 General Soils Testing

All general soils testing shall begin within 14 days of the receipt of the number and location of the soils tests from the Government (or the Designer of Record). Atterberg Limits determinations will be made on representative clay (CH) and clayey (CL & ML) fractions of the boring at a rate and/or at a location defined by the Government (or the Designer of Record). Grain size distribution determinations may be required; these may include both sieve and hydrometer testing. General soils testing shall be in accordance with EM 1110-2-1906.

8.3.4 Compressive Strength Tests

All compressive strength testing shall begin within 14 days of the receipt of the number and location of the soils tests from the Government (or the Designer of Record). An explanation of any atypical data, such as calibration factors, correction factors, shall be furnished in addition to the following. Upon request, the Contractor shall furnish to the Government duplicate samples of test specimens for possible testing by the Government.

8.3.4.1 Unconfined Compression Tests

Unconfined Compression Tests (UCT) described in EM 1110-2-1906 will be performed on representative samples on 3-inch general type and 5-inch undisturbed samples at an interval and/or at locations defined by the Government (or Designer of Record). UCT specimens shall have a diameter of 1.4 inches and a minimum length of 3.0 inches. UCT results shall include, but not be limited to, boring name, sample elevation, sample location, strain rate, specific gravity, water content, wet density, dry density, saturation, void ratio, diameter and height. In addition, the Contractor shall supply plotted compressive stress vs. axial strain plots, to include unconfined compressive strength, failure strain, and undrained shear strength.

8.3.4.2 Triaxial Shear Tests

Triaxial shear tests described in EM 1110-2-1906 will be required on selected 5 inch undisturbed samples. The 5 inch diameter sample shall be cut into 4 equal specimens such that each specimen can be trimmed for testing. The specimen size for triaxial testing shall be 1.4 inches in diameter and 3 to 3.5 inches in length. The triaxial shear test is defined by a suite of three tests performed at three different confining stresses (the maximum confining pressure shall be at least equal to the maximum normal pressure expected in the field with the project in place) performed on three trimmed specimens from the same 5-inch sample. The fourth specimen shall be tested if verification of one of the first three tests in necessary. The Triaxial shear testing will be Unconsolidated Undrained (Q) tests, Consolidated Undrained (R) tests with pore pressure data measured and recorded, and Consolidated Drained Direct (S) tests. The axial load induced to the specimens shall be done so at a rate of 1.0 percent per minute until an axial strain of 20 percent has been reached. A strain rate of 0.3 percent per minute shall be used for materials that achieve maximum deviator stress at about 3 to 6 percent strain. Results from triaxial tests shall include, but not be limited to, the boring name, sample elevation, sample location, Atterberg Limits, unit weight, specific gravity, water content, dry density, saturation, void ratio, diameter and height, back pressure, cell pressure, failure stress, ultimate stress, and deviator stress at failure. In addition, plotted stress strain curves and Mohr Circle plots shall be furnished for each specimen tested. Generated Mohr-Coulomb failure envelope plots (to include computer generated/selected compressive stress values (cohesion) and values for internal friction angles) shall be furnished.

8.3.5 Consolidation Testing

Consolidation tests described in EM 1110-2-1906 will be required on selected 5 inch undisturbed samples. The 5 inch diameter sample shall be trimmed to tightly fit a consolidation ring with diameter not less than 4.0 inches in diameter. The specimen should be loaded according to the following normal increments: 0.25, 0.5, 1.0, 2.0, 4.0, 8.0, and 16.0 tons per square foot. Lower starting load may be necessary for a sample with minor overburden. Readings of deformation (as determined from dial indicator readings) versus time shall be measured and recorded at the following times: 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 minutes and 1, 2, 4, 8, and 24 hours. If primary consolidation has not occurred in the first 24 hours, hold the load for an additional 24 hours each day until primary consolidation has occurred. Continuous saturation of the sample shall be maintained until each test is complete. Results from consolidation tests shall include, but not be limited to, the boring name, sample elevation, sample location, Atterberg Limits, specific gravity, water content, dry density, saturation, initial void ratio, and diameter and height of the sample. In addition, plotted curves of (1) applied pressure versus void ratio, (2) applied pressure vs. Cv, and (3) dial gage reading versus time for each load increment shall be furnished and (3) Casagrande construction to indicate the maximum past preconsolidation pressure.

8.3.6 Logging and Reporting

The results of the field borings and laboratory tests shall be shown and furnished on LMN form 721, Nov 69, as well as in a computer file format specified by the Government. The completed logs and test results shall be furnished to the Government no later than 15 days after testing has been completed. The soil borings shall also be reported and furnished as plotted stratified soil logs and shall contain all field/laboratory testing information. In addition the logs will be furnished and named as specified by the Government. They shall be furnished in a Windows 2000 compatible file format and/or Microstation 4.0 (or later) Intergraph CADD file format. The government will furnish the computer software necessary to plot the soil borings as stated in 8.3.2.

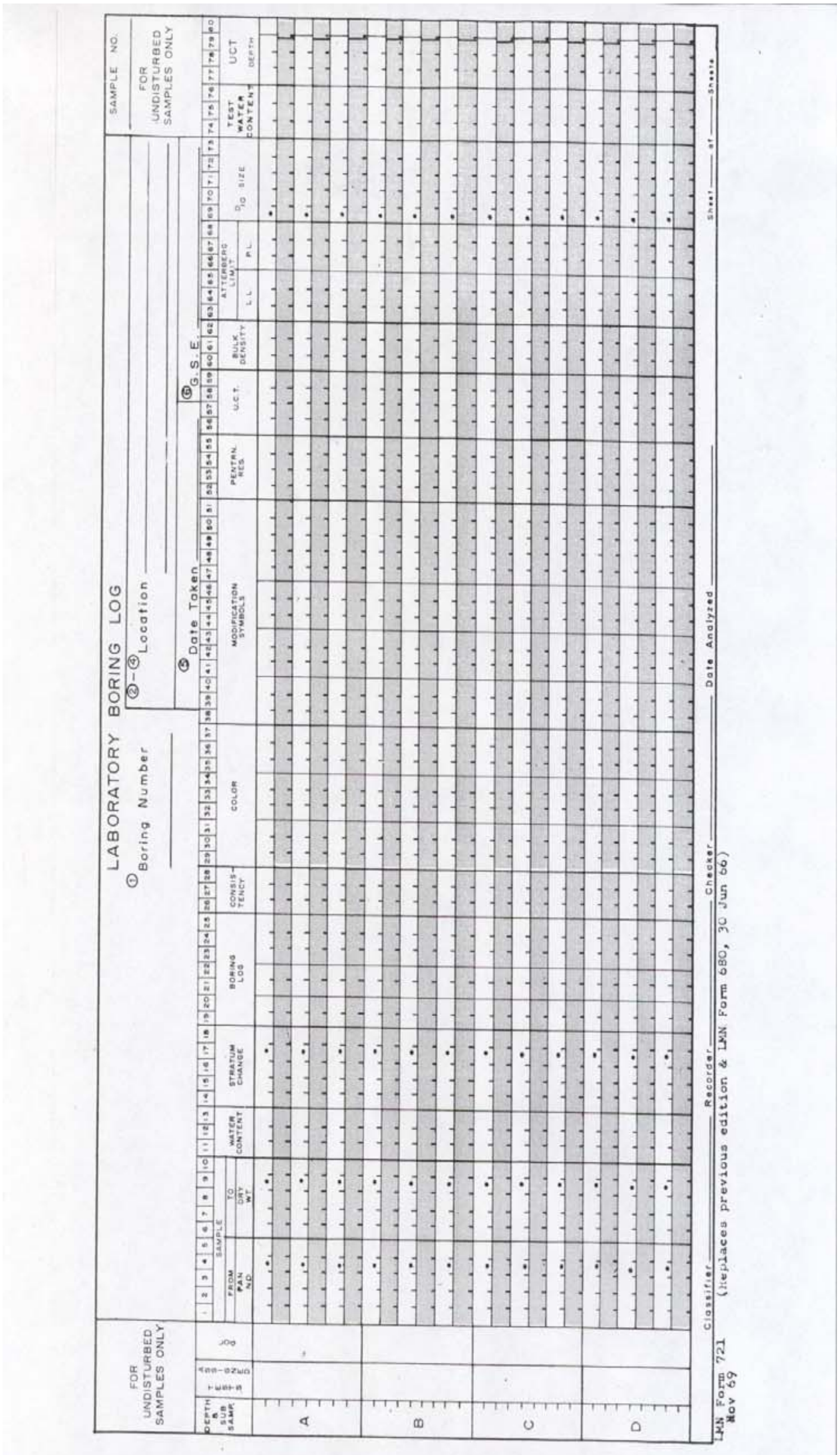
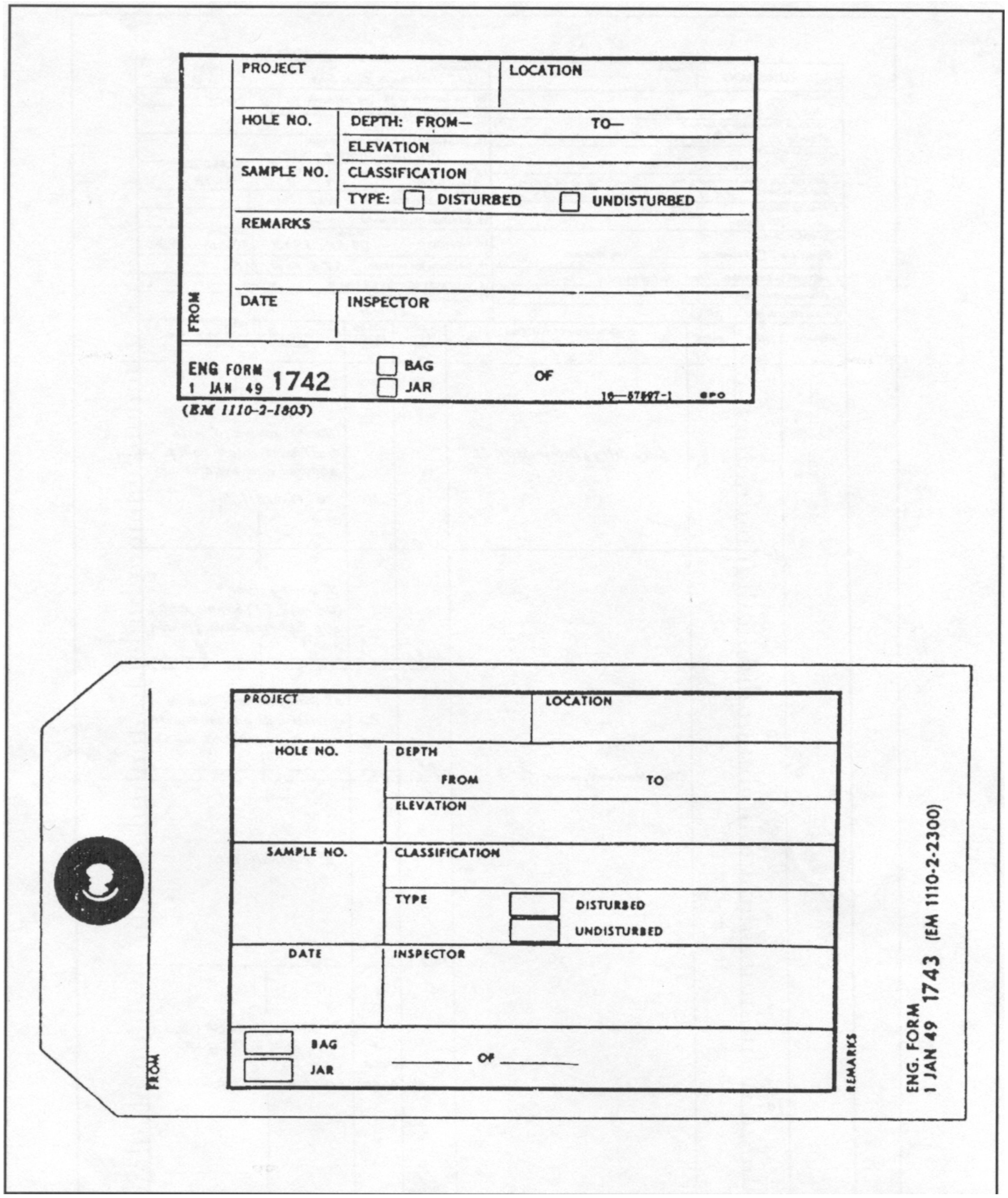


Figure 8.1. LMN Form 721



PROJECT
LOCATION
HOLE NO.
DEPTH: FROM— TO—
ELEVATION
SAMPLE NO.
CLASSIFICATION
TYPE: ☐ DISTURBED ☐ UNDISTURBED
REMARKS
DATE
INSPECTOR
FROM
ENG FORM 1 JAN 49 1742
☐ BAG
☐ JAR
OF
(EM 1110-2-1803)

PROJECT
LOCATION
HOLE NO.
DEPTH
FROM
TO
ELEVATION
SAMPLE NO.
CLASSIFICATION
TYPE
DISTURBED
UNDISTURBED
DATE
INSPECTOR
☐ BAG
☐ JAR
OF
FROM
REMARKS
ENG. FORM 1 JAN 49 1743 (EM 1110-2-2300)

Figure 8.2 ENG Forms 1742 & 1743

UPDATED 04 OCT 07

BORING LOG
FIELD DATA

Project ________________ Site ________________ Date ________

Location ________________________________ Job No. ________

Drill Rig ________ Inspector ________ Operator ________ Surface El ________ Boring No. ________

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

Sheet ________ of ________ Sheets

Figure 8.3 WES Form 819

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISION		TYPE	LETTER SYMBOL	TYPICAL NAMES
COARSE - GRAINED SOILS	GRAVELS		GW	GRAVEL,Well Graded,gravel-sand mixtures,little or no fines
			GP	GRAVEL,Poorly Graded,gravel-sand mixtures,little or no fines
		GRAVEL WITH FINES (Appreciable Amount of Fines)	GM	SILTY GRAVEL,gravel-sand-silt mixtures
			GC	CLAYEY GRAVEL,gravel-sand-clay mixtures
	SANDS	CLEAN SAND (Little or No Fines)	SW	SAND,Well-Graded,gravelly sands
			SP	SAND,Poorly-Graded,gravelly sands
		SANDS WITH FINES (Appreciable Amount of Fines)	SM	SILTY SAND,sand-silt mixtures
			SC	CLAYEY SAND,sand-clay mixtures
FINE - GRAINED SOILS		SILTS AND CLAYS (Liquid Limit < 50)	ML	SILT & very fine sand,silty or clayey fine sand or clayey silt with slight plasticity
			CL	LEAN CLAY,Sandy Clay,Silty Clay,of low to medium plasticity
			OL	ORGANIC SILTS,and organic silty clays of low plasticity
		SILTS AND CLAYS (Liquid Limit > 50)	MH	SILT,fine sandy or silty soil with high plasticity
			CH	FAT CLAY,inorganic clay of high plasticity
			OH	ORGANIC CLAYS of medium to high plasticity,organic silts
HIGHLY ORGANIC SOILS			Pt	PEAT,and other highly organic soil
WOOD			Wd	WOOD
SHELLS			SI	SHELLS
NO SAMPLE			NS	No Sample Retrieved

NOTE: Soils possessing characteristics of two groups are designated by combinations of group symbols.

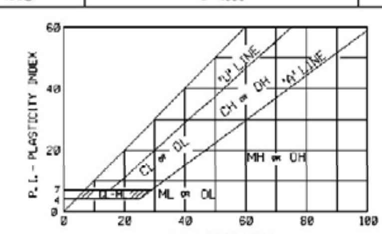
DESCRIPTIVE SYMBOLS

COLOR

COLOR	SYMBOL
TAN	T
YELLOW	Y
RED	R
BLACK	BK
GRAY	Gr
LIGHT GRAY	lGr
DARK GRAY	dGr
BROWN	Br
LIGHT BROWN	lBr
DARK BROWN	dBr
BROWNISH-GRAY	brGr
GRAYISH-BROWN	gyBr
GREENISH-GRAY	gnGr
GRAYISH-GREEN	gyGn
GREEN	Gn
BLUE	Bl
BLUE-GREEN	BlGn
WHITE	Wh
MOTTLED	Mot

CONSISTENCY FOR COHESIVE SOILS

CONSISTENCY	COHESION IN LBS./SQ.FT. FROM UNCONFINED COMPRESSION TEST	SYMBOL
VERY SOFT	< 250	vSo
SOFT	250-500	So
MEDIUM	500-1000	M
STIFF	1000-2000	St
VERY STIFF	2000-4000	vSt
HARD	> 4000	H



PLASTICITY CHART

For classification of fine-grained soils in accordance with ASTM D 2487

MODIFICATIONS

MODIFICATION	SYMBOL
Traces	Tr
Fine	F
Medium	M
Coarse	C
Concretions	cc
Rootlets	rt
Lignite fragments	lg
Shale fragments	sh
Sandstone fragments	sds
Shell fragments	slf
Organic matter	O
Clay strata or lenses	CS
Silt strata or lenses	SIS
Sand strata or lenses	SS
Sandy	S
Gravelly	G
Boulders	B
Slickensides	SL
Wood	Wd
Oxidized	Ox

NOTES:

FIGURES TO LEFT OF BORING UNDER COLUMN "W OR D_{10}"

Are natural water contents in percent dry weight

When underlined denotes D_{10} size in mm *

FIGURES TO LEFT OF BORING UNDER COLUMNS "LL" AND "PL"

Are liquid and plastic limits, respectively

SYMBOLS TO LEFT OF BORING

▽	Ground-water surface and date observed
Ⓒ	Denotes location of consolidation test**
Ⓢ	Denotes location of consolidated-drained direct shear test **
Ⓡ	Denotes location of consolidated-undrained triaxial compression test **
Ⓠ	Denotes location of unconsolidated-undrained triaxial compression test **
Ⓣ	Denotes location of sample subjected to consolidation test and each of the above three types of shear test **
FW	Denotes free water encountered in boring or sample

FIGURES TO RIGHT OF BORING

Are values of cohesion in lbs./sq.ft. from unconfined compression tests

In parenthesis are driving resistances in blows per foot determined with a standard split spoon sampler (1 3/8" I.D., 2" O.D.) and a 140 lb. driving hammer with a 30" drop

Where underlined with a solid line denotes laboratory permeability in centimeters per second of undisturbed sample

Where underlined with a dashed line denotes laboratory permeability in centimeters per second of sample remoulded to the estimated natural void ratio

*The D_{10} size of a soil is the grain diameter in millimeters of which 10% of the soil is finer, and 90% coarser than D_{10}

**Results of these tests are available for inspection in the U.S. Army Engineer District Office, if these symbols appear beside the boring logs on the drawings.

TYPICAL NOTES:

1. While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local variations characteristic of the subsurface materials of the region are anticipated and, if encountered, such variations will not be considered as differing materially within the purview of the contract clause entitled "Differing Site Conditions".
2. Ground-water elevations shown on the boring logs represent ground-water surfaces encountered in such borings on the dates shown. Absence of water surface data on certain borings indicates that no ground-water data are available from the boring but does not necessarily mean that ground-water will not be encountered at the locations or within the vertical reaches of such borings.
3. Consistency of cohesive soils shown on the boring logs is based on driller's log and visual examination and is approximate, except within those vertical reaches of the borings where shear strengths from unconfined compression tests are shown.
4. Unless otherwise noted:

   a. Undisturbed borings, indicated by the letter "U", are taken with a 5" I.D. Piston Type Sampler.

   b. General type borings are taken with a 1 7/8" I.D. Tube Sampler and/or a 1 3/8" I.D. Split Spoon Sampler.

REVISED 25 JUNE 92

SOIL BORING LEGEND

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: VOJKOVICH	DATE: APRIL 1993	CADD FILE: BORLEGAL.DGN
DRAWN BY: WOODS	PLOT DATE: 30 MAR 93	FILE NO. H-2-30962
CHECKED BY: RICHARDSON	SCALE: 20:1	

PLATE A

Figure 8.4 Unified Soil Classification System

Guide for
<u>* MOISTURE CONTENTS</u>
ADAPTED TO CEMVN-ED-F SOILS

CLASS	STIFF	MEDIUM	SOFT	V. SOFT	LIQUID LIMIT	PLASTICITY INDEX
CH-4	41-53	43-65	55-80	67-130	70-110	45-75
CH-3	32-43	34-55	44-67	55-114	55-80	30-55
CH-2	27-34	30-44	38-55	48-90	50-60	25-40
CL-6	23-30	25-39	33-48	40-79	40-50	20-35
CL-4	20-25	21-33	27-41	35-67	28-43	10-25
CH-OA	. . . .				110-160	75-97
CH-OB	. . . .				160-185	97-115
CH-OC	. . . .				185-	115-

* For brown or oxidized soils, subtract 10% from the above Moisture Contents.

NOTE: We are using this with the Unified Soil Classification System as a guide and supplementation breakdown for CH's and CL's. We use the CHOA, CHOB and CHOC for organic fat clays in lieu of "OH" and CLOA, CLOB and CLOC for organic lean clays in lieu of OL when used for lean clays. Also, double classes are not used, such as SC-SM or CL-ML. The major class governs and the secondary is recorded as a modification or stratum as appropriate.

Figure 8.5 Unified Soil Classification System Modified for New Orleans Soils

9.0 SURVEYS

9.1 Survey Standards Manual

All surveys shall conform to the latest published version of CEMVN-ED-SS-06-01 "USACE New Orleans District Guide for Minimum Survey Standards." This standard is approved for public release and distribution is unlimited. It is available at:
http://www.mvn.usace.army.mil/ed/edss/surveyingguidelines.asp

9.1.1 Purpose

The document provides guidance on performing detailed engineering surveys of facilities and civil works projects. Technical specifications, procedural guidance, and quality control criteria are outlined for surveying services performed in a consistent manner for the New Orleans District in support of hurricane and flood protection, hydrologic studies, construction, and mapping projects.

9.1.2 Applicability

The document applies to all in-house and A-E contract surveying services having responsibility for the planning, engineering and design, operation, maintenance, construction, and related real estate and regulatory functions of civil works, and environmental restoration projects. It is intended for use by hired-labor personnel, construction contractors, and Architect-Engineer (A-E) contractors. It is also applicable to surveys performed or procured by local interest groups under various cooperative or cost-sharing agreements.

9.1.3 Use of Manual

The Survey Standards document is intended to be a reference guide for control surveying, site plan mapping, utility and infrastructure utility feature mapping. These activities may be performed by hired-labor forces, contracted forces, or combinations thereof.

9.2 Quality Assurance

Survey work shall comply with the following Quality Assurance steps at a minimum. A-Es should reference the MVN Survey Section web page for procedures on contacting MVN for benchmark information and submittal procedures relative to their project. The page is located at: http://www.mvn.usace.army.mil/ed/edss/index.asp

9.2.1 Survey Plan

All A-E contract surveying services shall require a Survey Plan to be submitted to Engineering Division Surveys Section for Independent Technical Review prior to the planned surveying activities. The Survey Plan shall be constructed in accordance with the guidelines established in the "USACE New Orleans District Guide for Minimum Survey Standards." This requirement applies, whether the surveying activity is primary to the contract or task order or incidental to the contract or task order purpose. ITR does not impact mobilization or initiating surveying activities; the parties engaged in data collection remain responsible for appropriate surveying approach and methodologies and as such might be required to provide clarification, adjustments to the methods and data, and recollection.

9.2.2 Survey Report

All A-E contract surveying services shall require a Survey Report to be submitted to Engineering Division Surveys Section for Independent Technical Review within two weeks of completing the surveying activities and office processing. This requirement is independent of any other contractual deadlines. The Survey Report shall be constructed in accordance with the guidelines established in the "USACE New Orleans District Guide for Minimum Survey Standards." This requirement applies, whether the surveying activity is primary to the contract or task order or incidental to the contract or task order purpose. ITR does not impact mobilization or initiating surveying activities; the parties engaged in data collection remain responsible for appropriate surveying approach and methodologies and as such might be required to provide clarification, adjustments to the data, and recollection.

9.2.3 Submittal Format

Both the plan and report shall follow this general outline.

1. Job Number:
2. Contract Number:
3. Lat/Lon:
4. Job Title:
5. General Approach:
6. Horizontal Positioning:
    - 6.1 Datum:
    - 6.2 Control:
    - 6.3 Equipment:
    - 6.4 Methodology:
7. Vertical Positioning:
    - 7.1 Datum:
    - 7.2 Epoch:
    - 7.3 Control:
    - 7.4 Equipment:
    - 7.5 Methodology:

9.3 Adherence to IPET Report Lessons Learned

All A-E contract surveying services shall conform to the following requirements as summarized from the IPET Report, Lessons Learned for Flood Control and Hurricane Protection Projects. All reference datums, surveying methods, benchmarks, and spatial data must be clearly defined and documented. Any questions shall be directed to Engineering Division, Survey Section.

9.3.1 Metadata Embedded Dataset Specification

The metadata embedded dataset specification can be found in Section H at :
http://www.mvn.usace.army.mil/ed/edss/USACE_MVN_Min_Survey_Standards.PDF

9.3.2 Dual Elevations on Flood Control and Hurricane Protection Structures

All planning, design, construction, and operation & maintenance inspection documents containing elevation data on flood control structures should show both geodetic and water surface referenced elevations or at a minimum, show the relationship between the geodetic and water surface or local tidal datum. The relative water surface reference datum (i.e., LMSL) is used as the baseline for hydraulic modeling and related levee height design computations. The terrestrial geodetic datum typically used by surveyors for construction stake out and subsequent periodic subsidence modeling must be corrected to be relative to the local water datum. The base gage with its correction to NAVD88 defining a water level datum must be clearly defined, along with applicable tidal or river stage epochs, and conversion parameters to relate water level datums to the local geodetic datum.

9.3.3 Geospatial Data Source Feature or Metadata Records

All planning, design, and construction documents containing survey information shall contain detailed source (i.e., metadata) information on geospatial coordinates or terrain models included in those documents. This would include the location and repository for the original source data, field book numbers, monument descriptions, etc. Geospatial metadata incorporated in documents shall have sufficient detail such that there is no uncertainty (currently or in the future) as to the location of the original data, its origin, and other temporal relationships.

9.3.4 Epoch Designations of Published Topographic Elevations

Reported elevations of surface topography, subsurface bathymetry, and/or constructed structures in high subsidence areas should contain feature (metadata) information on the source datum and applicable adjustment epoch date. This applies to both geodetic elevations (e.g., 12.34 ft NAVD88 (2004.65)) and water level based elevations (e.g., (-) 5.25 ft LMSL (2000-2005) or 35.0 ft MLLW (1983-2001) or 12.3 ft LWRP (1974)). Hard copy or CADD data files should place this metadata information in the General Notes on the first sheet or digital file of a series, with appropriate references on subsequent sheets/files that depict topographic information and source files names and locations.

9.3.5 Definitions of NGVD29, NAVD88, Mean Sea Level, and Local Mean Sea Level

When referring to the mean water surface at or near a specific flood control project, LMSL should be used. A LMSL derived elevation should clearly identify the water level reference gage location and the time series (epoch) over which the mean surface elevation was computed. NOAA geodetic and tidal datasheets should be modified to clearly indicate orthometric heights/elevations differ from mean sea level elevations.

9.3.6 Coordination of Topographic Survey Data Collection, Processing, and Management

To minimize the confusion associated with several entities producing survey data, all surveys should be coordinated and archived by MVN Survey Section. This would standardize survey methods, survey control, deliverables, etc.

9.3.7 Vertical Control Monumentation Requirements and Stakeout Procedures on Flood Control Construction Projects

A minimum of three (3) permanent benchmarks (new or existing) shall be identified on design and construction drawings for all flood control projects. These marks should be established during the planning and design phase. The marks shall be situated in the middle and at each end of the project. They shall be established relative to existing NAVD88 control established by the NGS, using either conventional differential leveling and/or the latest NGS-approved differential GPS network observations, with appropriate corrections to the local hydraulic design surface. Prior to and during actual construction stake out, these primary reference marks should be verified externally and internally. Field records of these survey verifications shall be permanently archived.

9.3.8 LIDAR and Photogrammetric Mapping Calibration and Testing

Hurricane Protection Projects, requiring accurate, up-to-date topographic detail, should not attempt to utilize older mapping data of uncertain origin, resolution, and accuracy—especially if this data was not reliably quality assured (i.e., ground truthed). Contracts for aerial mapping services must contain quality assurance provisions for calibrating, ground truthing, and testing delivered mapping products. These methods should follow long-established testing methods outlined in standards such as USACE EM 1110-1-1000 (Photogrammetric Mapping), FGDC, ASPRS, and FEMA.

9.4 Typical Scope of Services for Structural Design Projects

The following outline provides the generally required survey information for typical structural design projects. This list is neither definitive nor all-inclusive.

9.4.1 Vertical and Horizontal Control

Horizontal and vertical controls shall be established in accordance with MVN Survey Section Standards. Establish control points and baselines to use as horizontal reference. All horizontal control should be tied to a USACE baseline.

9.4.2 Boundary Surveys

Research adjoining property owners then locate and tie existing property into horizontal control.

Research and locate aboveground and underground utilities and tie them to the horizontal and vertical controls.

Locate required rights-of-way and construction easements.

9.4.3 Topographic Surveys

Identify above ground features such as roads, canals, fences, buildings, bridges, floodwalls, piers, etc. and tie features to vertical and horizontal control points.

If project includes construction in or adjacent to an existing facility, take measurements and spot elevations to verify existing "as-built" drawings and to identify any deviations in the existing structural, architectural or mechanical features. This may also require under water probing.

9.4.4 Cross-Sections and Profiles

9.4.4.1 Major Structure Site

Cross-sections are typically taken at 25 to 50 ft. intervals perpendicular to the baseline. Intervals will depend on site topography and may include or be continuous with hydrographic surveys.

At any intake and discharge areas where hydraulic modeling is to be required, cross-sections should be no further apart than 25 ft.

Extend cross-sections to provide full coverage of area to include important features such as C/L of pavement, edge of roads, waters edge, drainage ditches, top of bank, etc.

A centerline profile is typically required along the proposed project C/L (structure or roadway). The profile should be extended to include important features and may include or be continuous with hydrographic surveys

9.4.4.2 Levee and Floodwall Sites

Cross-sections are typically taken at 50 to 100 ft. intervals perpendicular to the baseline. Intervals depend on site topography and may include or be continuous with hydrographic surveys.

Extend cross-sections to provide full coverage of area to include important features such as C/L of pavement, edge of roads, waters edge, drainage ditches, top of bank, etc.

10.0 CADD STANDARDS

10.1 General

This section provides guidance for creating detailed engineering CADD products for facilities and civil works projects. Technical specifications, procedural guidance, and quality control criteria are outlined for CADD services performed in a consistent manner for the New Orleans District in support of hurricane and flood protection, hydrologic studies, construction, and mapping projects.

10.2 Applicability

This section applies to all in-house and A-E contract services having responsibility for the planning, engineering and design, operation, maintenance, construction, and related real estate and regulatory functions of civil works, and environmental restoration projects. It is intended for use by other USACE FOAs and Architect-Engineer (A-E) contractors supporting MVN, PRO, HPO, and TFH. It is also applicable to CAD products created or procured by local interest groups under various cooperative or cost-sharing agreements.

10.3 CADD Standards

CADD drawings shall be prepared in accordance with the Architectural/Engineering/Construction (A/E/C) Computer-Aided Design and Drafting (CADD) Standard. CADD drawings shall also adhere to the requirements of the MVD CAD Supplement. MVN CAD Standards may also apply. Where standards are in disagreement, MVN standards shall supersede MVD and the A/E/C standards. Further, MVD standards shall supersede the A/E/C standards.

The contractor may also use cell libraries, seed files, border sheets and line style libraries provided by the Government in addition to those required by the Standard.

Standards and files are available at http://www.mvd.usace.army.mil/cad/.

The Contractor shall submit a written request for approval of any deviations from the Government's established CADD standard. No deviations from the government’s established CADD standard will be permitted unless prior written approval of such deviation has been received from the Government.

10.4 Files Names and Drawing Numbers

Files names and drawing numbers for plans and specifications shall be obtained from Denis J. Beer, P.E. at Denis.J.Beer@usace.army.mil.

10.5 Platforms

All CADD data shall be supplied in Bentley Systems, three-dimensional, MicroStation V8 native electronic digital format (i.e., .dgn, .cel), with a Windows XP operating system target platform. The contractor shall ensure that all digital files and data (e.g., model files, reference files, cell libraries) are compatible with the Government's target CADD system (i.e., basic and advanced CADD software, platform, database software), and adhere to the standards and requirements specified herein. The term "compatible" means that data can be accessed directly by the target CADD system without translation, preprocessing, or postprocessing of the electronic digital data files. It is the responsibility of the contractor to ensure this level of compatibility.

If required, the contractor shall also produce drawings and models that are compatible with InRoads, version 8.05 software. As an option, the Contractor may provide InRoads-compatible ASCII random point and break line files for generation of the DTMs, provided that these files produce DTMs that match the topographic/planimetric survey sheets furnished in MicroStation format. DTMs or point files shall be submitted with the drawings on CD-ROM.

Unless noted otherwise, all elements are to be drawn at elevation 0.0 with the active z-depth set to elevation 0.0. Any digital terrain model contours or related 3D interim design elements may keep their proper elevations. All plan view area linework and aerial digital photography shall maintain horizontal control as provided by the government and shall not be moved out of their proper State Plane NAD83 datum position. Each plate shall be in its own individual CADD file and each CADD file shall contain all elements (i.e. there shall not be multiple reference files for a single plate).

Any supplied scanned electronic digital files of georeferenced data shall be delivered in georeferenced TIFF format in the North American Datum 1983 (NAD83), State Plane Coordinate System, and appropriate Zone corresponding to the data location. All other scanned electronic digital files shall be delivered in the native MicroStation digital format which is fully compatible with Bentley System's Descartes 2004 Edition software.

Any nongraphical databases delivered with prepared drawings shall be in Oracle or Microsoft Access compatible format. The database version delivered shall be compatible with the Government's target CADD system. All linkages of nongraphical data with graphic elements, relationships between database tables, and report formats shall be maintained. All database tables shall conform to the structure and field-naming guidance provided by the Government.

10.6 Deliverables

A copy of all CADD data and files developed under this contract shall be delivered to the Government on electronic digital media as an attachment with each submittal as required in the Schedule of Work. The electronic digital media shall be developed and submitted in the Government's target CADD system.

The external label for each electronic digital media shall contain, as a minimum, the following information:

(1) The Contract Number (and Delivery Order Number if applicable) and date.

(2) The format and version of operating system software.

(3) The sequence number of the digital media.

Before a CADD file is placed on the delivery electronic digital media, the following procedures shall be performed:

(1) Remove all extraneous graphics outside the border area.

(2) Remove any unused references from files.

(3) Make sure all reference files are attached without device or directory specifications.

(4) Include all files, both graphic and nongraphic, required for the project (i.e., color tables, pen tables, font libraries, cell libraries, user command files, plot files). All files not provided as Government furnished materials must be provided to the Government as a part of the electronic digital deliverables.

(5) Make sure that all support files such as those listed above are in the same directory and that references to those files do not include device or directory specifications.

(6) Include any standard sheets (i.e., abbreviation sheets, standard symbol sheets) necessary for a complete project.

(7) Document any fonts, tables, etc., developed by the A-E or not provided among the Government furnished materials. The contractor shall obtain Government approval before using anything other than the Government's standard fonts, linetypes, tables, or cells.

(8) Provide in a Microsoft Word file a list of drawing file names.

10.7 Drawing Development Documentation

Complete documentation concerning the development of each finished drawing shall be included in the drawing outside of the border. This documentation shall include the following:

(1) How the data were input (e.g., keyed in, downloaded from a survey total station instrument (include name and model)).

(2) Brief drawing development history (e.g., date started, modification date(s) with brief description of item(s) modified, author's name).

(3) The names of the reference files, cells, symbols, details, tables, and schedule files required for the finished drawing.

(4) Level assignments and lock settings.

(5) Text fonts, line styles used, and pen settings.

10.8 Ownership

The Government, for itself and such others as it deems appropriate, will have unlimited rights under this contract to all information and materials developed under this contract and furnished to the Government and documentation thereof, reports, and listings, and all other items pertaining to the work and services pursuant to this agreement including any copyright. Unlimited rights under this contract are rights to use, duplicate, or disclose text, data, drawings, and information, in whole or in part in any manner and for any purpose whatsoever without compensation to or approval from the Contractor. The Government will at all reasonable times have the right to inspect the work and will have access to and the right to make copies of the above-mentioned items. All text, electronic digital files, data, and other products generated under this contract shall become the property of the Government. By reference, the following DFAR clauses are included in this contract as a part of the requirements herein:

a. DFAR 252.227-7013, “Rights in Technical Data - Noncommercial Items.”

b. DFAR 252.227-7017, “Identification and Assertion of Use, Release, or Disclosure Restrictions.”

c. DFAR 252.227-7020, “Rights in Special Works.”

d. DFAR 252.227-7028, “Technical Data or Computer Software Previously Delivered to the Government.”

e. DFAR 252.227-7037, “Validation of Restrictive Markings on Technical Data.”

f. DFAR 252.227-7025, “Limitations on the Use or Disclosure of Government-Furnished Information Marked with Restrictive Legends.”

g. DFAR 252.227-7014, “Rights in Noncommercial Computer Software and Noncommercial Computer Software Documentation.”

Copyright: Any software and computer data/information developed as a component of this contract shall have the following statement attached to documentation:

“This computer program is a work effort for the United States Government and is not protected by copyright (17 U.S. Code 105). Any person who fraudulently places a copyright notice on, or does any other act contrary to the provisions of 17 U.S. Code 506(c) shall be subject to the penalties provided therein. This notice shall not be altered or removed from this software or digital media, and is to be on all reproductions.”

11.0 SIGNATURES

A sample signature page used by MVN follows.

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

[Project Name]
[Project Location], Louisiana
Solicitation No. XXXXX
File No. XXXXX
Dwgs. X through X

This Project was designed by [the firm of XXXXX or XXXXX district]. The initials or signatures and registration designations are for the New Orleans District of the U.S. Army Corps of Engineers. USACE employees appearing on these project documents are within the scope of their employment as required by ER1110-1-8152.

The following are the official written record of signatures required by ER1110-1-8152 for the above job so as to facilitate electronic bid sets (EBS).

SUBMITTED BY: ________________________________ Date __________
Design Engineer

SUBMITTED BY: ________________________________ Date __________
Functional Team Leader

SUBMITTED BY: ________________________________ Date __________
Chief, Civil Branch

APPROVED BY: ________________________________ Date __________
Chief, Engineering Division

APPROVED BY: ________________________________ Date __________
Colonel, C.E. District Engineer

UPDATED 04 OCT 07

12.0 TYPICAL DRAWINGS AND DETAILS

Safety is a Part of Your Contract

CONCRETE NOTES:

1. CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH (f'c) OF 3000 PSI AT 28 DAYS, 90 DAYS IF POZZOLAN IS USED, UNLESS OTHERWISE NOTED.
2. STABILIZATION SLAB CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH (f'c) OF 2500 PSI AT 28 DAYS, 90 DAYS IF POZZOLAN IS USED.
3. REINFORCING STEEL SHALL HAVE A MINIMUM YIELD STRENGTH (Fy) OF 60,000 PSI.
4. REINFORCING SHALL BE SPACED TO MISS RECESSES FOR GATE LATCHES.
5. CONSTRUCTION JOINTS SHALL BE PROVIDED WHERE SHOWN ON THE DRAWINGS. WHERE NOT SHOWN, CONSTRUCTION JOINTS SHALL BE PLACED AT LOCATIONS LEAST LIKELY TO IMPAIR THE INTERGRITY OF THE CONCRETE STRUCTURE. THESE ADDITIONAL CONSTRUCTION JOINT LOCATIONS SHALL BE APPROVED BY THE CONTRACTING OFFICER.
6. UNLESS OTHERWISE NOTED, PROVIDE 3/4" CHAMFER AT ALL EXPOSED JOINTS, EDGES, EXTERNAL CORNERS, AND VERTICAL EXPANSION JOINTS.
7. RESERVED
8. ALL BENDS OF REINFORCEMENT AND ALL BAR SPACERS AND SUPPORTS SHALL BE IN ACCORDANCE WITH SP-66, AMERICAN CONCRETE INSTITUTE DETAILING MANUAL - 1994.
9. REINFORCING BAR DESIGNATION NUMBERS CONFORM TO THE NUMBERING SYSTEM OF THE CONCRETE REINFORCING STEEL INSTITUTE.
10. REINFORCING BARS SHALL BE CONTINUOUS AT ALL CORNERS UNLESS OTHERWISE NOTED.
11. REINFORCEMENT, WHERE NECESSARY TO AVOID OPENINGS, PIPES, EMBEDDED ITEMS AND OTHER OBSTRUCTIONS, SHALL BE BENT OR SHIFTED AS DIRECTED BY THE CONTRACTING OFFICER.
12. THE EMBEDMENT AND SPLICE TABLE SHALL BE USED IN DETERMINING LAP SPLICES AND EMBEDMENT LENGTHS WHERE LENGTHS ARE NOT OTHERWISE INDICATED. SPLICE LENGTHS SHALL BE BASED ON THE SMALLER BAR BEING LAPPED. THE CONTRACTOR WILL BE ALLOWED TO MAKE SPLICES IN ADDITION TO THOSE INDICATED IN THE DRAWINGS, WHERE ESSENTIAL TO CONSTRUCTIBILITY, SUBJECT TO APPROVAL BY THE CONTRACTING OFFICER. SPLICES OTHER THAN THOSE SHOWN ON THE DRAWINGS AND OTHER THAN ANY ADDITIONAL SPLICES REQUIRED BY THE CONTRACTING OFFICER, WILL BE AT THE CONTRACTOR'S EXPENSE.
13. ALL EXTERIOR FORMED SURFACES NOT COVERED BY BACKFILL SHALL BE CLASS "A" FINISH AND SURFACES COVERED BY BACKFILL SHALL BE CLASS "D" FINISH, UNLESS OTHERWISE NOTED.
14. FOR "T-WALL" CONCRETE PLACEMENT, THE CONTRACTOR SHALL EITHER PLACE A CONSTRUCTION JOINT AT APPROXIMATELY MID-WALL HEIGHT OR SHALL EMPLOY TEMPORARY "WINDOWS" IN THE FORMS TO FACILITATE CONCRETE PLACEMENT AND CONSOLIDATION.

REINFORCEMENT EMBEDMENT AND SPLICE TABLES - 3000 PSI

BAR SIZE	BASIC TABLE				ALTERNATE TABLE			
	MINIMUM EMBEDMENT LENGTH, INCHES		MINIMUM LAP LENGTH INCHES		MINIMUM EMBEDMENT LENGTH, INCHES		MINIMUM LAP LENGTH INCHES	
	TOP	OTHER	TOP	OTHER	TOP	OTHER	TOP	OTHER
3	21	16	28	21	13	12	17	13
4	28	22	37	28	17	13	22	17
5	36	27	46	36	21	16	28	21
6	43	33	56	43	26	20	33	26
7	62	48	81	62	37	29	49	37
8	71	55	93	71	43	33	44	43
9	80	62	104	80	48	37	50	48
10	89	68	116	89	53	41	56	53
11	98	75	127	98	59	45	61	59

REINFORCEMENT EMBEDMENT AND SPLICE TABLES - 4000 PSI

BAR SIZE	BASIC TABLE				ALTERNATE TABLE			
	MINIMUM EMBEDMENT LENGTH, INCHES		MINIMUM LAP LENGTH INCHES		MINIMUM EMBEDMENT LENGTH, INCHES		MINIMUM LAP LENGTH INCHES	
	TOP	OTHER	TOP	OTHER	TOP	OTHER	TOP	OTHER
3	18	14	24	18	12	12	14	12
4	25	19	32	25	15	12	19	15
5	31	24	40	31	18	14	24	18
6	37	28	48	37	22	17	29	22
7	54	42	70	54	32	25	42	32
8	62	47	80	62	37	28	48	37
9	69	53	90	69	42	32	54	42
10	77	59	100	77	46	36	60	46
11	85	65	110	85	51	39	66	51

NOTES:

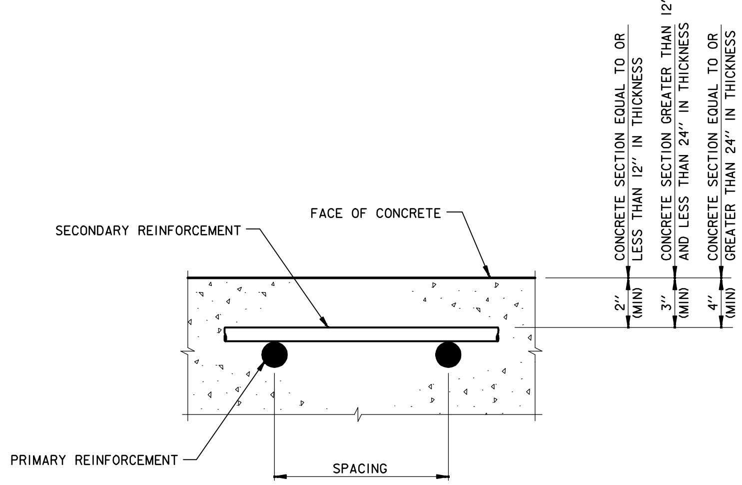
1. USE THE BASIC TABLE IF ALL OF THE FOLLOWING CONDITIONS ARE MET:
   A) CENTER TO CENTER BAR SPACING LATERALLY IS AT LEAST 3 BAR DIAMETERS.
   B) DISTANCE FROM THE CENTER OF A BAR TO THE NEAREST CONCRETE SURFACE MUST BE AT LEAST 2 BAR DIAMETERS.
2. THE ALTERNATE TABLE MAY BE USED IF ALL OF THE FOLLOWING CONDITIONS ARE MET:
   A) CENTER TO CENTER BAR SPACING LATERALLY IS AT LEAST 5 BAR DIAMETERS.
   B) DISTANCE FROM THE CENTER OF A BAR TO THE NEAREST CONCRETE SURFACE MUST BE AT LEAST 2.5 BAR DIAMETERS.
3. IF CONCRETE COVER OR EDGE DISTANCE IS LESS THAN 1 BAR DIAMETERS OR THE CENTER TO CENTER BAR SPACING LATERALLY IS LESS THAN 3 DIAMETERS, SEE ACI 318 FOR APPROPRIATE GUIDANCE.
4. TOP BARS ARE HORIZONTAL BARS AND BARS INCLINED LESS THAN 45 DEGREES WITH RESPECT TO A HORIZONTAL PLANE WHICH ARE PLACED SUCH THAT MORE THAN 12 INCHES OF CONCRETE IS CAST IN THE MEMBER BELOW THE BAR.
5. THE TABLES SHOWN ABOVE ARE FOR NORMAL WEIGHT CONCRETE AND UNCOATED REINFORCING BARS. IF EPOXY COATED BARS ARE USED, SEE ACI 318 FOR ADDITIONAL CONSIDERATIONS.

GENERAL NOTES:

1. AZIMUTHS SHOWN ARE MEASURED CLOCKWISE FROM THE NORTH.
2. ELEVATIONS ARE IN FEET AND REFER TO NAVD88 (2004.65).
3. DIMENSIONS AND/OR ELEVATIONS MARKED THUS (±) ARE APPROXIMATE. CONTRACTOR SHALL VERIFY ACTUAL DIMENSIONS IN THE FIELD.
4. DIMENSIONS AND/OR ELEVATIONS MARKED THUS (N.T.S.) ARE NOT SHOWN TO SCALE.
5. DRAWINGS ARE GENERALLY TO SCALE, BUT SHOULD NOT BE SCALED. N.T.S. IS SHOWN ONLY WHERE DRAWING IS OBVIOUSLY OUT OF SCALE.
6. BENCH MARKS AND BASE LINES HAVE BEEN ESTABLISHED AT THE SITE BY THE GOVERNMENT.
7. FOR BORING LOGS, SEE DWGS. XX-XX.

STEEL NOTES:

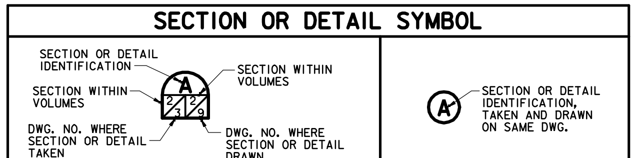
1. ALL STRUCTURAL STEEL SHALL BE ASTM A36, UNLESS OTHERWISE NOTED.
2. TO PREVENT CORROSION BY MOISTURE BETWEEN STEEL SURFACES IN CONTACT, ALL SUCH CONTACTS SHALL BE SEALED WATERTIGHT BY RUNNING A CONTINUOUS 1/4" FILLET WELD ALONG ALL EDGES OF THE CONTACT, UNLESS OTHERWISE NOTED.
3. ALL WELDING SHALL BE ELECTRIC WELDING. WORKMANSHIP AND TECHNIQUE, WHERE APPLICABLE, SHALL CONFORM TO THE AMERICAN WELDING SOCIETY STRUCTURAL WELDING CODE, SEE SPECIFICATIONS.
4. WELDING SYMBOLS SHOWN ARE THOSE ADOPTED BY THE AMERICAN WELDING SOCIETY AND INDICATE ONLY SIZE AND TYPE OF WELDS REQUIRED. DETAILED INFORMATION SHALL BE SHOWN ON THE SHOP DRAWINGS AND SUBMITTED BY THE CONTRACTOR FOR APPROVAL.
5. DIMENSIONS SHOWN OR CALLED FOR ARE THE FINAL DIMENSIONS; ALLOWANCES MUST BE MADE FOR MACHINING.
6. ITEMS MARKED C.R.S. SHALL BE CORROSION RESISTANT STEEL (STAINLESS STEEL), SEE SPECIFICATIONS.



REINFORCEMENT CLEARANCE DETAIL

NOT TO SCALE

SECTION OR DETAIL SYMBOL



ABBREVIATIONS

ALT. SP.	= ALTERNATE SPACING
AZ	= AZIMUTH
B/L	= BASELINE
BF	= BOTTOM FACE
BL	= BOTTOM LAYER
C	= CENTER
CB	= CATCH BASIN
C.I.	= CAST IRON
CJ	= CONSTRUCTION JOINT
CL	= CLEAR COVER
C/L OR ℄	= CENTER LINE
C.R.S.	= CORROSION RESISTANT STEEL
Ø	= DIAMETER
D	= DRAIN
D.I.	= DROP INLET
D.P.	= DRAIN PIPE
D/S	= DOWN STREAM
D.V.	= DRAIN VALVE
D.V. MH.	= DRAIN VALVE MANHOLE
E	= ELECTRICAL
EF	= EACH FACE
EL.	= ELEVATION
ES	= EQUALLY SPACED
F.H.	= FIRE HYDRANT
FF	= FAR FACE
FS	= FAR SIDE
G	= GAS
H.S.	= HIGH STRENGTH
LP	= LIGHT POLE
LS	= LIGHT STANDARD
MH	= MANHOLE
NF	= NEAR FACE
NS	= NEAR SIDE
N.T.S.	= NOT TO SCALE
O.C.	= ON CENTER
OPT.	= OPTIONAL
OS	= OFFSET
P	= POWER
P.C.	= POINT OF CURVATURE
P.T.	= POINT OF TANGENCY
S	= SEWER
SB/L	= SUBBASELINE
S.C.O.	= SEWER CLEANOUT
STD. HK.	= STANDARD HOOK
STA.	= STATION
T	= TELEPHONE
TD	= TRENCH DRAIN
TF	= TOP FACE
TEL.M.H.	= TELEPHONE MANHOLE
TL	= TOP LAYER
TP	= TEST PILE
U.O.N.	= UNLESS OTHERWISE NOTED
U/S	= UP STREAM
W	= WATER
W/L	= WALL LINE
W.M.	= WATER METER
W.V.	= WATER VALVE

US Army Corps of Engineers New Orleans District

MARK | DESCRIPTION | DATE | APPR.

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

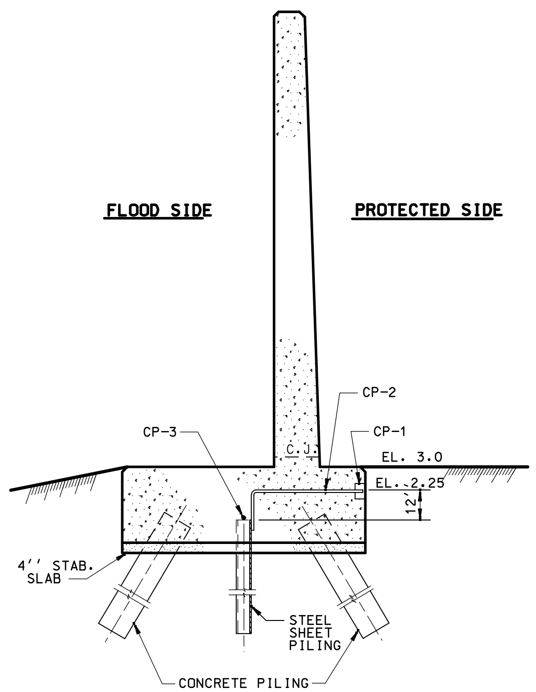
DESIGNED BY:
CHECKED BY:
DRAWN BY:
DATE: 1 OCT 07
PLOT SCALE: 1
PLOT DATE:
DESIGN FILE NAME: AETYP12A.DGN
SUBMITTED BY: DESIGN ENGINEER
SOLICITATION NO. DACW29-

GENERAL NOTES

FILE NUMBER

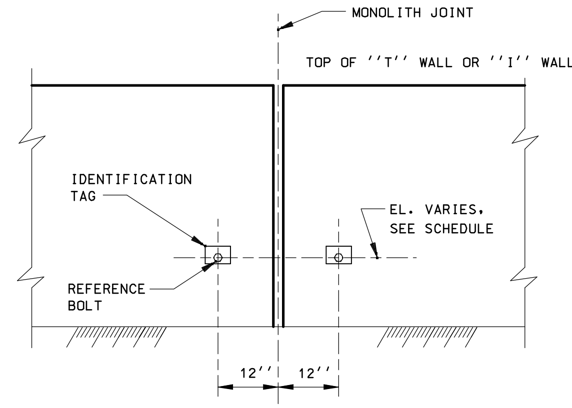
DWG. OF

Safety is a Part of Your Contract



CORROSION PROTECTION DETAIL
T-TYPE WALL

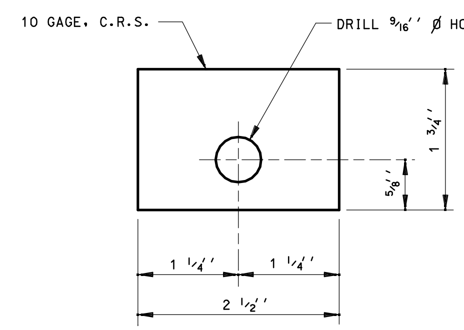
SCALE: 3/8'' = 1'- 0''



PROTECTED SIDE

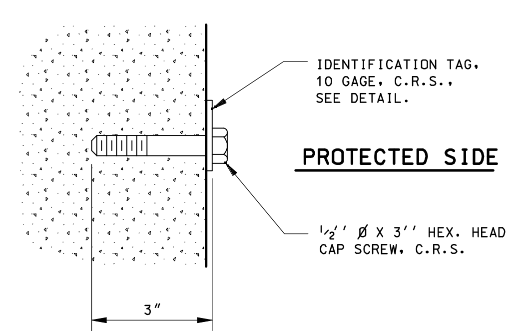
TYPICAL ELEVATION OF SETTLEMENT REFERENCE MARKER

SCALE: 3/4'' = 1'- 0''



IDENTIFICATION TAG

SCALE: 12'' = 1' - 0''



REFERENCE BOLT

SCALE: 6'' = 1' - 0''

LEGEND

CP-1 6'' STANDARD IRON BODY FERRULE WITH BRASS SCREW PLUG. FOR LOCATION OF FERRULES, SEE FERRULE SCHEDULE.

CP-2 #6 REINFORCING BAR WELDED TO SHEET PILE AND TERMINATED IN THE FERRULE WITHIN 1/2'' OF THE COVER.

CP-3 #6 REINFORCING BAR TO BE WELDED TO THE TOP OF EACH STEEL SHEET PILE. #6 REINFORCING BAR SHALL NOT EXTEND ACROSS THE MONOLITH JOINT. INSTALL BOND CABLE AT ALL T-WALL AND I-WALL JOINTS AND AT ALL TRANSITIONS FROM T-WALL TO I-WALL JOINTS. BOND CABLE SHALL HAVE AN 8'' DIAMETER LOOP TO ALLOW FOR STRESSES. BOND CABLES SHALL BE WELDED AS SPECIFIED TO ADJACENT STEEL PILES 12'' BELOW THE BOTTOM OF BASE SLAB FOR T-WALL JOINTS, 7'' BELOW BOTTOM OF CONCRETE CAP FOR I-WALL JOINTS AND AT TRANSITIONS FROM T-WALL TO I-WALL JOINTS. WELDED CONNECTIONS SHALL BE COATED WITH SPLICING EPOXY TO OBTAIN MOISTURE PROOF JOINT. SEE SPECIFICATIONS.

FERRULE SCHEDULE

W/L STATION	W/L STATION	W/L STATION

SETTLEMENT REFERENCE MARKER SCHEDULE

S.R.M. NO.	W/L STATION * (C/L MONOLITH JOINT)	ELEVATION **

* W/L STATIONS ARE APPROXIMATE. LOCATE REFERENCE BOLTS AT NEAREST W/L STATION TO THOSE SHOWN.

** NOTE: THE CONTRACTOR SHALL TAKE FINAL ELEVATIONS OF ALL SETTLEMENT REFERENCE MARKERS AND SHALL SUBMIT THIS DATA TO THE CONTRACTING OFFICER REPRESENTATIVE (COR). THE COR WILL FURNISH THIS DATA TO ENGINEERING DIVISION, ATTENTION OF: CELMN-ED-T.

NOTES

1. FOR GENERAL NOTES, SEE DWG. XX.

SCALE: 3/8" = 1'- 0"

SCALE: 3/4'' = 1'- 0''

SCALE: 6'' = 1' - 0''

SCALE: 12'' = 1' - 0''

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

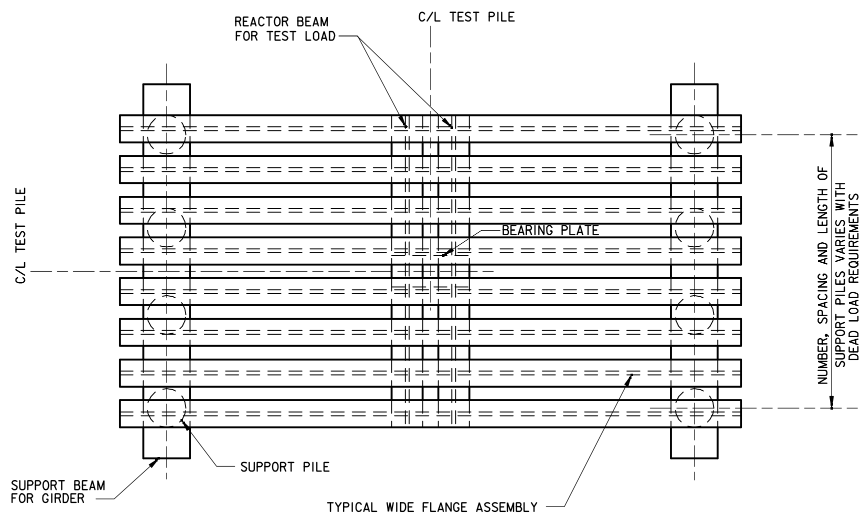
DESIGNED BY: CHECKED BY: DRAWN BY: DATE: 01 OCT 07

MISCELLANEOUS DETAILS

FILE NUMBER

DWG. OF

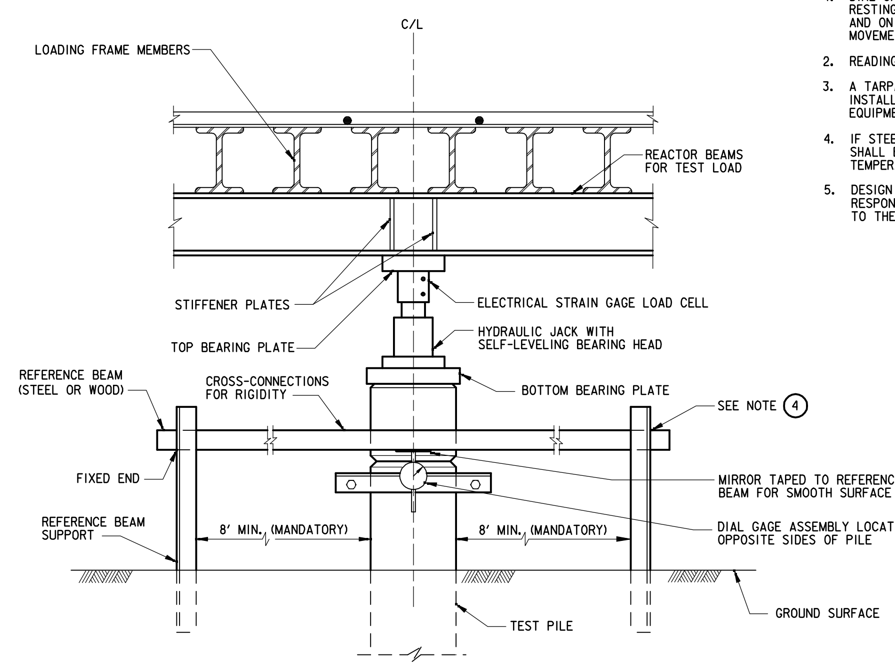
Safety is a Part of Your Contract



PLAN

MANDATORY LOADING FRAME NOTES:

1. LOADING FRAME SHOWN WITHOUT DEAD LOAD.
2. CONTRACTOR TO PROVIDE PILE LAYOUT FOR EACH PARTICULAR DEAD LOAD TEST.
3. SECURE DEAD WEIGHT LOAD TO LOADING FRAME WITH CHAINS AND BINDERS.
4. DESIGN OF THE LOADING FRAME IS THE RESPONSIBILITY OF THE CONTRACTOR AND MUST BE SUBMITTED TO THE CONTRACTING OFFICER FOR APPROVAL.



SECTION B
TEST APPARATUS

SCALE: 1" = 1'- 0"

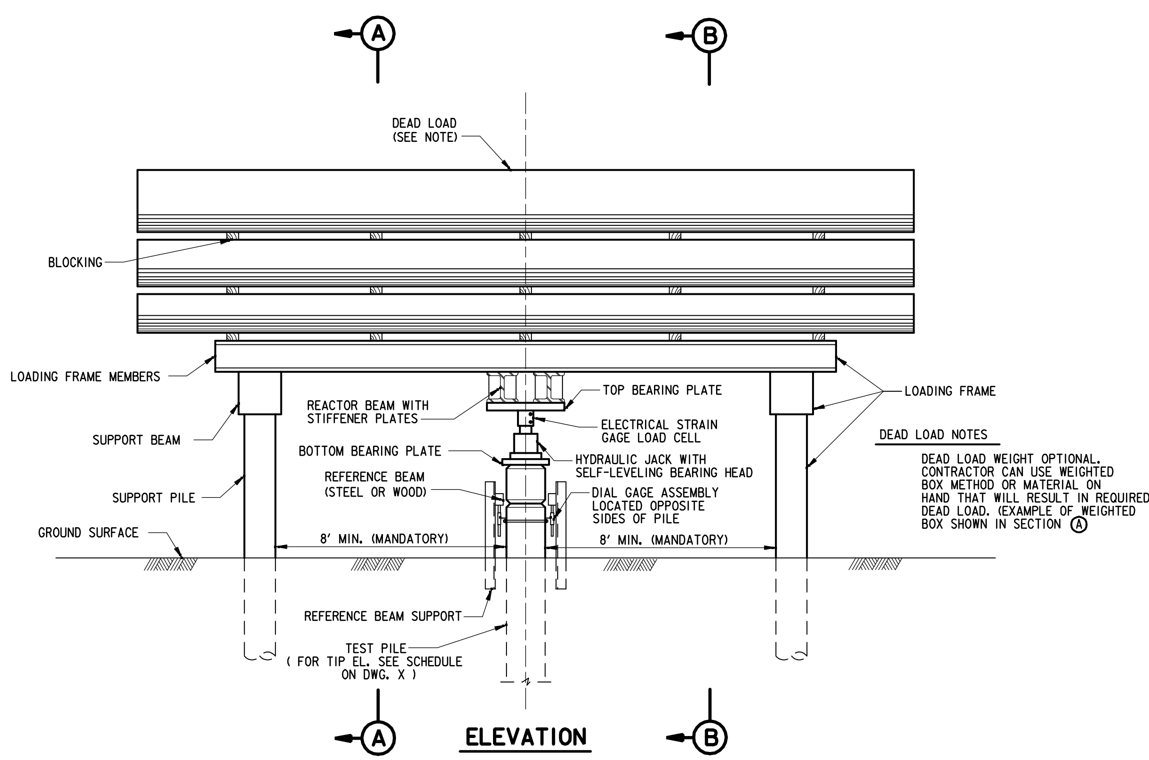
MANDATORY TEST APPARATUS NOTES:

1. DIAL GAGE SHOULD BE ATTACHED TO THE PILE WITH THE STEM RESTING ON THE REFERENCE BEAM IN THE COMPRESSED POSITION AND ON THAT SIDE OF THE REFERENCE BEAM WHERE THE MOVEMENT WILL BE AWAY FROM THE BEAM.
2. READINGS ON THE OPPOSITE SIDES OF THE PILE ARE NECESSARY.
3. A TARPAULIN OF MINIMUM DIMENSION 12' X 12' SHALL BE INSTALLED BY THE CONTRACTOR TO PROTECT THE MEASURING EQUIPMENT FROM THE DIRECT EFFECTS OF THE WEATHER.
4. IF STEEL REFERENCE BEAMS ARE USED, ONE END OF EACH BEAM SHALL BE FREE TO MOVE AS THE LENGTH OF BEAMS CHANGE WITH TEMPERATURE VARIATIONS.
5. DESIGN OF THE TEST APPARATUS PILE SET-UP IS THE RESPONSIBILITY OF THE CONTRACTOR AND MUST BE SUBMITTED TO THE CONTRACTING OFFICER FOR APPROVAL.

NOTE:

THE CONTRACTOR HAS THE OPTION OF USING REACTION PILES IN LIEU OF THE LOADING FRAME. (SEE SPECFICATIONS)

THE LOADING FRAME AND TEST APPARATUS IS FOR ILLUSTRATION PURPOSES ONLY, UNLESS NOTED MANDATORY.

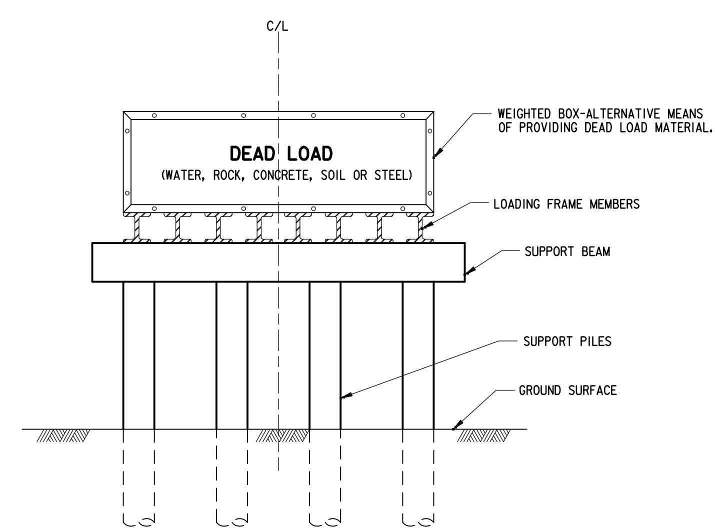


DEAD LOAD NOTES

DEAD LOAD WEIGHT OPTIONAL. CONTRACTOR CAN USE WEIGHTED BOX METHOD OR MATERIAL ON HAND THAT WILL RESULT IN REQUIRED DEAD LOAD. (EXAMPLE OF WEIGHTED BOX SHOWN IN SECTION A)

ELEVATION
LOADING FRAME

SCALE: 1/2" = 1'- 0"



SECTION A

SCALE: 1/2" = 1'- 0"

SCALE: 1/2" = 1'- 0"

0 2' 4' 6' 8'

SCALE: 1" = 1'- 0"

12" 0 1' 2' 3' 4' 5'

US Army Corps of Engineers New Orleans District

MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:
CHECKED BY:
DRAWN BY:
DATE: 1 OCT 07

PLOT SCALE: 12

PLOT DATE:

DESIGN FILE NAME: AETYP15.DGN

SUBMITTED BY:

SOLICITATION NO. DACW29-

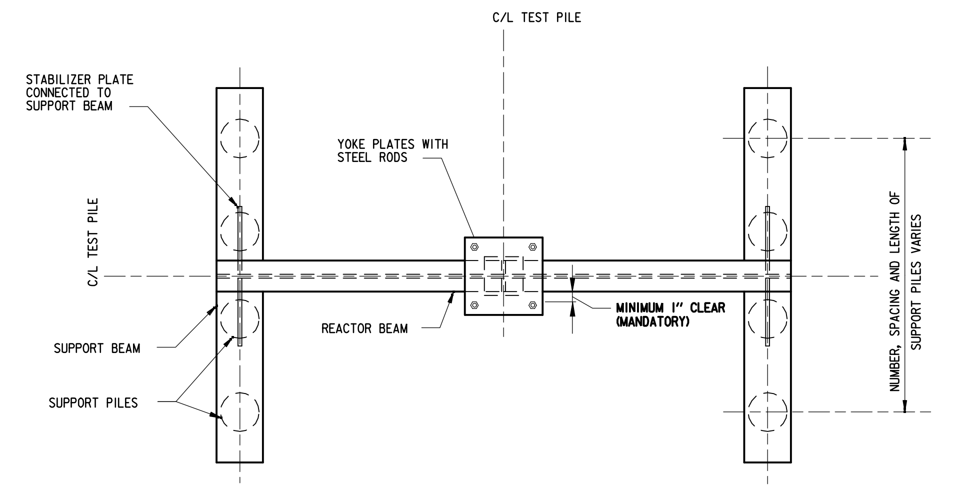
COMPRESSION PILE TEST

FILE NUMBER

DWG. OF

12-3

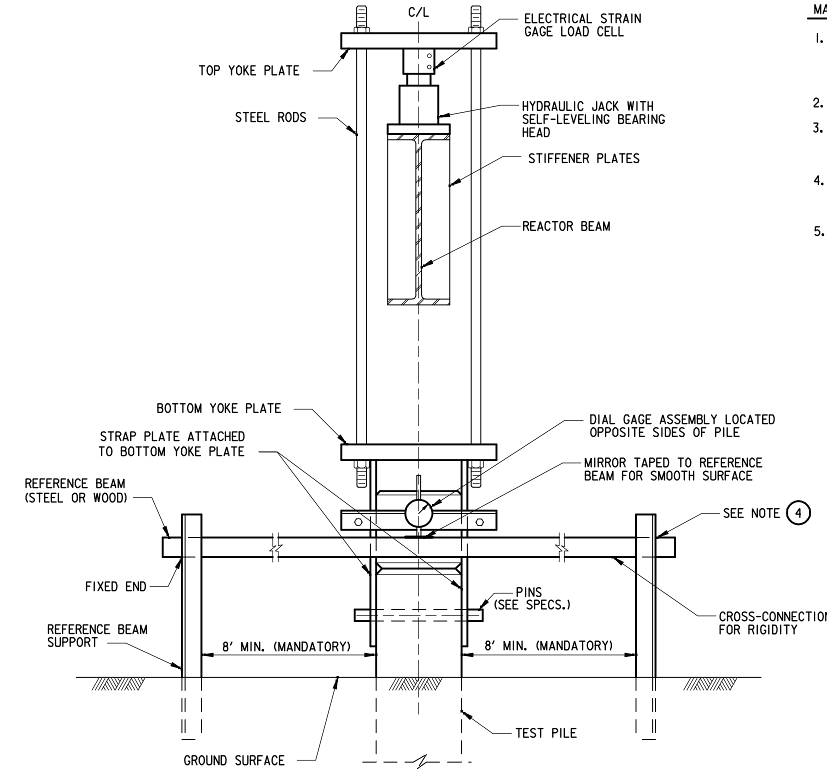
Safety is a Part of Your Contract



MANDATORY LOADING FRAME NOTE:

1. DESIGN OF THE LOADING FRAME IS THE RESPONSIBILITY OF THE CONTRACTOR AND MUST BE SUBMITTED TO THE CONTRACTING OFFICER FOR APPROVAL.

PLAN



SECTION B

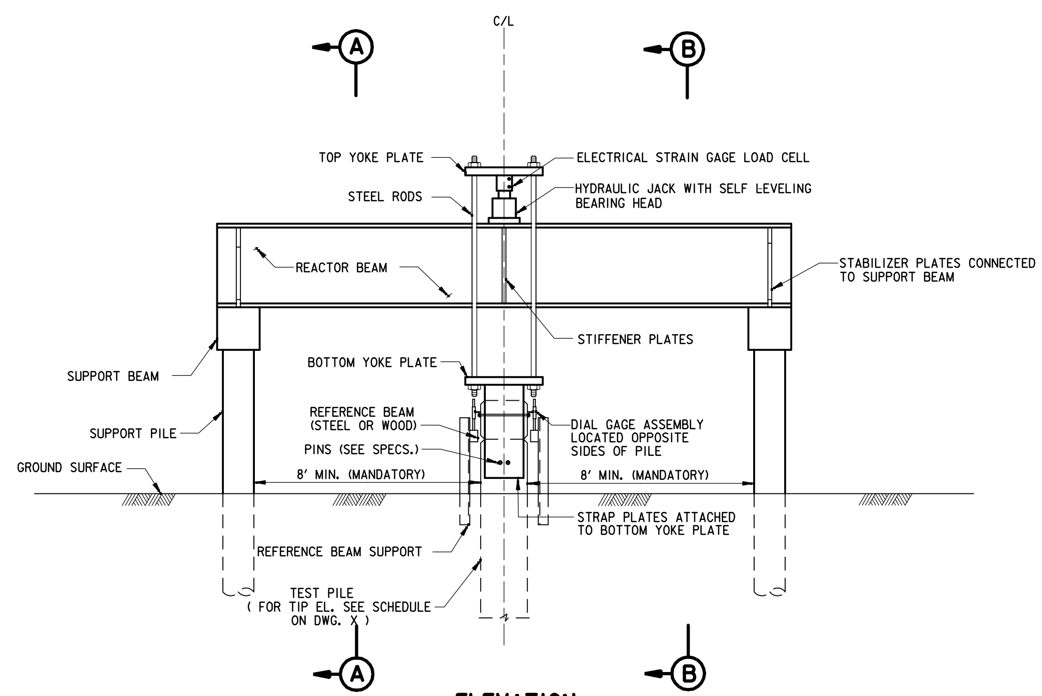
TEST APPARATUS

SCALE: 1" = 1'- 0"

MANDATORY TEST APPARATUS NOTES:

1. DIAL GAGE SHOULD BE ATTACHED TO THE PILE WITH THE STEM RESTING ON THE REFERENCE BEAM IN THE COMPRESSED POSITION AND ON THAT SIDE OF THE REFERENCE BEAM WHERE THE MOVEMENT WILL BE AWAY FROM THE BEAM.
2. READINGS ON THE OPPOSITE SIDES OF THE PILE ARE NECESSARY.
3. A TARPAULIN OF MINIMUM DIMENSION 12' X 12' SHALL BE INSTALLED BY THE CONTRACTOR TO PROTECT THE MEASURING EQUIPMENT FROM THE DIRECT EFFECTS OF THE WEATHER.
4. IF STEEL REFERENCE BEAMS ARE USED, ONE END OF EACH BEAM SHALL BE FREE TO MOVE AS THE LENGTH OF BEAMS CHANGE WITH TEMPERATURE VARIATIONS.
5. DESIGN OF THE TEST APPARATUS PILE SET-UP IS THE RESPONSIBILITY OF THE CONTRACTOR AND MUST BE SUBMITTED TO THE CONTRACTING OFFICER FOR APPROVAL.

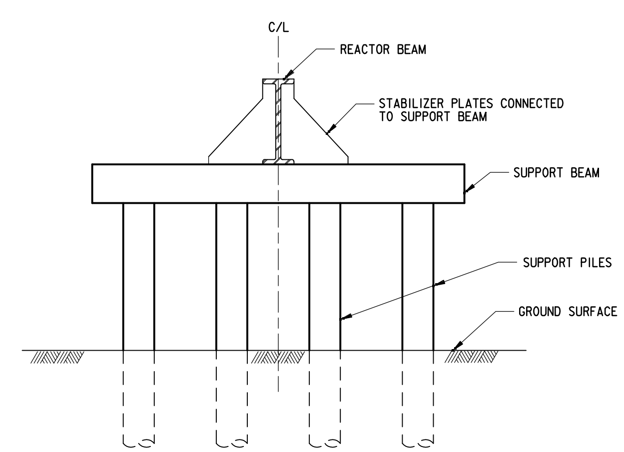
THE LOADING FRAME AND TEST APPARATUS IS FOR ILLUSTRATION PURPOSES ONLY, UNLESS NOTED MANDATORY.



ELEVATION

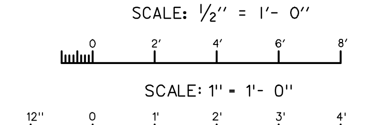
LOADING FRAME

SCALE: 1/2" = 1'- 0"



SECTION A

SCALE: 1/2" = 1'- 0"



US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:
CHECKED BY:
DRAWN BY:
DATE: 1 OCT 07
PLOT SCALE: 12
PLOT DATE:
DESIGN FILE NAME: AETYP16.DGN
SUBMITTED BY:
SOLICITATION NO. DACW29-

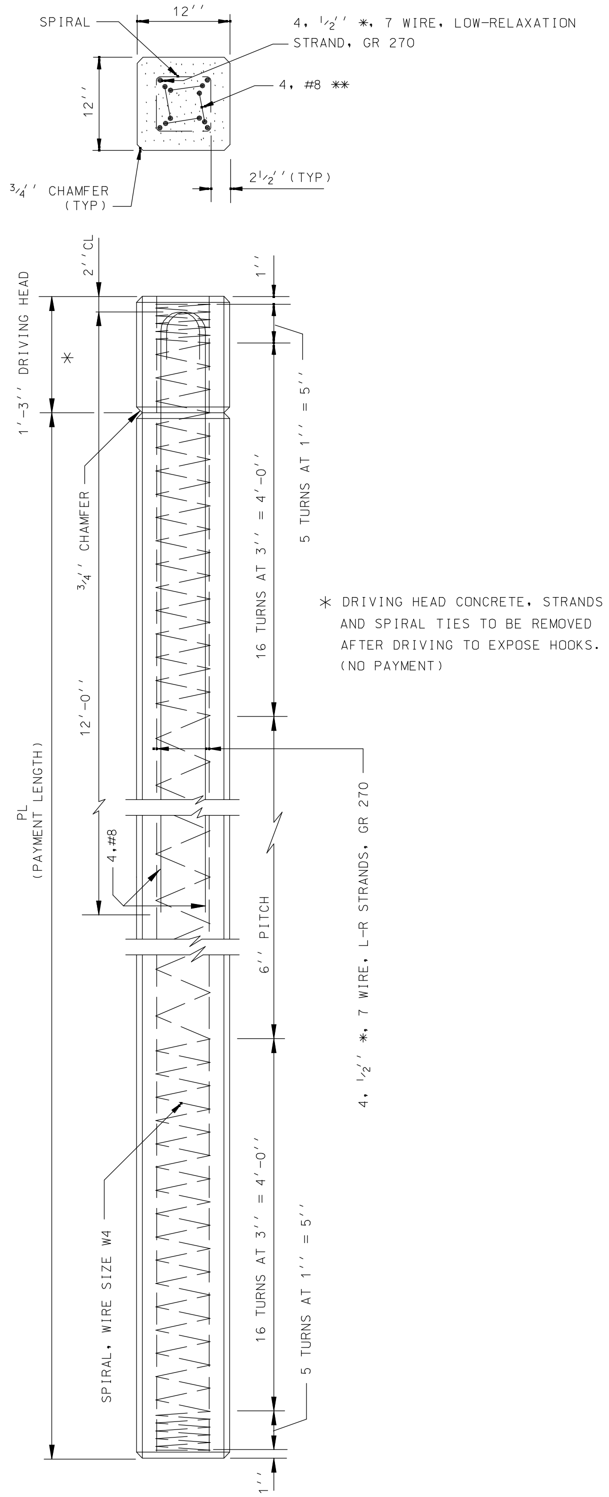
TENSION PILE TEST

FILE NUMBER

DWG. OF

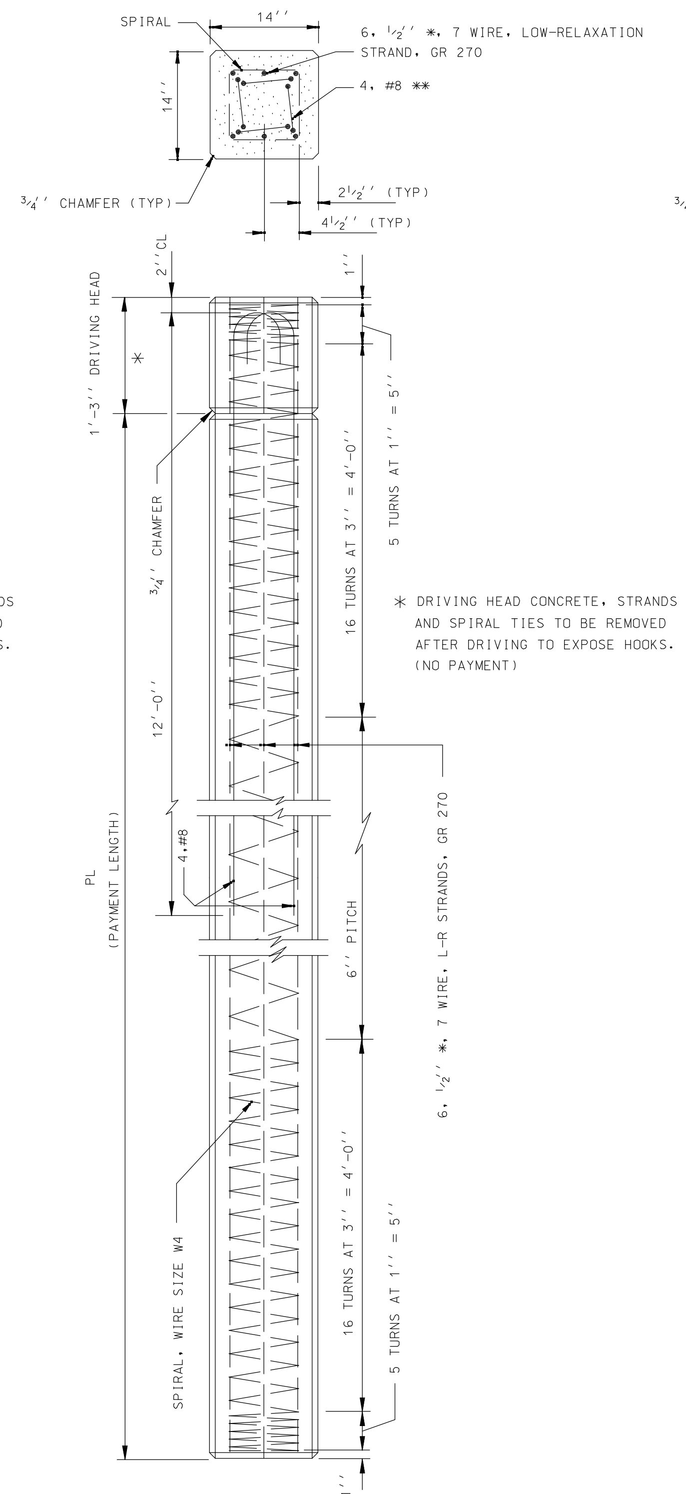
Safety is a Part of Your Contract

** STANDARD 180° HOOK ON REINFORCING FOR ALL PILES. TO ALLOW FOR PROPER PLACEMENT OF CONCRETE, HOOKS SHOULD BE PLACED SO THAT BEND OF HOOK IS APPROXIMATELY PARALLEL TO ADJACENT FACE OF PILE AS SHOWN.



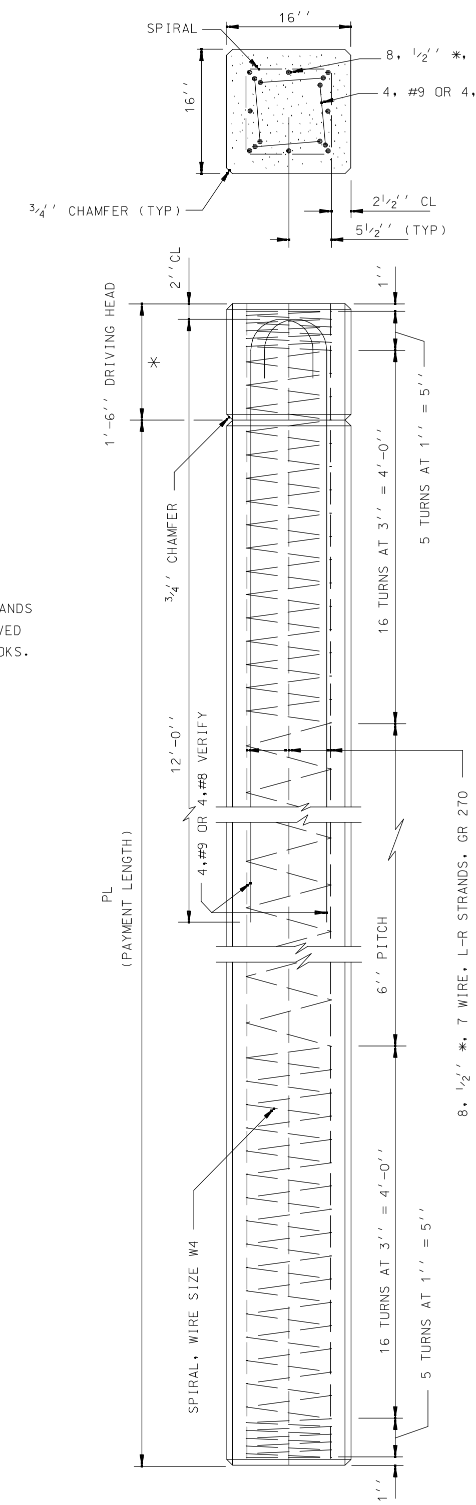
NOTE: GRIND PRESTRESSED STRANDS FLUSH WITH PILE HEAD AND PILE TIP.

12'' X 12''
PRESTRESSED PRECAST CONCRETE PILE



NOTE: GRIND PRESTRESSED STRANDS FLUSH WITH PILE HEAD AND PILE TIP.

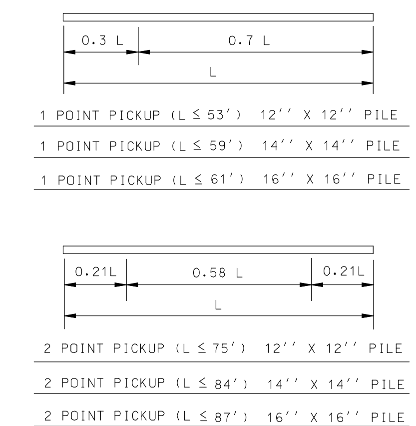
14'' X 14''
PRESTRESSED PRECAST CONCRETE PILE



NOTE: GRIND PRESTRESSED STRANDS FLUSH WITH PILE HEAD AND PILE TIP.

16'' X 16''
PRESTRESSED PRECAST CONCRETE PILE

* DRIVING HEAD CONCRETE, STRANDS AND SPIRAL TIES TO BE REMOVED AFTER DRIVING TO EXPOSE HOOKS. (NO PAYMENT)



NOTES: 1. PICKUP POINTS TO BE PLAINLY MARKED ON PILES
2. PICKUP POINTS SHOWN FOR 5,000 PSI CONCRETE ONLY

PILE SCHEDULE

W/L STATIONS	PILE SIZE	NUMBER OF PILES	PILE BATTER	PILE TIP ELEV.	PAYMENT LENGTH FLOOD SIDE	PAYMENT LENGTH PROTECT SIDE

NOTE: EXACT LENGTHS OF PRESTRESSED CONCRETE PILING SHALL BE DETERMINED AFTER PILE TESTS ARE CONDUCTED. LENGTHS SHOWN ARE FOR ESTIMATING PURPOSES ONLY.

NOTES

1. FOR GENERAL NOTES, SEE DWG. XX.
2. ONLY 5,000 OR 6,000 PSI CONCRETE SHALL BE USED FOR PRESTRESS CONCRETE PILES.

REFERENCE DRAWINGS

1. PILING AND MONOLITH LAYOUT, DWGS. XX AND XX.
2. PILE TEST SITE DETAILS, DWG. XX.

SCALE: 1 1/2'' = 1' - 0''

12'' 0 1' 2' 3'

US Army Corps of Engineers New Orleans District

MARK	DESCRIPTION	DATE	APPR.

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: CHECKED BY: DRAWN BY: DATE: 1 OCT 07
PLOT SCALE: PLOT DATE:
DESIGN FILE NAME: AETYP13.DGN
SUBMITTED BY: DESIGN ENGINEER
SOLICITATION NO. DACW29-

CONCRETE PILE DETAILS
LOW-RELAXATION STRANDS

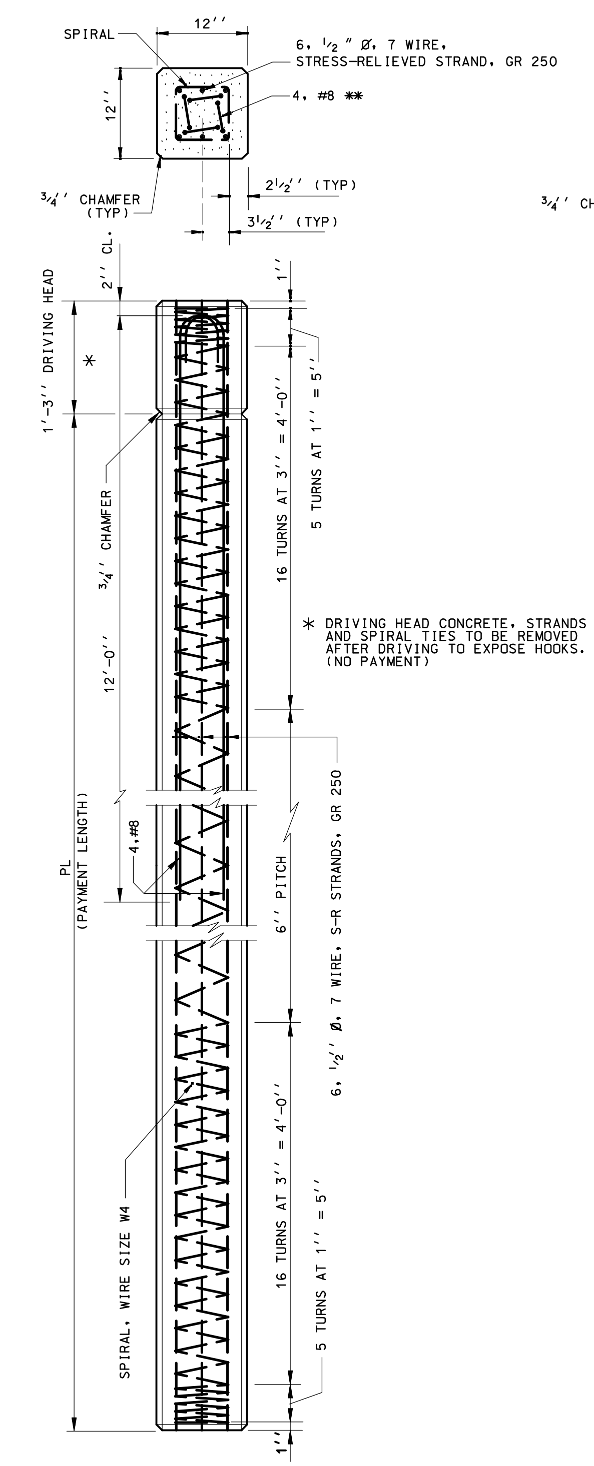
FILE NUMBER

DWG. OF

VALUE ENGINEERING — YOUR KEY TO HIGHER PROFITS

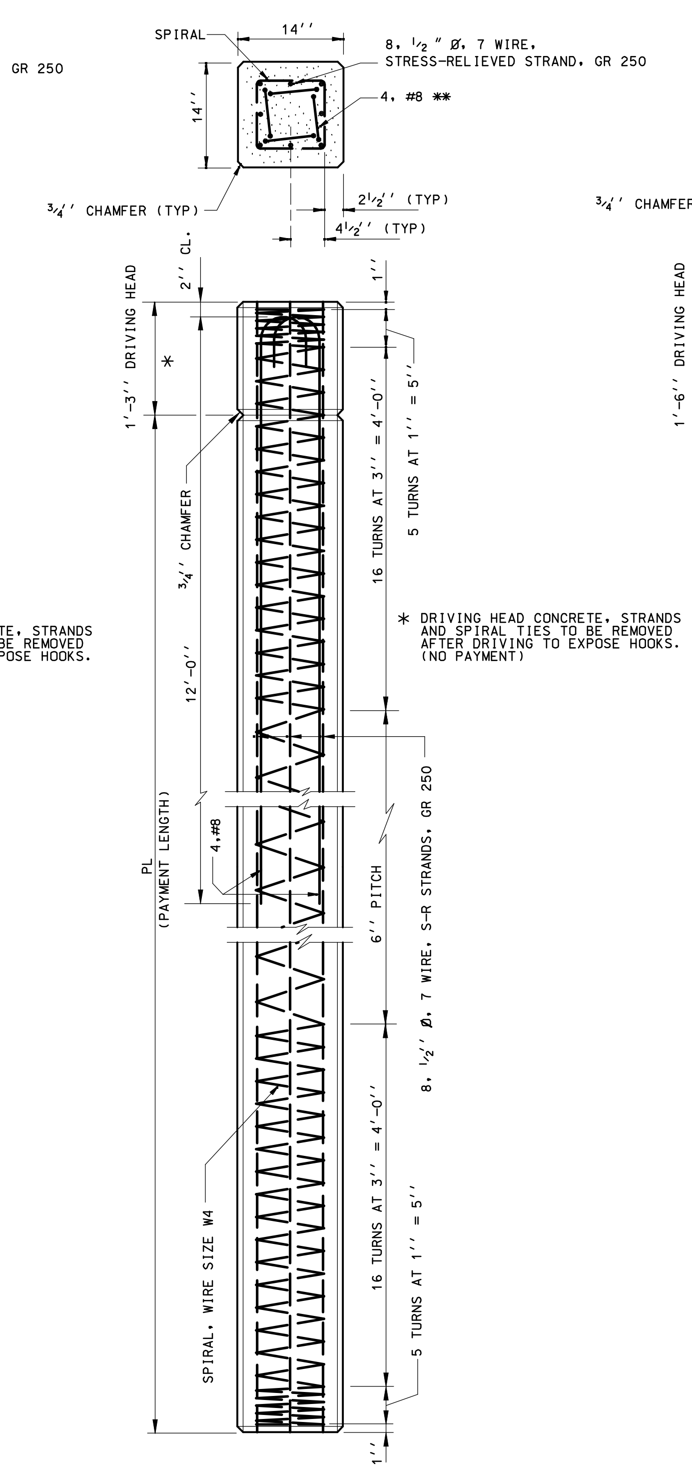
Safety is a Part of Your Contract

** STANDARD 180° HOOK ON REINFORCING FOR ALL PILES. TO ALLOW FOR PROPER PLACEMENT OF CONCRETE, HOOKS SHOULD BE PLACED SO THAT BEND OF HOOK IS APPROXIMATELY PARALLEL TO ADJACENT FACE OF PILE AS SHOWN.



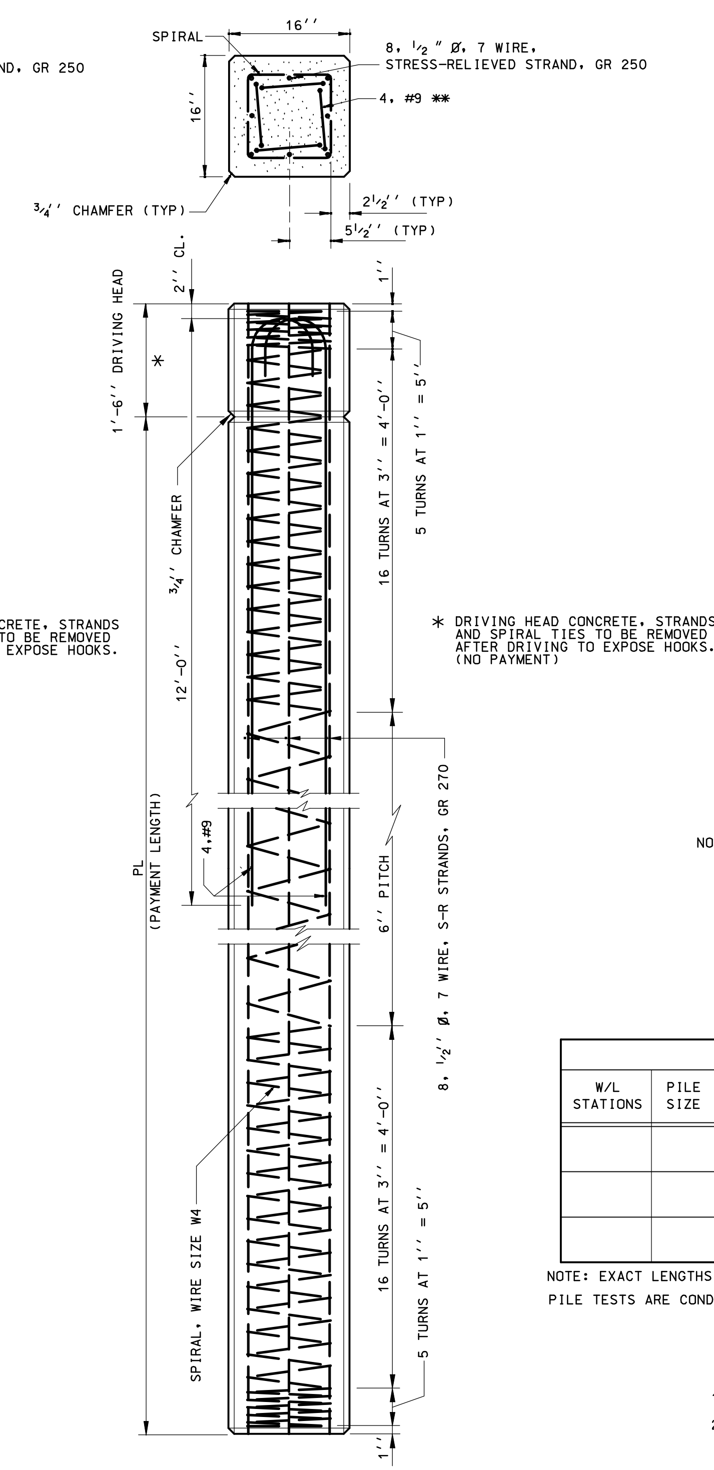
NOTE: GRIND PRESTRESSED STRANDS FLUSH WITH PILE HEAD AND PILE TIP.

12'' X 12''
PRESTRESSED PRECAST CONCRETE PILE



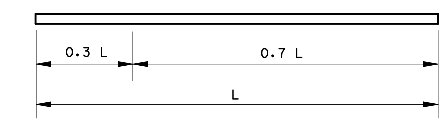
NOTE: GRIND PRESTRESSED STRANDS FLUSH WITH PILE HEAD AND PILE TIP.

14'' X 14''
PRESTRESSED PRECAST CONCRETE PILE

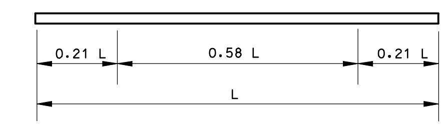


NOTE: GRIND PRESTRESSED STRANDS FLUSH WITH PILE HEAD AND PILE TIP.

16'' X 16''
PRESTRESSED PRECAST CONCRETE PILE



1 POINT PICKUP (L ≤ 54') 12'' X 12'' PILE
1 POINT PICKUP (L ≤ 58') 14'' X 14'' PILE
1 POINT PICKUP (L ≤ 60') 16'' X 16'' PILE



2 POINT PICKUP (L ≤ 78') 12'' X 12'' PILE
2 POINT PICKUP (L ≤ 83') 14'' X 14'' PILE
2 POINT PICKUP (L ≤ 86') 16'' X 16'' PILE

NOTES:
1. PICKUP POINTS TO BE PLAINLY MARKED ON PILES
2. PICKUP POINTS SHOWN FOR 5,000 PSI CONCRETE ONLY

PILE SCHEDULE

W/L STATIONS	PILE SIZE	NUMBER OF PILES	PILE BATTER	PILE TIP ELEVATION	PAYMENT LENGTH	
					FLOOD SIDE	PROTECTED SIDE

NOTE: EXACT LENGTHS OF PRESTRESSED CONCRETE PILING SHALL BE DETERMINED AFTER PILE TESTS ARE CONDUCTED. LENGTHS SHOWN ARE FOR ESTIMATING PURPOSES ONLY.

NOTES

1. FOR GENERAL NOTES, SEE DWG. XX.
2. ONLY 5,000 OR 6,000 PSI CONCRETE SHALL BE USED FOR PRESTRESS CONCRETE PILES.

REFERENCE DRAWINGS

1. PILING AND MONOLITH LAYOUT, DWGS. XX AND XX.
2. PILE TEST SITE DETAILS, DWG. XX.

US Army Corps of Engineers New Orleans District

MARK | DATE | APPR. | DESCRIPTION

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY: CHECKED BY: DRAWN BY: DATE: 1 Oct 07
PLOT SCALE: 1
PLOT DATE:
DESIGN FILE NAME: AETYP02.DGN
SUBMITTED BY:
SOLICITATION NO.

CONCRETE PILE DETAILS STRESS-RELIEVED STRANDS

FILE NUMBER
DWG. OF

12-6

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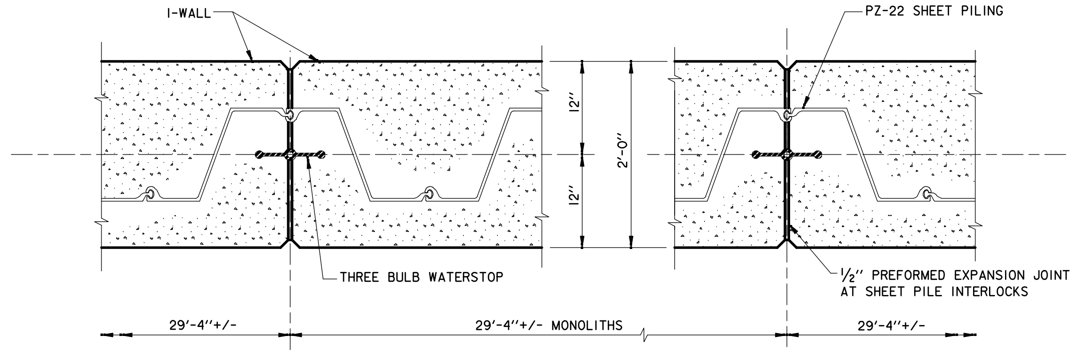
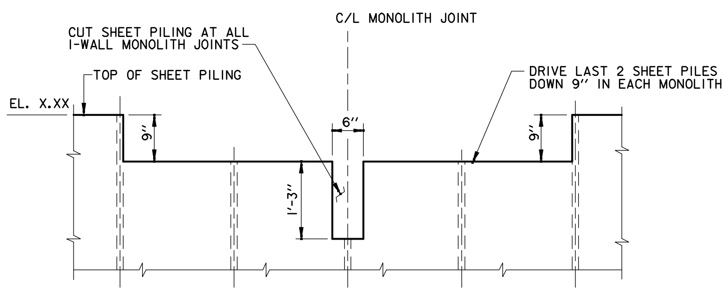


TYPICAL MONOLITH AT SHEET PILE INTERLOCKS

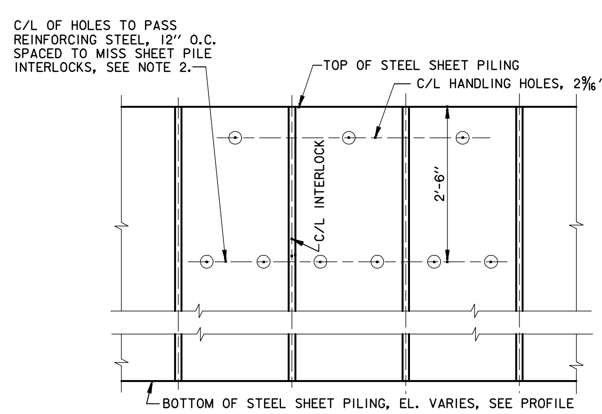
SCALE: 1 1/2" = 1'- 0"

NOTE:
I-WALL MONOLITHS SHALL BE 29'-4"+/- UNLESS OTHERWISE INDICATED ON THE PROFILE. EACH MONOLITH SHALL END AT THE CENTER OF THE NEAREST SHEET PILE INTERLOCK.



SHEET PILING DETAILS
I-WALL MONOLITH JOINTS

SCALE: 1" = 1'- 0"



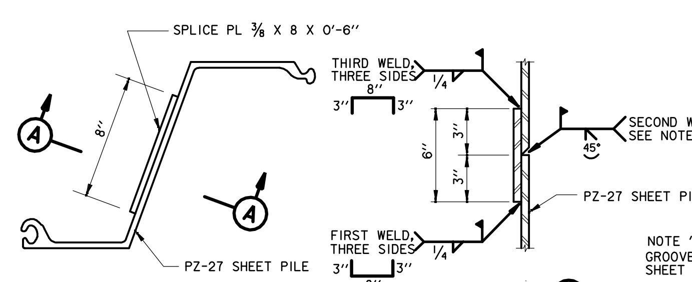
ELEVATION

DETAILS OF HOLES IN SHEET PILING

SCALE: 1" = 1'- 0"

SHEET PILE NOTES

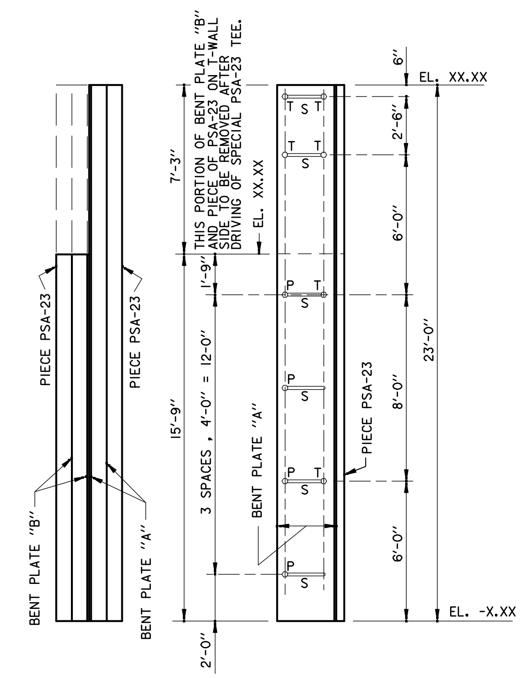
1. A MINIMUM OF 6 INCHES OF CONCRETE COVER SHALL BE PROVIDED OVER SHEET PILING AT ALL POINTS.
2. HOLES CUT IN STEEL SHEET PILING FOR PASSING REINFORCING BARS SHALL NOT EXCEED 2"ø. WHERE HOLES FALL WITHIN THE WEB OF THE STEEL SHEET PILE, THE HOLE SHALL BE SLOTTED 4" HORIZONTALLY TO ACCOMMODATE PASSING THE REINFORCING BARS. REINFORCING BARS SHALL BE MOVED TO MISS SHEET PILE INTERLOCKS.
3. MONOLITH JOINTS SHALL BE LOCATED A MINIMUM OF 5 FEET FROM POINTS OF INTERSECTION.
4. ANY SUBSTITUTIONS SHALL BE SUBMITTED TO THE CONTRACTING OFFICER REPRESENTATIVE FOR APPROVAL.
5. STEEL SHEET PILE SURFACE PREPARATION AND PAINTING SHALL BE IN ACCORDANCE WITH SECTION 09940 OF THE SPECIFICATIONS.
6. STATIONS FOR SPECIAL Z-PILE TEES ARE APPROXIMATE.



NOTE "A"
GROOVE WELD SHALL EXTEND THE FULL LENGTH OF THE SHEET PILE WEB AND FLANGES EXCLUDING THE INTERLOCKS.

SHEET PILE SPLICE DETAIL

SCALE: 3" = 1' - 0"



PROTECTED SIDE ELEVATION — I-WALL SIDE ELEVATION

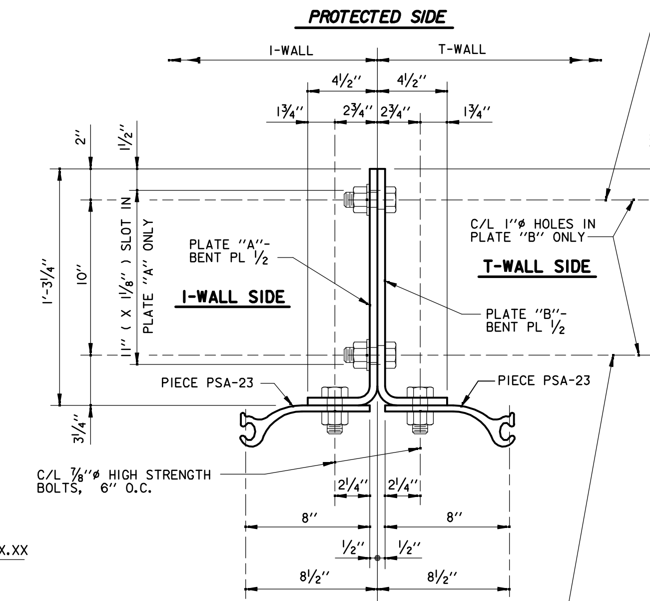
STA. XXX+XX.XX W/L

ELEVATION OF SPECIAL PSA-23 TEE SHOWING BENT PLATES BOLT SPACING

SCALE: HORIZ. 3/4" = 1'- 0"
VERT. 3/8" = 1'- 0"

LEGEND
P = PERMANENT BOLT
T = TEMPORARY BOLT
S = SLOTTED HOLE (PLATE "A" ONLY)

PERMANENT BOLTS
C/L 7/8"ø HIGH STRENGTH BOLTS W/ FLAT WASHERS.
SEE ELEVATION OF SPECIAL PSA-23 TEE SHOWING BENT PLATES BOLT SPACING.

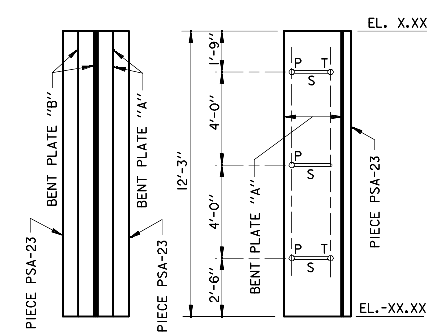


TEMPORARY DRIVING BOLTS
C/L 7/8"ø HIGH STRENGTH BOLTS W/ FLAT WASHERS.
SEE ELEVATION OF SPECIAL PSA-23 TEE SHOWING BENT PLATES BOLT SPACING.
THESE BOLTS ARE TO BE REMOVED WHEN EACH BOLT IS APPROXIMATELY 6" ABOVE GROUND LINE AS SPECIAL PSA-23 TEE IS DRIVEN, SO THAT UPON COMPLETION OF DRIVING NO TEMPORARY BOLTS SHALL REMAIN.

NOTE:
7/8"ø HIGH STRENGTH BOLTS SHALL BE HEAVY HEX BOLTS, ASTM A325, TYPE 3 W/ HEAVY HEX NUTS, ASTM A563, GRADE C3, PLAIN, WEATHERING STEEL AND HARDENED STEEL WASHERS (WHERE REQUIRED), ASTM F436, PLAIN, TYPE 3 (WEATHERING STEEL).

DETAIL 1
SPECIAL PSA-23 TEE

SCALE: 3" = 1' - 0"



PROTECTED SIDE ELEVATION — I-WALL SIDE ELEVATION

STA. XXX+XX.XX W/L - SHOWN
STA. XXX+XX.XX W/L - OPPOSITE HAND

ELEVATION OF SPECIAL PSA-23 TEE SHOWING BENT PLATES BOLT SPACING

SCALE: HORIZ. 3/4" = 1'- 0"
VERT. 3/8" = 1'- 0"

NOTES
1 . FOR GENERAL NOTES, SEE DWG.

SCALE: 3/8" = 1'- 0"
SCALE: 3/4" = 1'- 0"
SCALE: 1" = 1'- 0"
SCALE: 1 1/2" = 1'- 0"
SCALE: 3" = 1'- 0"

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

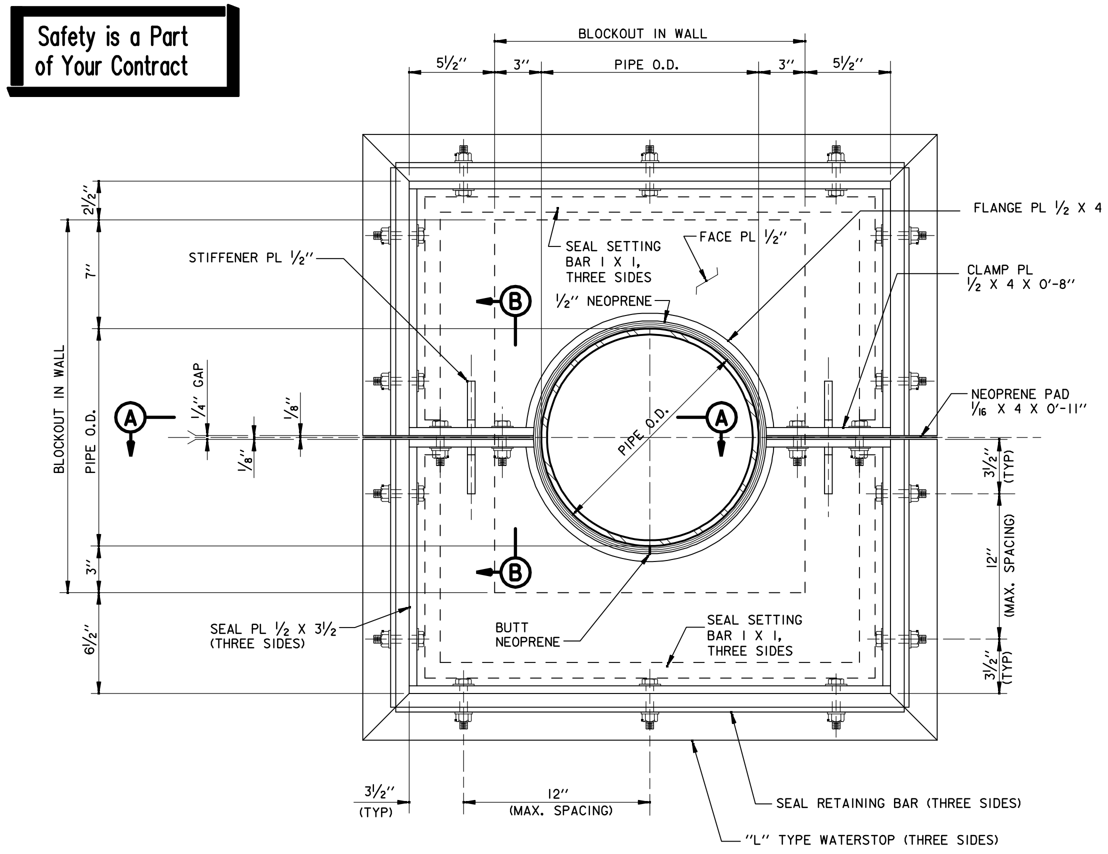
US Army Corps of Engineers New Orleans District

DESIGNED BY: CHECKED BY: DRAWN BY: DATE: 1 OCT 07
PLOT SCALE: 4
DESIGN FILE NAME: AETYP17.DGN
SOLICITATION NO. DACW29-

SHEET PILE DETAILS

FILE NUMBER
DWG. OF

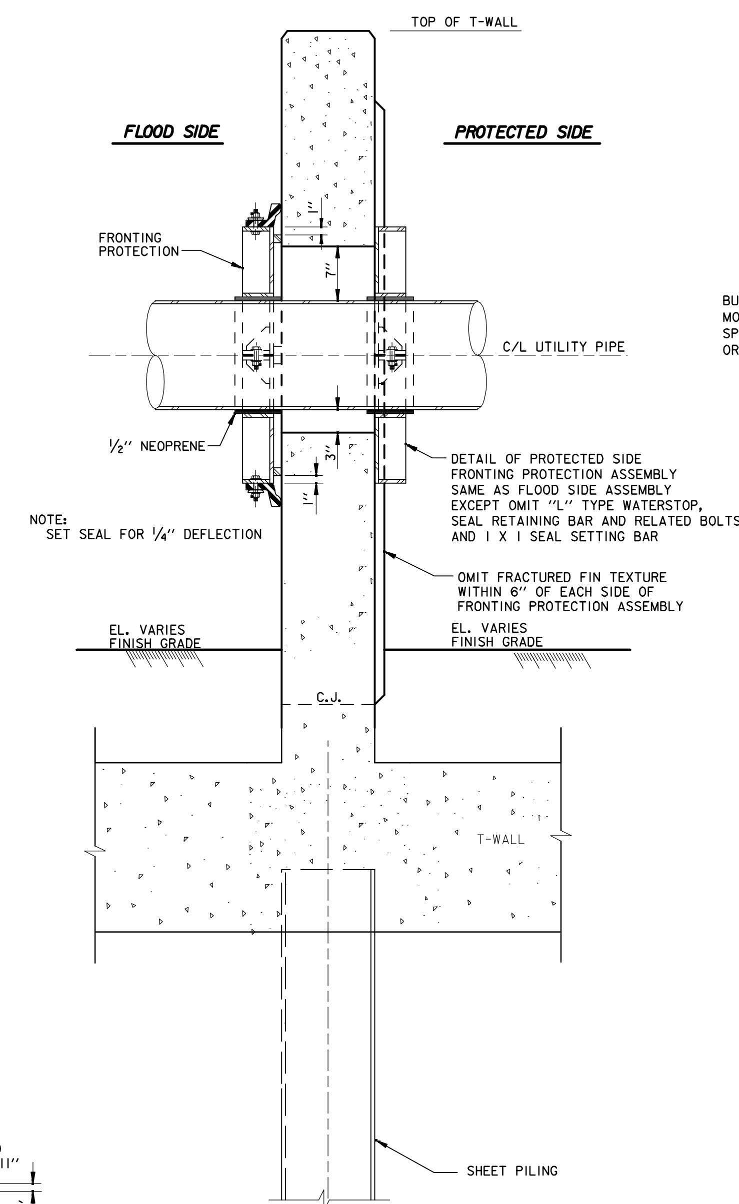
Safety is a Part of Your Contract



FLOOD SIDE ELEVATION

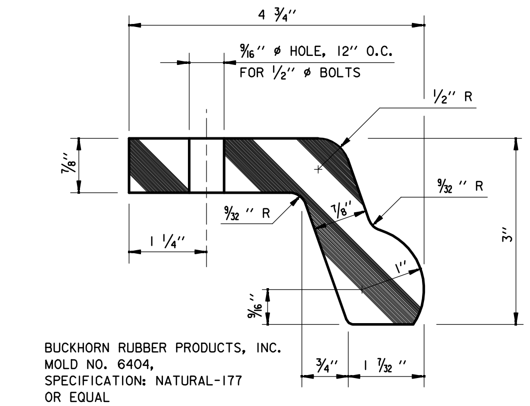
FRONTING PROTECTION UTILITY DETAIL

SCALE: 3" = 1'-0"



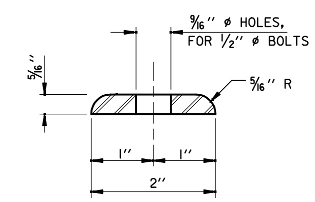
SECTION THRU FLOODWALL

SCALE: 1 ½" = 1'-0"



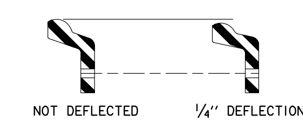
" L " TYPE WATERSTOP

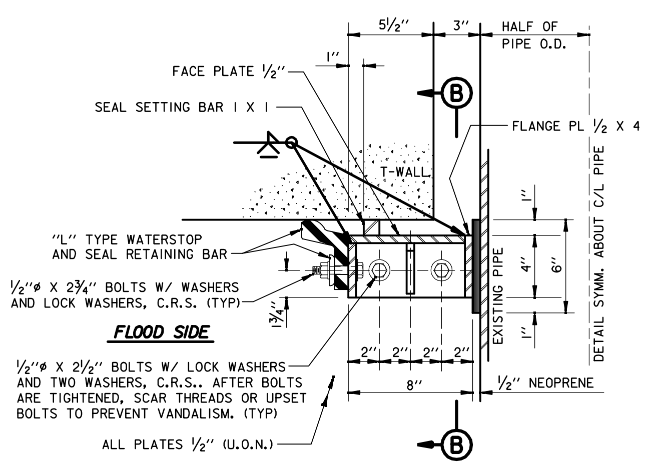
SCALE: 12" = 1'-0"



SEAL RETAINING BAR

SCALE: 12" = 1'-0"

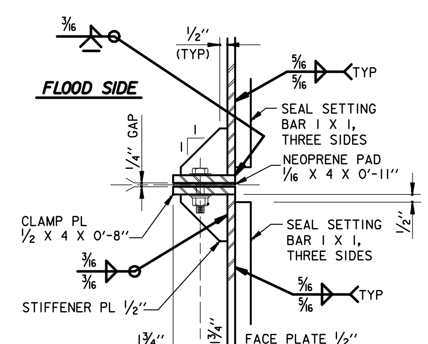




NOTE:
SET SEAL FOR A MINIMUM OF A ¼" INITIAL DEFLECTION.

SECTION (A)

SCALE: 3" = 1'-0"



SECTION (B)

SCALE: 3" = 1'-0"

NOTES:
1. THIS DETAIL WAS DRAWN FOR A 14" O.D. PIPE.
2. THIS DETAIL IS BASED ON THE NEW WALL FACE BEING PERPENDICULAR TO THE PIPE.
3. ALL MATERIAL FOR FRONTING PROTECTION SHALL BE ALUMINIUM, UNLESS OTHERWISE NOTED.
4. ALL STRUCTURAL ALUMINUM PLATES SHALL BE ASTM B 209, TYPE 6061-T6.
5. ALL CORRISON RESISTANT STEEL (C.R.S.) SHALL BE TYPE 316.
6. WELDS SHOWN ARE TYPICAL FOR SIMILAR JOINTS AND ALL WELDS ON WALL SIDE OF FACE PLATE SHALL BE FLUSH WITH BASE METAL.
7. AFTER THE TWO FRAMES ARE LOOSELY CLAMPED ON THE PIPE, THE TOTAL ASSEMBLY SHALL BE PUSHED AGAINST THE WALL, SETTING THE SEALS, THEN TIGHTEN CLAMP PLATES TO CLOSE THE ¼" GAP.
8. UPON COMPLETION OF THE ASSEMBLY, APPLY A ANAEROBIC ADHESIVE (LOCTITE THREADLOCKER 290 OR EQUAL) TO ALL NUT AND BOLT JUNCTURES.

SCALE: 1 ½" = 1'-0"

SCALE: 3" = 1'-0"

SCALE: 12" = 1'-0"

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U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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NEW ORLEANS, LOUISIANA

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PLOT SCALE: 4
SOLICITATION NO.: DACW29-

UTILITIES - A

FILE NUMBER

DWG. OF

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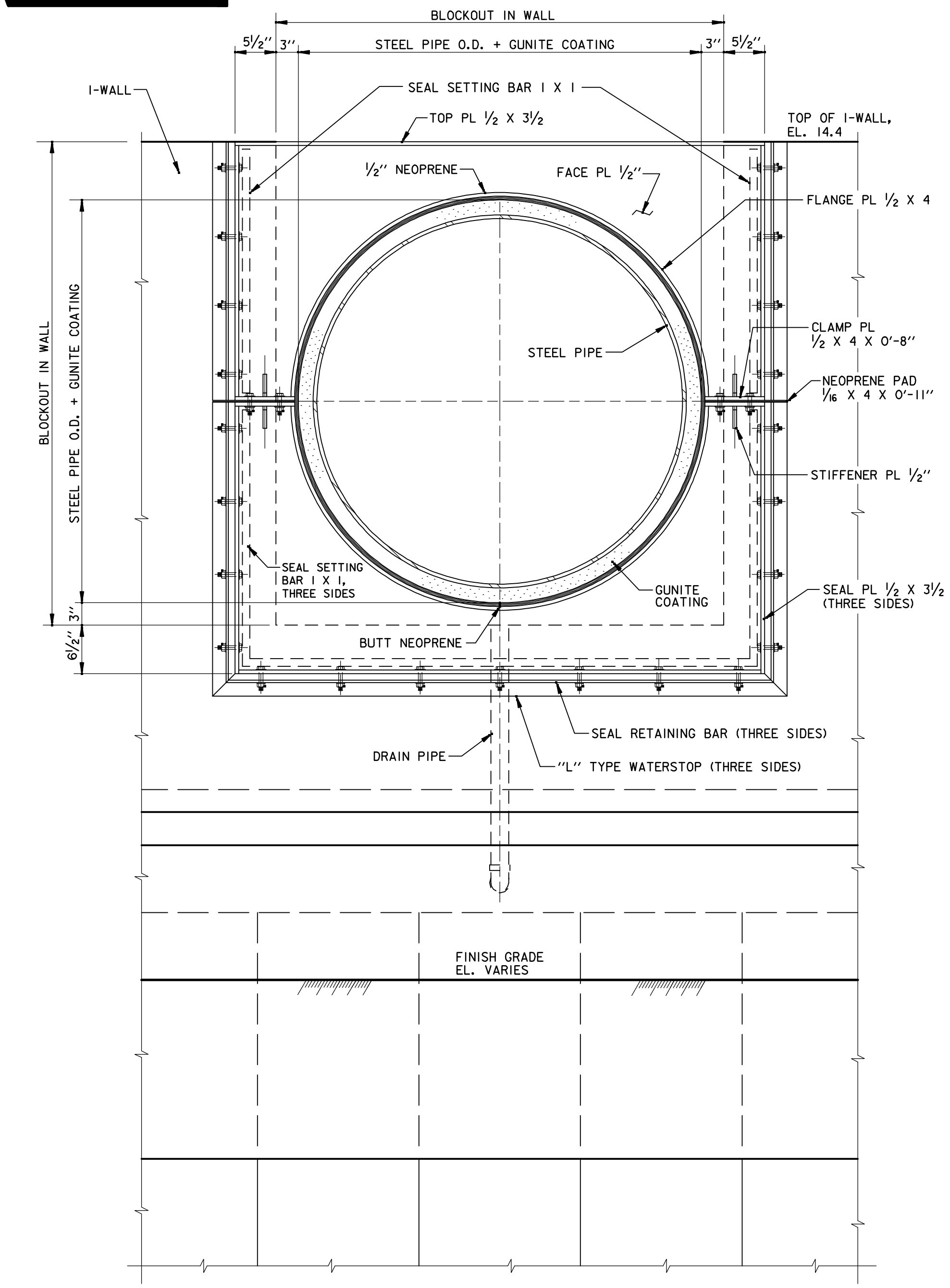
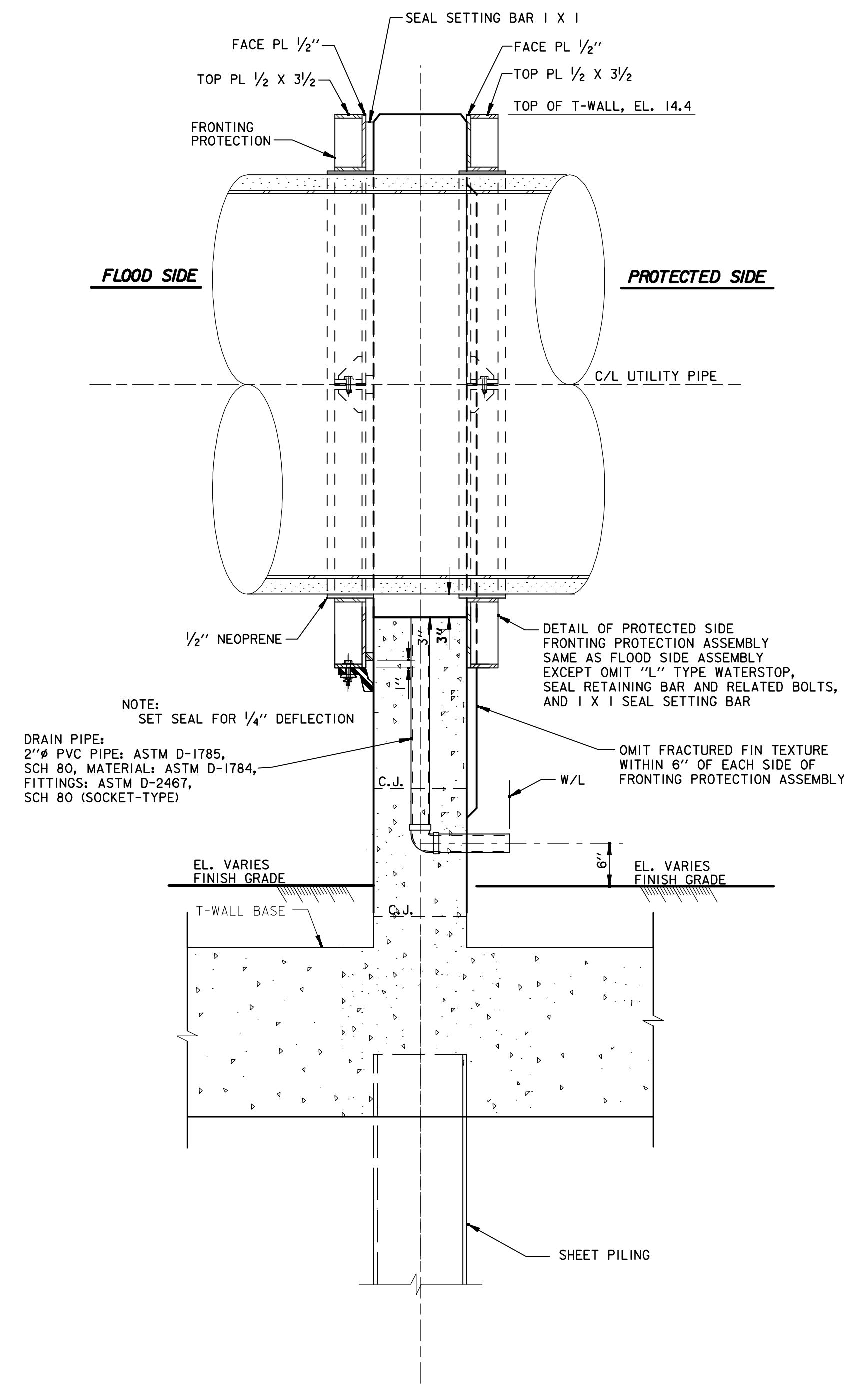
NOTES:

1. THIS DETAIL WAS DRAWN FOR A 50" O.D. PIPE PLUS 2" THICK GUNITE COATING ALL AROUND.
2. THIS DETAIL IS BASED ON THE NEW WALL FACE BEING PERPENDICULAR TO THE PIPE.
3. ALL MATERIAL FOR FRONTING PROTECTION SHALL BE ALUMINIUM, UNLESS OTHERWISE NOTED.
4. ALL STRUCTURAL ALUMINUM PLATES SHALL BE ASTM B 209, TYPE 6061-T6.
5. ALL CORRISON RESISTANT STEEL (C.R.S.) SHALL BE TYPE 316.
6. WELDS SHOWN ARE TYPICAL FOR SIMILAR JOINTS AND ALL WELDS ON WALL SIDE OF FACE PLATE SHALL BE FLUSH WITH BASE METAL.
7. AFTER THE TWO FRAMES ARE LOOSELY CLAMPED ON THE PIPE, THE TOTAL ASSEMBLY SHALL BE PUSHED AGAINST THE WALL, SETTING THE SEALS, THEN TIGHTEN CLAMP PLATES TO CLOSE THE 1/4" GAP.
8. UPON COMPLETION OF THE ASSEMBLY, APPLY A ANAEROBIC ADHESIVE (LOCTITE THREADLOCKER 290 OR EQUAL) TO ALL NUT AND BOLT JUNCTURES.



FLOOD SIDE ELEVATION

FRONTING PROTECTION UTILITY DETAIL



SECTION THRU FLOODWALL

SCALE: 1 1/2" = 1' - 0"

12" 0 1' 2' 3'

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UTILITIES - B

FILE NUMBER

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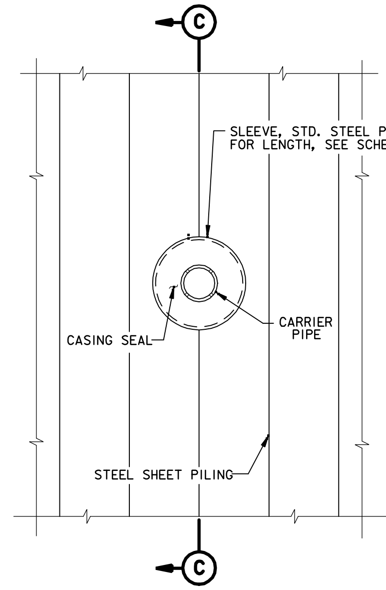
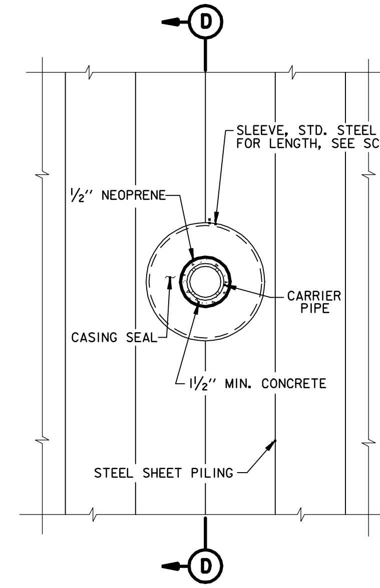
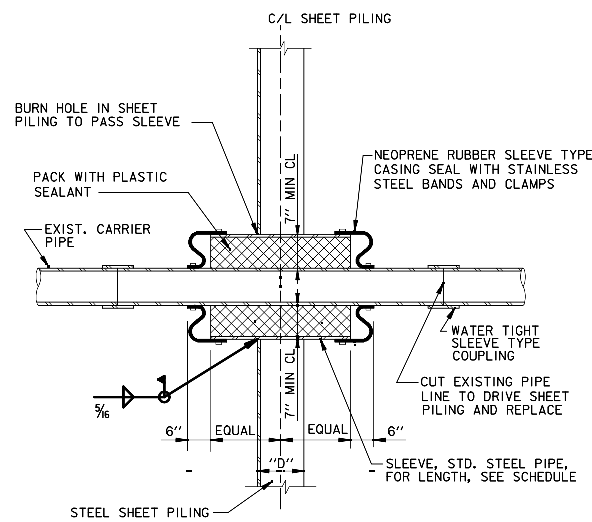


ELEVATION

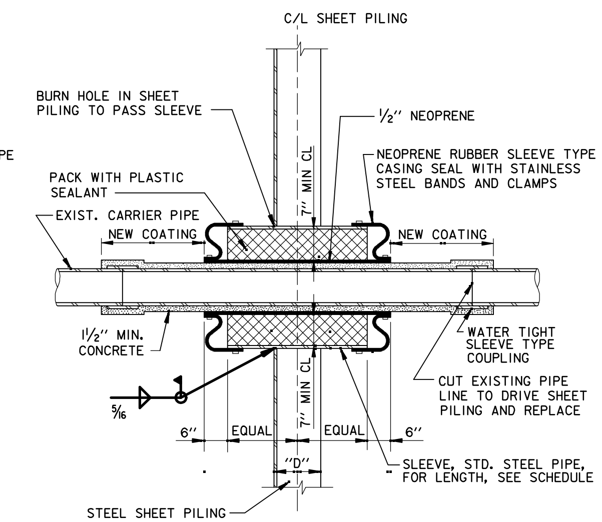


ELEVATION



SECTION C

TYPICAL PIPE THRU STEEL SHEET PILING



SECTION D

TYPICAL GAS AND PETROLEUM PIPES THRU STEEL SHEET PILING (CONCRETE COATED PIPE)

PIPE TABULATION AND SLEEVE TABLE

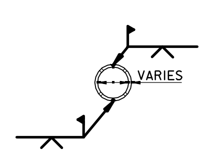
UTILITY MARK	APPROX. B/L STA.	EXISTING PIPE NEW PIPE (N)	MINIMUM SLEEVE SIZE		
			NOMINAL SIZE	INSIDE DIA.	OUTSIDE DIA.

SLEEVE SCHEDULE

SHEET PILE DEPTH ''D''	SLEEVE LENGTH
12'' OR LESS	3'-0''
GREATER THAN 12''	3'-4''

NOTES:

1. THE DETAILS SHOWN FOR GAS AND PETROLEUM PIPE PENETRATIONS SHALL APPLY TO PIPELINES CARRING FLAMMABLE MATERIALS OR ANY OTHER POTENTIALLY DANGEROUS COMMODITY.
2. THE ½'' NEOPRENE WRAP IS ONLY REQUIRED ON METALLIC CARRIER PIPES.



IF CONDITIONS PERMIT, AN ALTERNATE METHOD OF PASSING A UTILITY LINE THROUGH SHEET PILE CAN BE ACCOMPLISHED WITHOUT CUTTING THE UTILITY LINE. THIS METHOD CONSIST OF LATERALLY DISPLACING THE UTILITY LINE, DRIVING THE SHEET PILING, NOTCHING THE SHEET PILE AND INSTALLING SLEEVES IN HALVES.

SLEEVE INSTALLATION IN HALVES

SCALE: ¾'' = 1'- 0''

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UTILITIES - C

FILE NUMBER

DWG. OF

12-10

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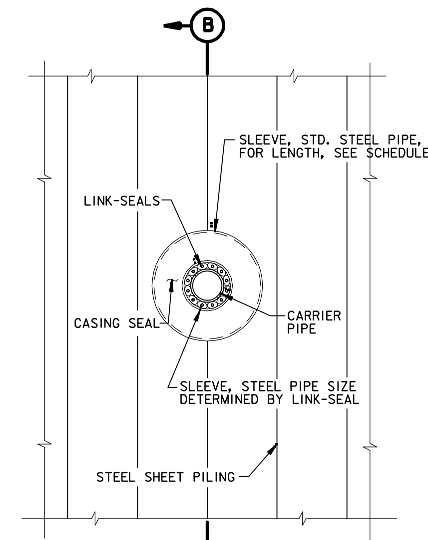
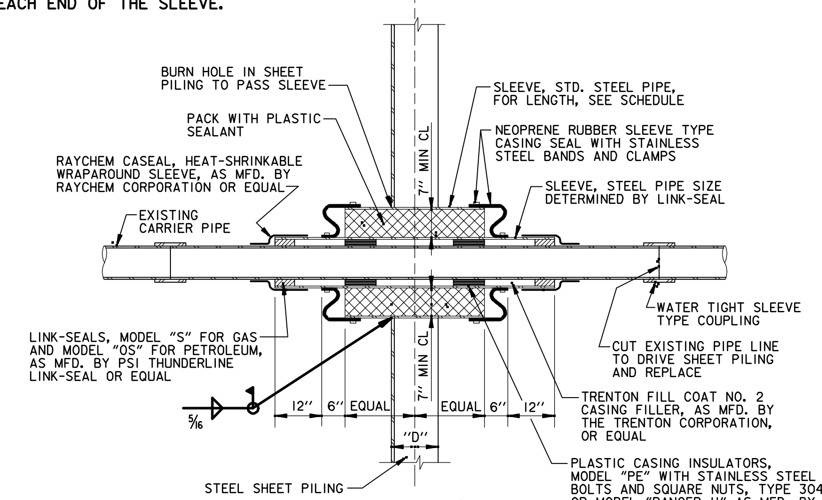


NOTE:
RAYCHEM CASEAL, HEAT-SHRINKABLE WRAPAROUND SLEEVE NOT SHOWN FOR CLARITY

ELEVATION

NOTES:

1. THE DETAILS SHOWN FOR GAS AND PETROLEUM PIPE PENETRATIONS SHALL APPLY TO PIPELINES CARRING FLAMMABLE MATERIALS OR ANY OTHER POTENTIALLY DANGEROUS COMMODITY.
2. A MINIMUM OF TWO (2) INSULATORS MUST BE USED, ONE (1) NEAR EACH END OF THE SLEEVE.



SECTION B

TYPICAL GAS AND PETROLEUM PIPES THRU STEEL SHEET PILING

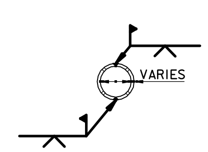
(OTHER THAN CONCRETE COATED PIPE, OPTION I)

PIPE TABULATION AND SLEEVE TABLE

UTILITY MARK	APPROX. B/L STA.	EXISTING PIPE NEW PIPE (N)	MINIUMUM SLEEVE SIZE NOMINAL SIZE	INSIDE DIA.	OUTSIDE DIA.

SLEEVE SCHEDULE

SHEET PILE DEPTH "D"	SLEEVE LENGTH
12" OR LESS	3'-0"
GREATER THAN 12"	3'-4"



IF CONDITIONS PERMIT, AN ALTERNATE METHOD OF PASSING A UTILITY LINE THROUGH SHEET PILE CAN BE ACCOMPLISHED WITHOUT CUTTING THE UTILITY LINE. THIS METHOD CONSIST OF LATERALLY DISPLACING THE UTILITY LINE, DRIVING THE SHEET PILING, NOTCHING THE SHEET PILE AND INSTALLING SLEEVES IN HALVES.

SLEEVE INSTALLATION IN HALVES

SCALE: 3/4" = 1'- 0"

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CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DATE: 1 OCT 07 | PLOT SCALE: 16 | SOLICITATION NO. DACW29-

UTILITIES
GAS AND PETROLEUM
OPTION I

FILE NUMBER

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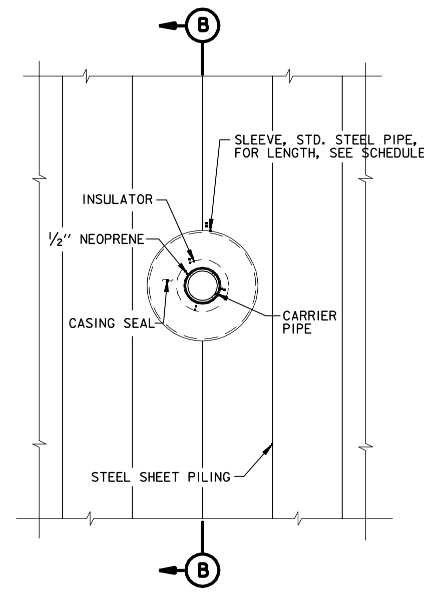
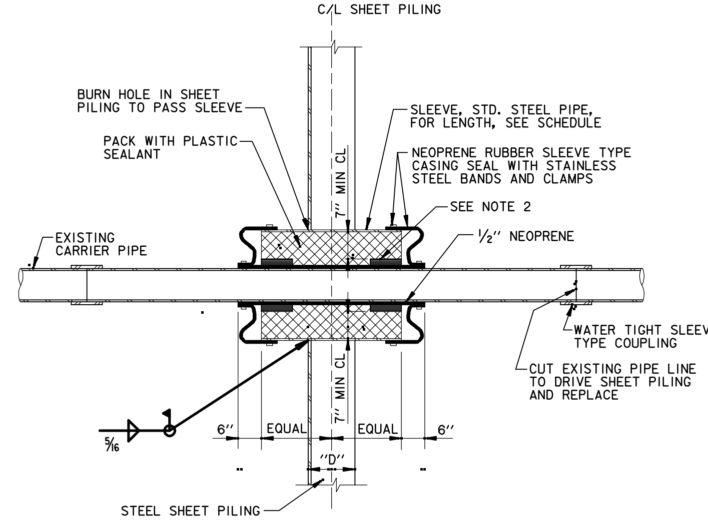


ELEVATION



SECTION B

TYPICAL GAS AND PETROLEUM PIPES THRU STEEL SHEET PILING

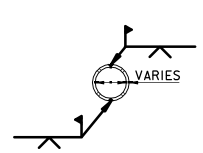
(OTHER THAN CONCRETE COATED PIPE, OPTION 2)

PIPE TABULATION AND SLEEVE TABLE

UTILITY MARK	APPROX. B/L STA.	EXISTING PIPE NEW PIPE (N)	MINIMUM SLEEVE SIZE		
			NOMINAL SIZE	INSIDE DIA.	OUTSIDE DIA.

SLEEVE SCHEDULE

SHEET PILE DEPTH "D"	SLEEVE LENGTH
12" OR LESS	3'-0"
GREATER THAN 12"	3'-4"



IF CONDITIONS PERMIT, AN ALTERNATE METHOD OF PASSING A UTILITY LINE THROUGH SHEET PILE CAN BE ACCOMPLISHED WITHOUT CUTTING THE UTILITY LINE. THIS METHOD CONSIST OF LATERALLY DISPLACING THE UTILITY LINE, DRIVING THE SHEET PILING, NOTCHING THE SHEET PILE AND INSTALLING SLEEVES IN HALVES.

SLEEVE INSTALLATION IN HALVES

NOTES:

1. THE DETAILS SHOWN FOR GAS AND PETROLEUM PIPE PENETRATIONS SHALL APPLY TO PIPELINES CARRING FLAMMABLE MATERIALS OR ANY OTHER POTENTIALLY DANGEROUS COMMODITY.
2. PIPELINE INSULATORS SHALL BE PLACED OVER THE ½" NEOPRENE. A MINIMUM OF 2 INSULATORS MUST BE USED, ONE (1) NEAR EACH END OF THE SLEEVE. FUSION COATED STEEL CASING INSULATORS, MODEL "C8G-2" OR "C12G-2" WITH A 14 GAGE BAND AND STAINLESS STEEL CONNECTING HARDWARE OR MODEL "RANGER II", MAXIMUM HEIGHT OF INSULATOR RISER + RUNNER SHALL BE 2", AS MFD. BY PIPELINE SEAL AND INSULATOR, INC. INTERNAL CLEARANCES SHALL BE MEASURED FROM THE OUTSIDE OF THE RUNNER TO THE INSIDE OF THE SLEEVE AND MUST BE AT LEAST 7" CLEARANCE ALL AROUND. THIS MAY REQUIRE INCREASING THE SLEEVE DIAMETER FROM THAT SHOWN ABOVE.

SCALE: ¾" = 1'- 0"

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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NEW ORLEANS, LOUISIANA

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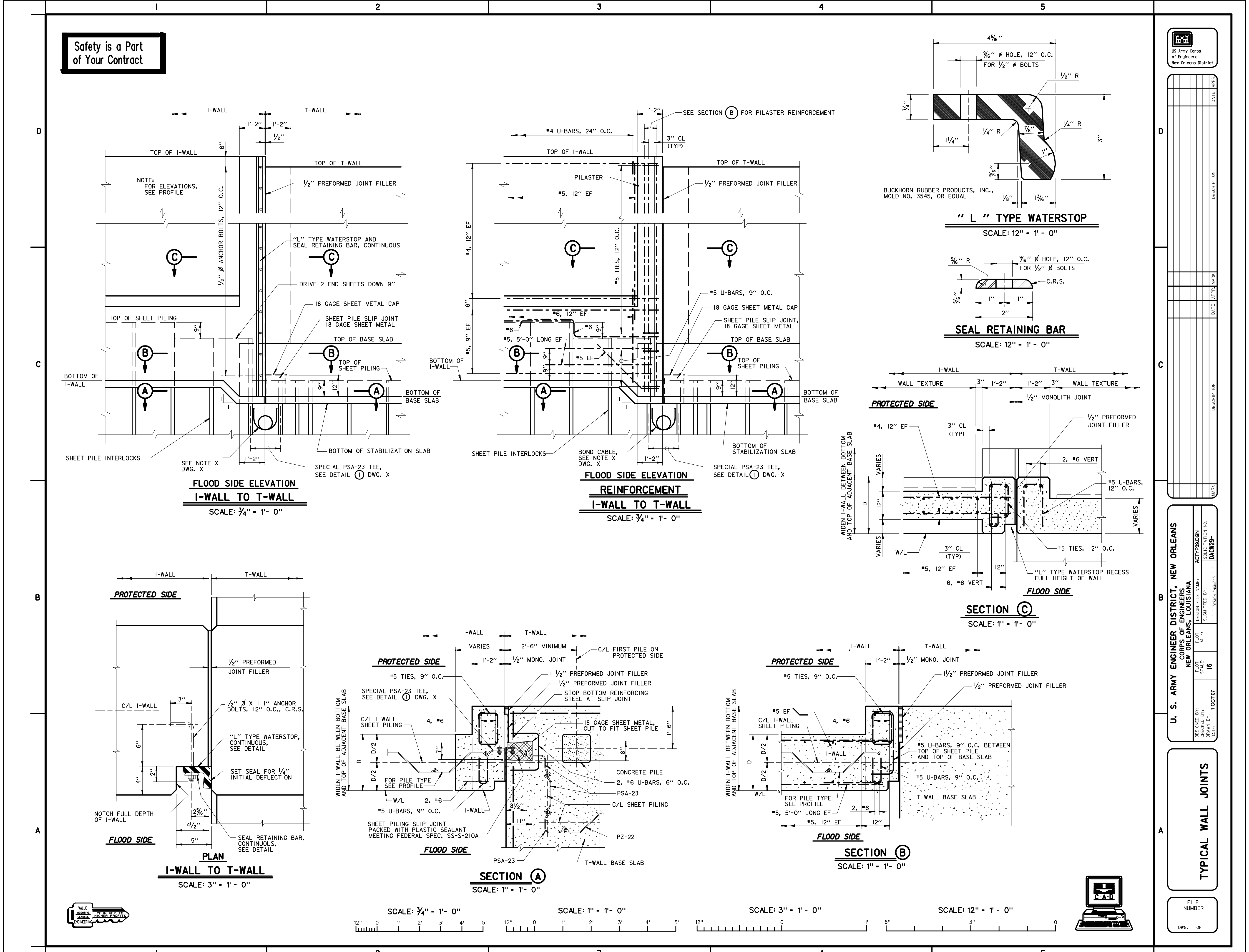
UTILITIES GAS AND PETROLEUM OPTION 2

FILE NUMBER
DWG. OF



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FLOOD SIDE ELEVATION
I-WALL TO T-WALL
SCALE: 3/4" = 1'- 0"
I-WALL
T-WALL
TOP OF I-WALL
TOP OF T-WALL
NOTE: FOR ELEVATIONS, SEE PROFILE
1/2" Ø ANCHOR BOLTS, 12" O.C.
1/2" PREFORMED JOINT FILLER
"L" TYPE WATERSTOP AND SEAL RETAINING BAR, CONTINUOUS
DRIVE 2 END SHEETS DOWN 9"
18 GAGE SHEET METAL CAP
SHEET PILE SLIP JOINT 18 GAGE SHEET METAL
TOP OF BASE SLAB
TOP OF SHEET PILING
BOTTOM OF I-WALL
BOTTOM OF BASE SLAB
BOTTOM OF STABILIZATION SLAB
SHEET PILE INTERLOCKS
SEE NOTE X DWG. X
SPECIAL PSA-23 TEE, SEE DETAIL I DWG. X
FLOOD SIDE ELEVATION
REINFORCEMENT
I-WALL TO T-WALL
SCALE: 3/4" = 1'- 0"
SEE SECTION B FOR PILASTER REINFORCEMENT
#4 U-BARS, 24" O.C.
3" CL (TYP)
PILASTER
#5, 12" EF
#4, 12" EF
#5 TIES, 12" O.C.
#5 U-BARS, 9" O.C.
#6, 12" EF
#5, 5'-0" LONG EF
#5, 9" EF
#5 EF
SHEET PILE SLIP JOINT, 18 GAGE SHEET METAL
BOND CABLE, SEE NOTE X DWG. X
"L" TYPE WATERSTOP
SCALE: 12" = 1'- 0"
9/16" Ø HOLE, 12" O.C. FOR 1/2" Ø BOLTS
BUCKHORN RUBBER PRODUCTS, INC., MOLD NO. 3545, OR EQUAL
SEAL RETAINING BAR
SCALE: 12" = 1'- 0"
C.R.S.
SECTION C
SCALE: 1" = 1'- 0"
PROTECTED SIDE
FLOOD SIDE
WALL TEXTURE
1/2" MONOLITH JOINT
2, #6 VERT
#5 U-BARS, 12" O.C.
#5 TIES, 12" O.C.
"L" TYPE WATERSTOP RECESS FULL HEIGHT OF WALL
6, #6 VERT
WIDEN I-WALL BETWEEN BOTTOM AND TOP OF ADJACENT BASE SLAB
PLAN
I-WALL TO T-WALL
SCALE: 3" = 1'- 0"
C/L I-WALL
1/2" PREFORMED JOINT FILLER
1/2" Ø X 1 1" ANCHOR BOLTS, 12" O.C., C.R.S.
"L" TYPE WATERSTOP, CONTINUOUS, SEE DETAIL
SET SEAL FOR 1/4" INITIAL DEFLECTION
NOTCH FULL DEPTH OF I-WALL
SEAL RETAINING BAR, CONTINUOUS, SEE DETAIL
SECTION A
SCALE: 1" = 1'- 0"
C/L FIRST PILE ON PROTECTED SIDE
2'-6" MINIMUM
1/2" MONO. JOINT
1 1/2" PREFORMED JOINT FILLER
STOP BOTTOM REINFORCING STEEL AT SLIP JOINT
18 GAGE SHEET METAL, CUT TO FIT SHEET PILE
CONCRETE PILE
2, #6 U-BARS, 6" O.C.
PSA-23
C/L SHEET PILING
PZ-22
T-WALL BASE SLAB
C/L I-WALL SHEET PILING
FOR PILE TYPE SEE PROFILE
SHEET PILING SLIP JOINT PACKED WITH PLASTIC SEALANT MEETING FEDERAL SPEC. SS-S-210A
SECTION B
SCALE: 1" = 1'- 0"
#5 U-BARS, 9" O.C. BETWEEN TOP OF SHEET PILE AND TOP OF BASE SLAB
#5, 5'-0" LONG EF
SCALE: 3/4" = 1'- 0"
SCALE: 1" = 1'- 0"
SCALE: 3" = 1'- 0"
SCALE: 12" = 1'- 0"
US Army Corps of Engineers New Orleans District
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
AETYP08.DGN
DACW29-
PLOT SCALE: 16
DATE: 1 OCT 07
TYPICAL WALL JOINTS
FILE NUMBER

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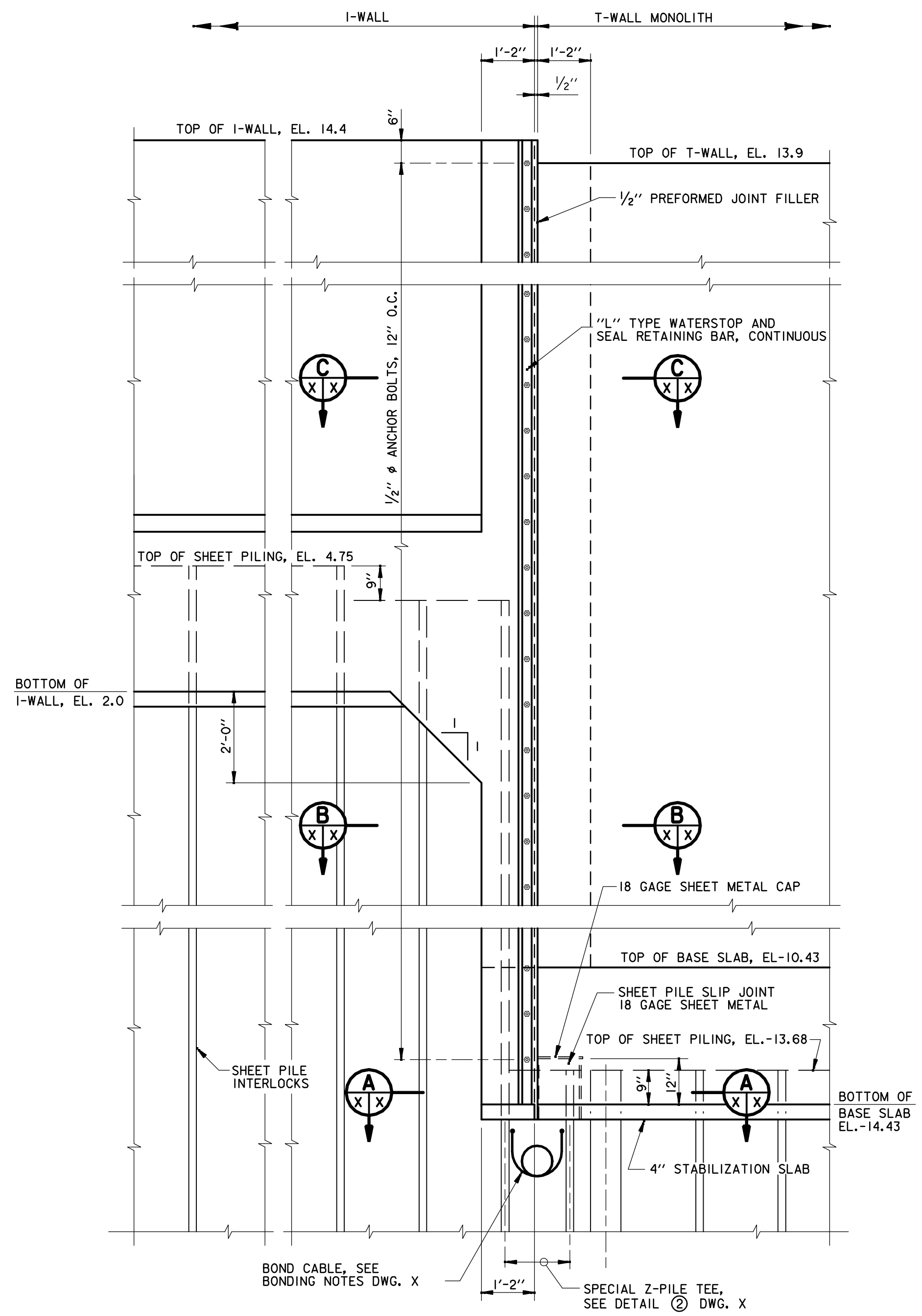
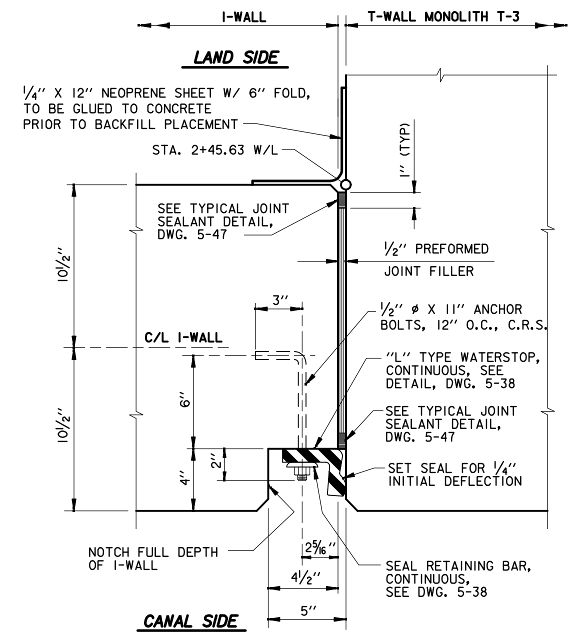


CANAL SIDE ELEVATION

I-WALL TO T-WALL MONOLITH T-3

SCALE: 3/4" = 1'- 0"



PLAN

I-WALL TO T-WALL MONOLITH T-3

SCALE: 3" = 1'- 0"

NOTES:

1. FOR GENERAL NOTES, SEE DWG. X
2. FOR LAP LENGTHS AND EMBEDMENT LENGTHS, SEE TABLE, DWG. X
3. FOR CONCRETE NOTES, SEE DWG. X
4. FOR PLAN, SEE DWG. X
5. FOR PROFILE, SEE DWGS. X AND X.
6. FOR SHEET PILE LAYOUT, SEE DWG. X
7. FOR SHEET PILE JOINT DETAILS, SEE DWG. X
8. FOR ARCHITECTURAL FINISH, SEE DWG.X

SCALE: 3/4" = 1'- 0"

12" 0 1' 2' 3' 4' 5'

SCALE: 3" = 1'- 0"

12" 0 1'

US Army Corps of Engineers New Orleans District

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
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NEW ORLEANS, LOUISIANA

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PLOT SCALE: 16

DESIGN FILE NAME: AETYPO9A.DGN

SOLICITATION NO. DACW29-

TYPICAL WALL JOINTS
ALTERNATE - MASONRY

FILE NUMBER

DWG. OF

12-14

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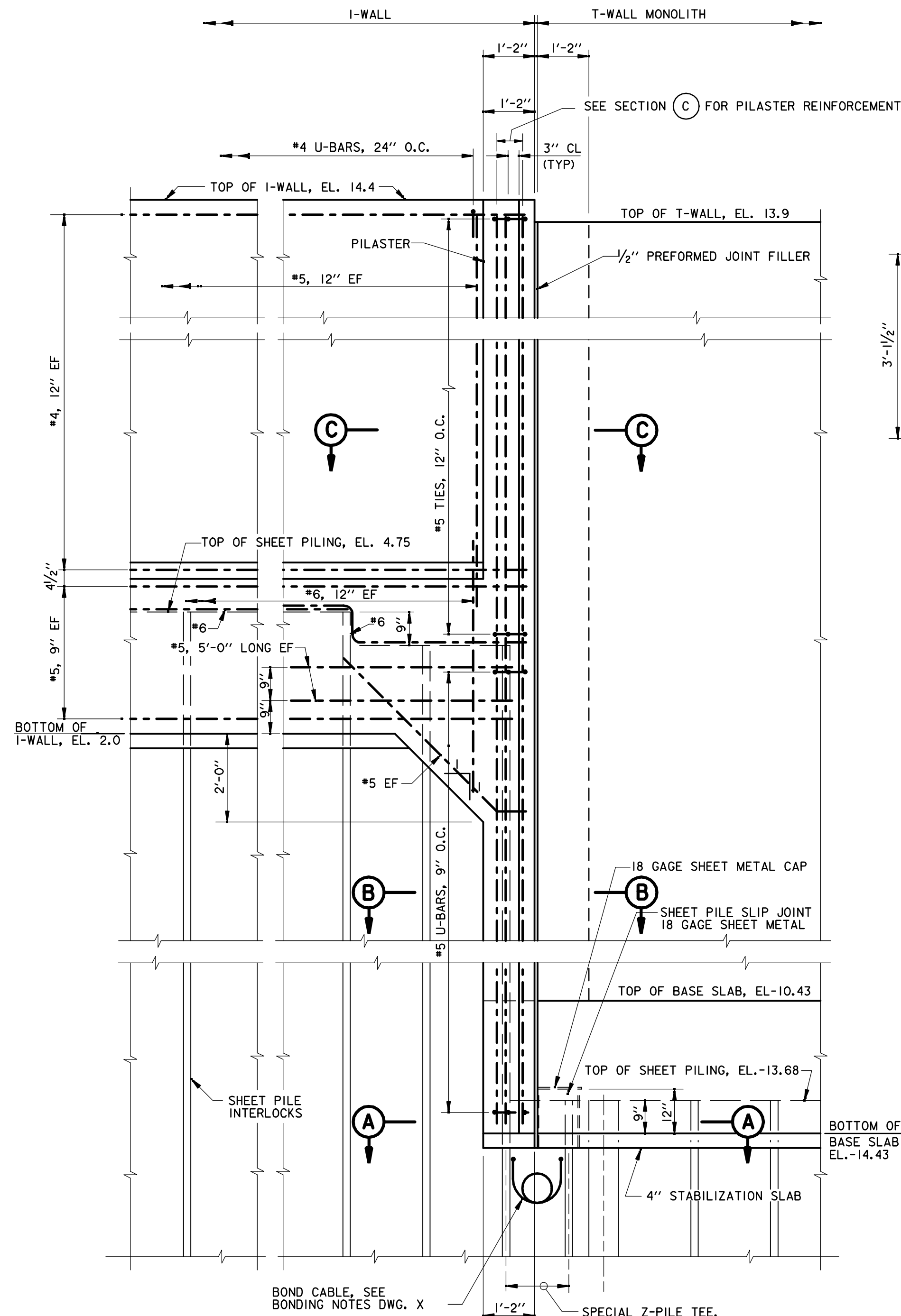
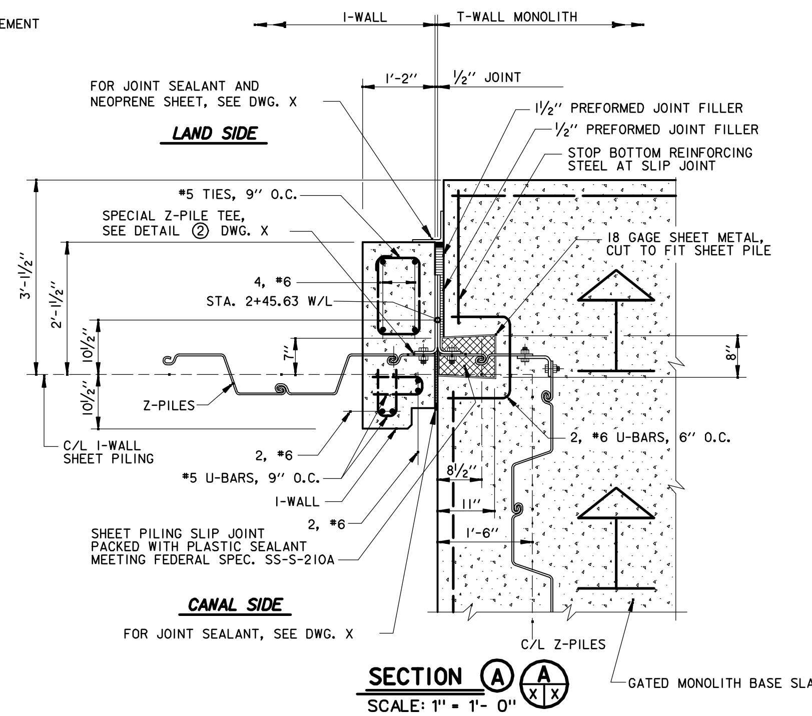
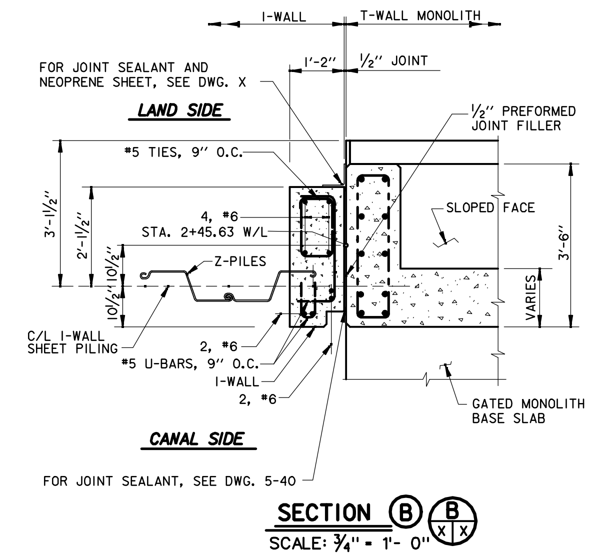


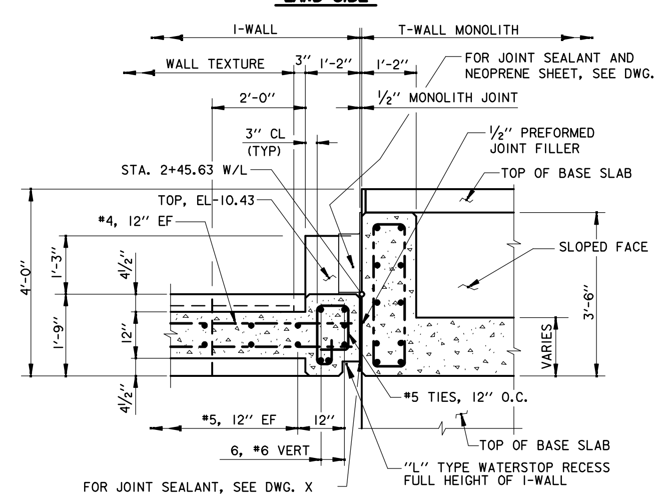
CANAL SIDE ELEVATION
REINFORCEMENT
I-WALL TO T-WALL MONOLITH T-3
SCALE: 3/4" = 1'- 0"



SECTION A
SCALE: 1" = 1'- 0"



SECTION B
SCALE: 3/4" = 1'- 0"



SECTION C
SCALE: 3/4" = 1'- 0"

NOTES:

1. FOR GENERAL NOTES, SEE DWG. X
2. FOR LAP LENGTHS AND EMBEDMENT LENGTHS, SEE TABLE, DWG. X
3. FOR CONCRETE NOTES, SEE DWG. X
4. FOR PLAN, SEE DWG. X
5. FOR PROFILE, SEE DWGS. X AND X
6. FOR SHEET PILE LAYOUT, SEE DWG. X
7. FOR SHEET PILE JOINT DETAILS, SEE DWG. X
8. FOR ARCHITECTURAL FINISH, SEE DWG. X

SCALE: 3/4" = 1'- 0"

SCALE: 1" = 1'- 0"

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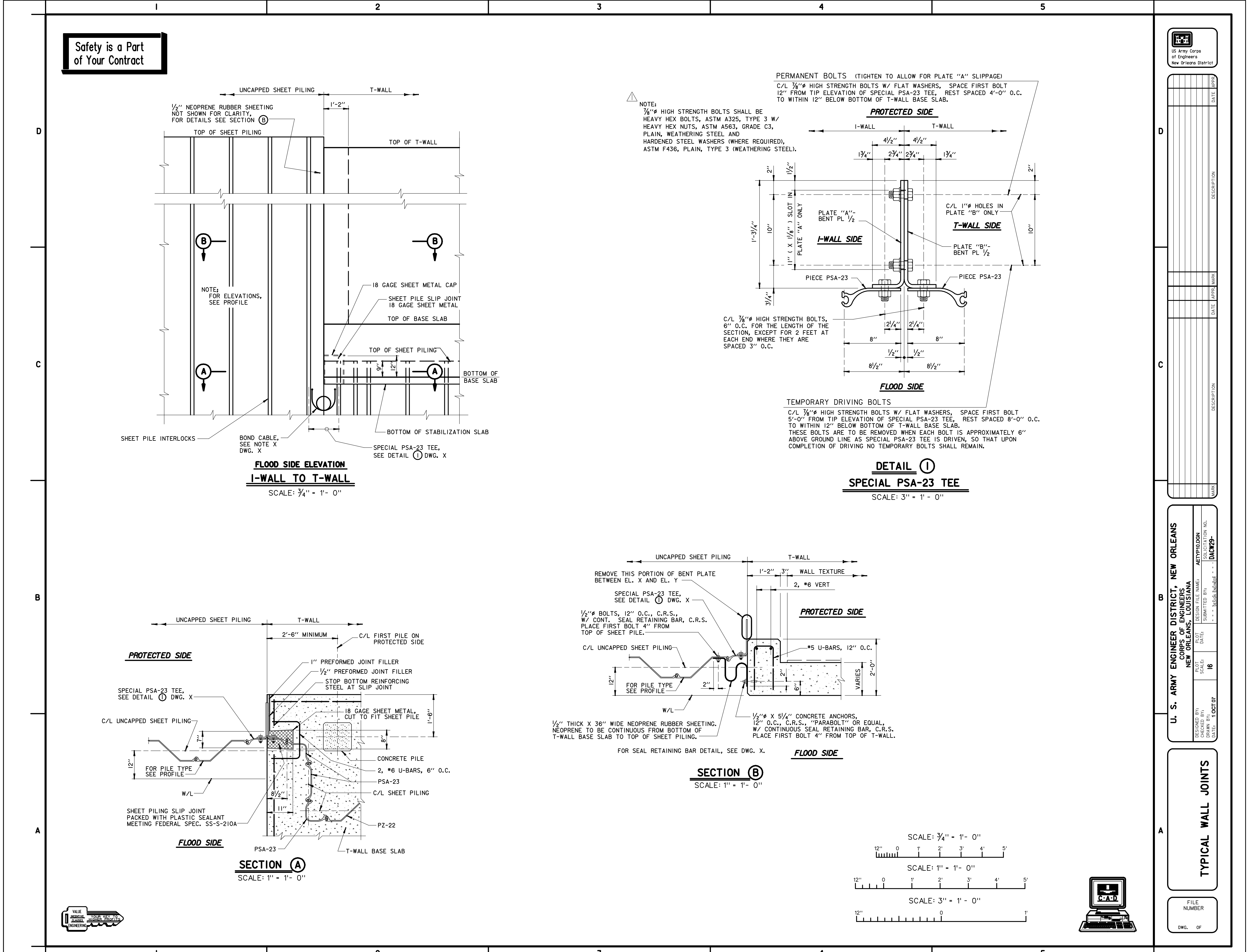
NEW ORLEANS, LOUISIANA

DESIGN FILE NAME: AETYP098.DGN
SOLICITATION NO.: DACW29-
PLOT SCALE: 16
DATE: 1 OCT 07

TYPICAL WALL JOINTS
ALTERNATE - REINFORCEMENT

FILE NUMBER

DWG. OF


Safety is a Part of Your Contract
UNCAPPED SHEET PILING
T-WALL
1'-2"
1/2" NEOPRENE RUBBER SHEETING NOT SHOWN FOR CLARITY, FOR DETAILS SEE SECTION B
TOP OF SHEET PILING
TOP OF T-WALL
NOTE: FOR ELEVATIONS, SEE PROFILE
18 GAGE SHEET METAL CAP
SHEET PILE SLIP JOINT 18 GAGE SHEET METAL
TOP OF BASE SLAB
TOP OF SHEET PILING
BOTTOM OF BASE SLAB
SHEET PILE INTERLOCKS
BOND CABLE, SEE NOTE X DWG. X
BOTTOM OF STABILIZATION SLAB
SPECIAL PSA-23 TEE, SEE DETAIL 1 DWG. X
FLOOD SIDE ELEVATION
I-WALL TO T-WALL
SCALE: 3/4" = 1'- 0"
NOTE:
7/8"ø HIGH STRENGTH BOLTS SHALL BE HEAVY HEX BOLTS, ASTM A325, TYPE 3 W/ HEAVY HEX NUTS, ASTM A563, GRADE C3, PLAIN, WEATHERING STEEL AND HARDENED STEEL WASHERS (WHERE REQUIRED), ASTM F436, PLAIN, TYPE 3 (WEATHERING STEEL).
PERMANENT BOLTS (TIGHTEN TO ALLOW FOR PLATE "A" SLIPPAGE)
C/L 7/8"ø HIGH STRENGTH BOLTS W/ FLAT WASHERS. SPACE FIRST BOLT 12" FROM TIP ELEVATION OF SPECIAL PSA-23 TEE. REST SPACED 4'-0" O.C. TO WITHIN 12" BELOW BOTTOM OF T-WALL BASE SLAB.
PROTECTED SIDE
I-WALL
T-WALL
PLATE "A"- BENT PL 1/2
I-WALL SIDE
C/L 1"ø HOLES IN PLATE "B" ONLY
T-WALL SIDE
PLATE "B"- BENT PL 1/2
11" (X 1 1/8") SLOT IN PLATE "A" ONLY
PIECE PSA-23
PIECE PSA-23
C/L 7/8"ø HIGH STRENGTH BOLTS, 6" O.C. FOR THE LENGTH OF THE SECTION, EXCEPT FOR 2 FEET AT EACH END WHERE THEY ARE SPACED 3" O.C.
FLOOD SIDE
TEMPORARY DRIVING BOLTS
C/L 7/8"ø HIGH STRENGTH BOLTS W/ FLAT WASHERS. SPACE FIRST BOLT 5'-0" FROM TIP ELEVATION OF SPECIAL PSA-23 TEE. REST SPACED 8'-0" O.C. TO WITHIN 12" BELOW BOTTOM OF T-WALL BASE SLAB.
THESE BOLTS ARE TO BE REMOVED WHEN EACH BOLT IS APPROXIMATELY 6" ABOVE GROUND LINE AS SPECIAL PSA-23 TEE IS DRIVEN, SO THAT UPON COMPLETION OF DRIVING NO TEMPORARY BOLTS SHALL REMAIN.
DETAIL 1
SPECIAL PSA-23 TEE
SCALE: 3" = 1'- 0"
UNCAPPED SHEET PILING
T-WALL
2'-6" MINIMUM
C/L FIRST PILE ON PROTECTED SIDE
PROTECTED SIDE
1" PREFORMED JOINT FILLER
1/2" PREFORMED JOINT FILLER
STOP BOTTOM REINFORCING STEEL AT SLIP JOINT
SPECIAL PSA-23 TEE, SEE DETAIL 1 DWG. X
18 GAGE SHEET METAL, CUT TO FIT SHEET PILE
C/L UNCAPPED SHEET PILING
FOR PILE TYPE SEE PROFILE
CONCRETE PILE
2, #6 U-BARS, 6" O.C.
PSA-23
W/L
C/L SHEET PILING
SHEET PILING SLIP JOINT PACKED WITH PLASTIC SEALANT MEETING FEDERAL SPEC. SS-S-210A
PZ-22
FLOOD SIDE
PSA-23
T-WALL BASE SLAB
SECTION A
SCALE: 1" = 1'- 0"
UNCAPPED SHEET PILING
T-WALL
REMOVE THIS PORTION OF BENT PLATE BETWEEN EL. X AND EL. Y
1'-2"
3"
WALL TEXTURE
2, #6 VERT
SPECIAL PSA-23 TEE, SEE DETAIL 1 DWG. X
1/2"ø BOLTS, 12" O.C., C.R.S., W/ CONT. SEAL RETAINING BAR, C.R.S. PLACE FIRST BOLT 4" FROM TOP OF SHEET PILE.
PROTECTED SIDE
C/L UNCAPPED SHEET PILING
#5 U-BARS, 12" O.C.
FOR PILE TYPE SEE PROFILE
VARIES
W/L
1/2" THICK X 36" WIDE NEOPRENE RUBBER SHEETING. NEOPRENE TO BE CONTINUOUS FROM BOTTOM OF T-WALL BASE SLAB TO TOP OF SHEET PILING.
1/2"ø X 5 1/4" CONCRETE ANCHORS, 12" O.C., C.R.S., "PARABOLT" OR EQUAL, W/ CONTINUOUS SEAL RETAINING BAR, C.R.S. PLACE FIRST BOLT 4" FROM TOP OF T-WALL.
FOR SEAL RETAINING BAR DETAIL, SEE DWG. X.
FLOOD SIDE
SECTION B
SCALE: 1" = 1'- 0"
SCALE: 3/4" = 1'- 0"
SCALE: 1" = 1'- 0"
SCALE: 3" = 1'- 0"
US Army Corps of Engineers New Orleans District
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
DESIGN FILE NAME: AETYP10.DGN
SOLICITATION NO. DACW29-
PLOT SCALE: 16
DATE: 1 OCT 07
TYPICAL WALL JOINTS
FILE NUMBER
DWG. OF

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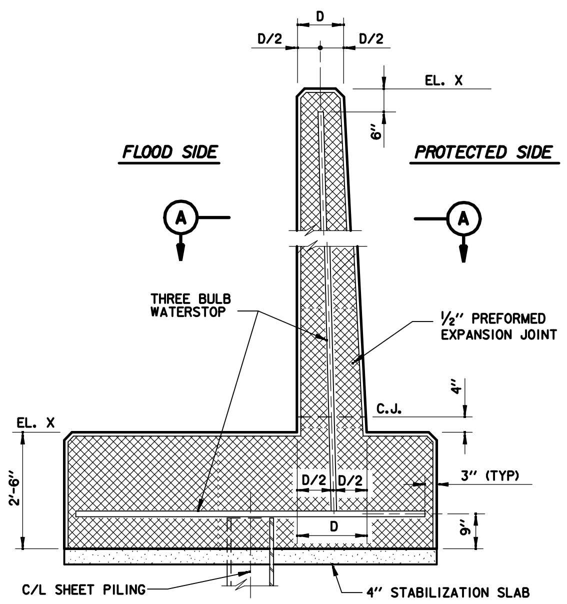
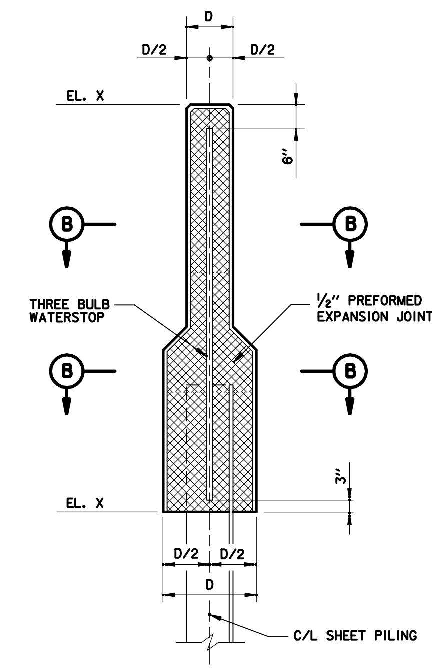


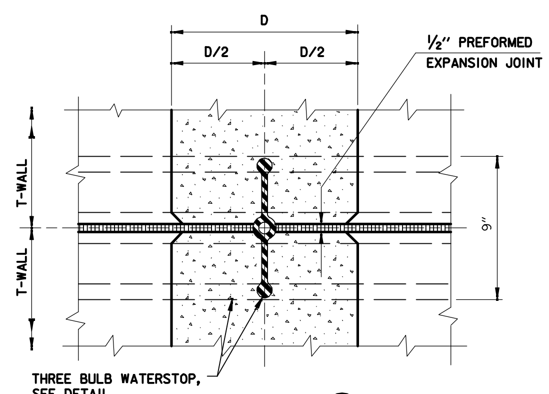
TYPICAL T-WALL JOINT

SCALE: 3/4" = 1'- 0"



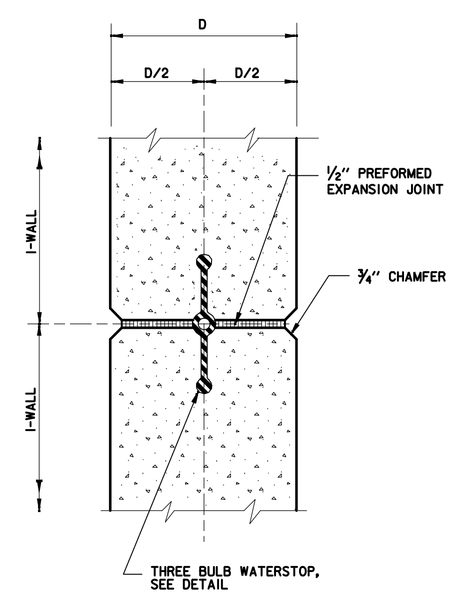
TYPICAL I-WALL JOINT

SCALE: 3/4" = 1'- 0"



SECTION A

SCALE: 3" = 1' - 0"

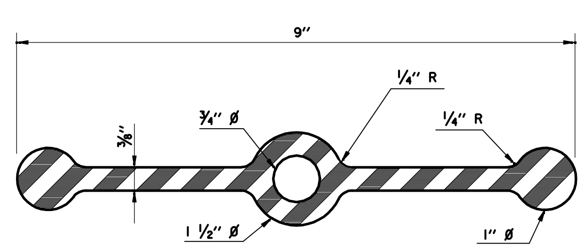


SECTION B

SCALE: 3" = 1' - 0"

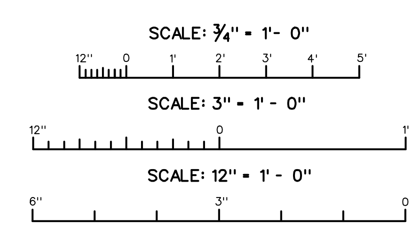
REFERENCE DRAWINGS

FOR PLAN AND PROFILE, SEE DWGS.
FOR TYPICAL WALL SECTIONS, SEE DWG.
FOR MISCELLANEOUS DETAILS, SEE DWG.



THREE BULB WATERSTOP

SCALE: 12" = 1' - 0"



US Army Corps of Engineers New Orleans District

MARK | DESCRIPTION | DATE | APPR.

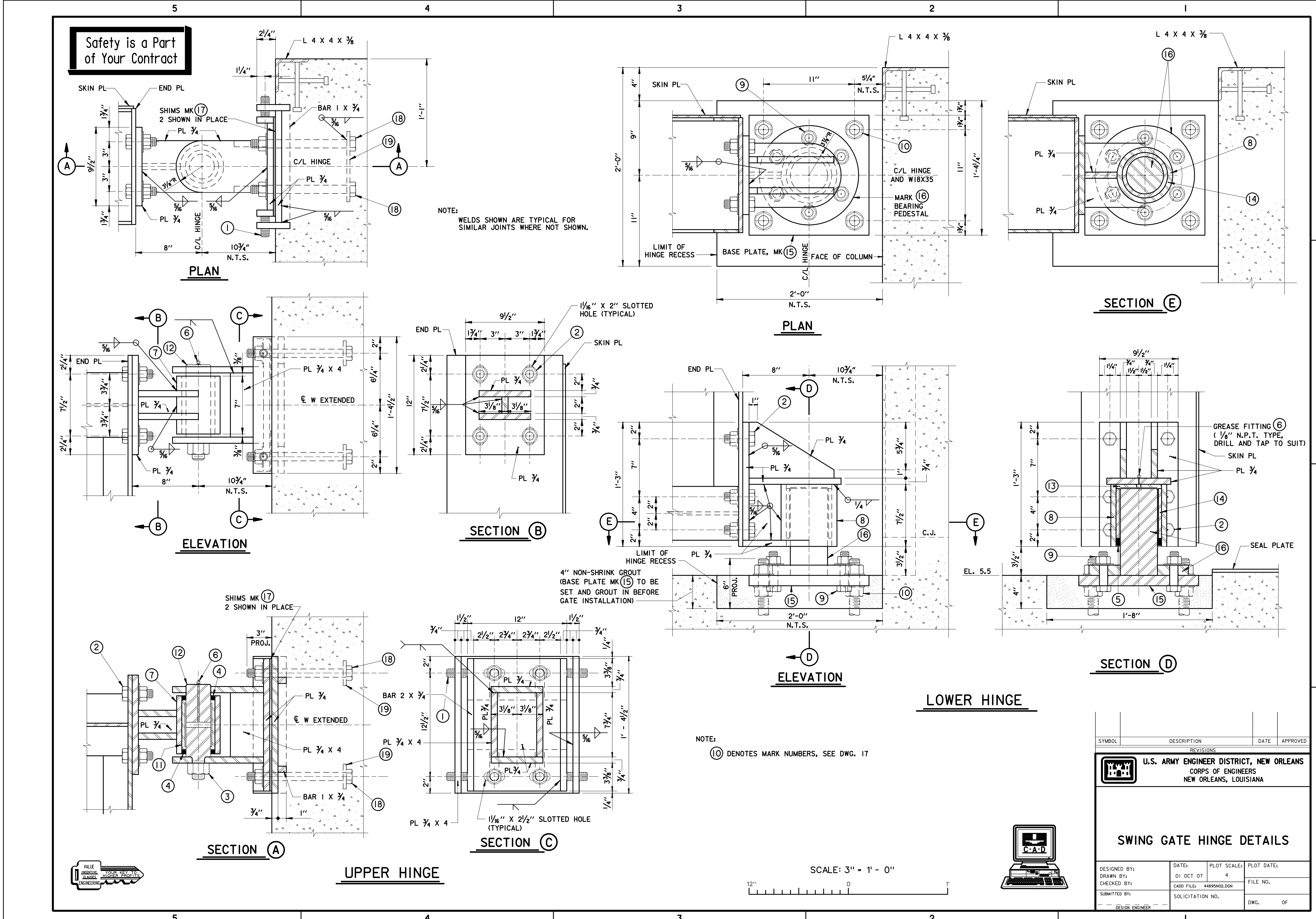
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

DESIGNED BY:
CHECKED BY:
DRAWN BY:
DATE: 1 OCT 07
PLOT SCALE: 1
PLOT DATE:
DESIGN FILE NAME: AETYPDRJ.DGN
SUBMITTED BY:
SOLICITATION NO. DACW29-

TYPICAL JOINT DETAILS

FILE NUMBER

DWG. OF


Safety is a Part of Your Contract
PLAN
ELEVATION
SECTION A
SECTION B
SECTION C
UPPER HINGE
PLAN
SECTION E
ELEVATION
SECTION D
LOWER HINGE
NOTE:
WELDS SHOWN ARE TYPICAL FOR SIMILAR JOINTS WHERE NOT SHOWN.
NOTE:
(10) DENOTES MARK NUMBERS, SEE DWG. 17
SKIN PL
END PL
SHIMS MK (17) 2 SHOWN IN PLACE
L 4 X 4 X 3/8
BAR 1 X 3/4
BAR 2 X 3/4
C/L HINGE
C/L HINGE AND W18X35
MARK (16) BEARING PEDESTAL
BASE PLATE, MK (15)
FACE OF COLUMN
LIMIT OF HINGE RECESS
4" NON-SHRINK GROUT (BASE PLATE MK (15) TO BE SET AND GROUT IN BEFORE GATE INSTALLATION)
GREASE FITTING (6) (1/8" N.P.T. TYPE, DRILL AND TAP TO SUIT)
SEAL PLATE
EL. 5.5
C.J.
1 1/16" X 2" SLOTTED HOLE (TYPICAL)
1 1/16" X 2 1/2" SLOTTED HOLE (TYPICAL)
℄ W EXTENDED
PL 3/4 X 4
N.T.S.
SCALE: 3" = 1' - 0"
SYMBOL
DESCRIPTION
DATE
APPROVED
REVISIONS
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
SWING GATE HINGE DETAILS
DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
DESIGN ENGINEER
DATE:
01 OCT 07
PLOT SCALE:
4
PLOT DATE:
CADD FILE: 44895N02.DGN
FILE NO.
SOLICITATION NO.
DWG. OF
VALUE INCENTIVE CLAUSES ENGINEERING
YOUR KEY TO HIGHER PROFITS

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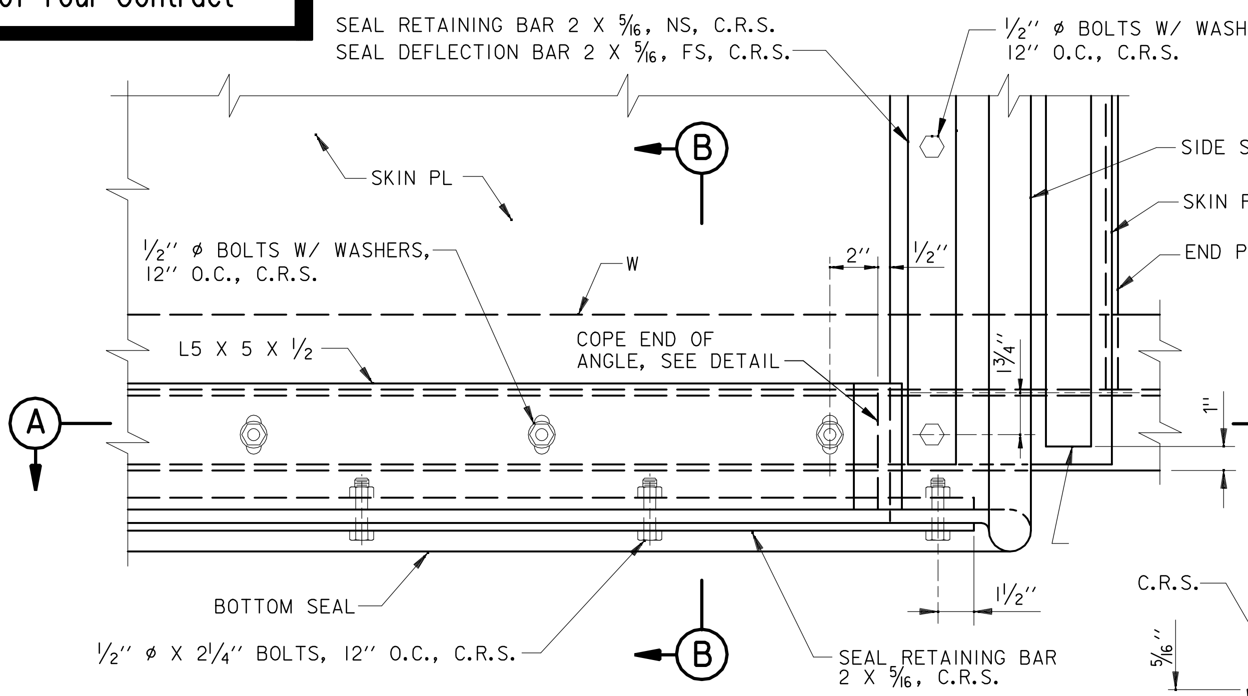
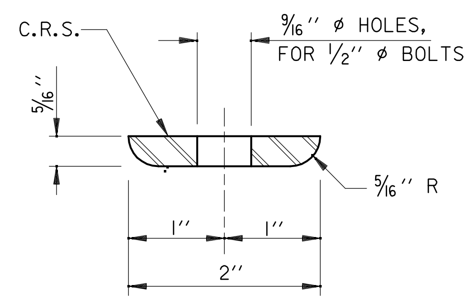


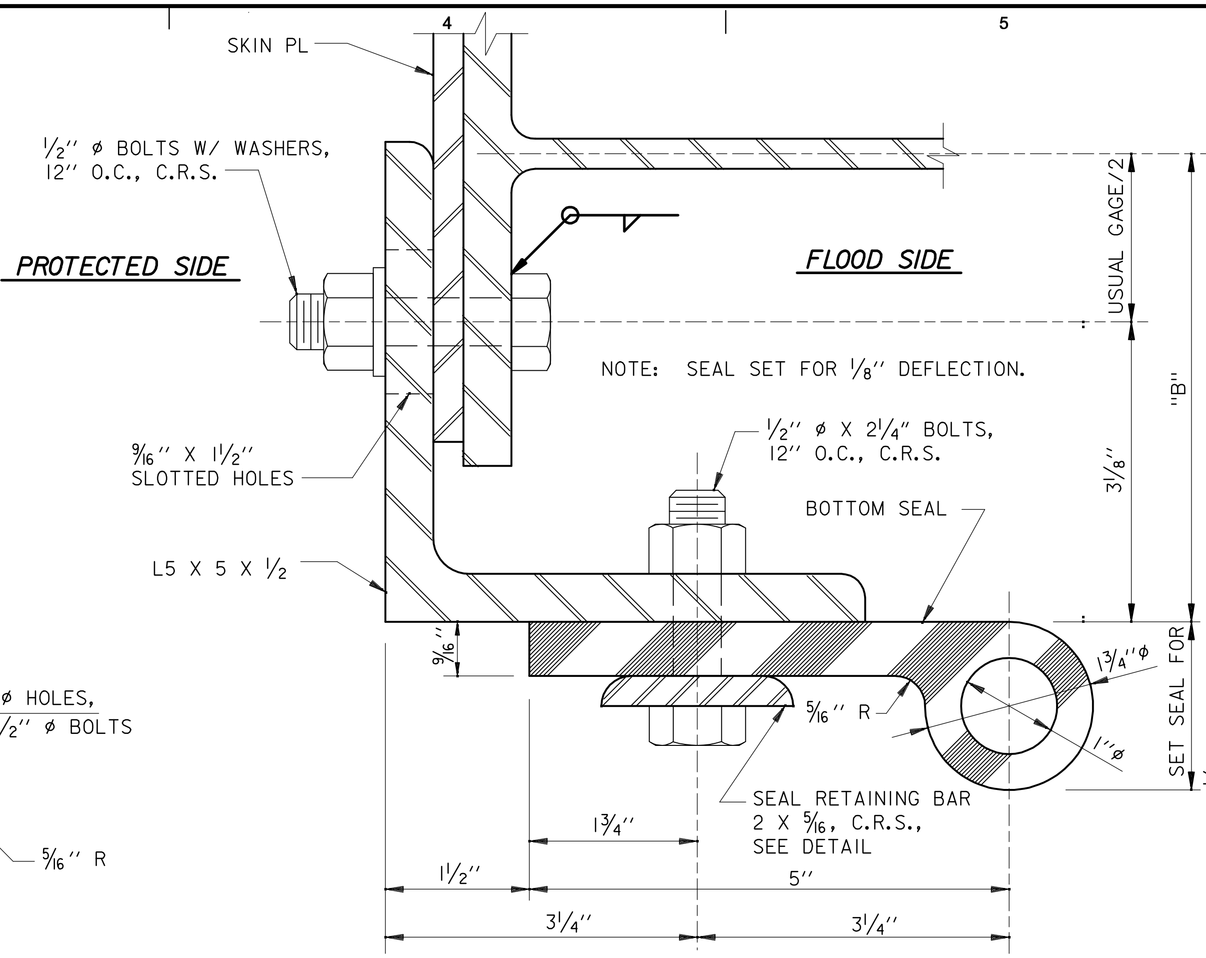
END ELEVATION - PROTECTED SIDE

SCALE: 3" = 1' - 0"



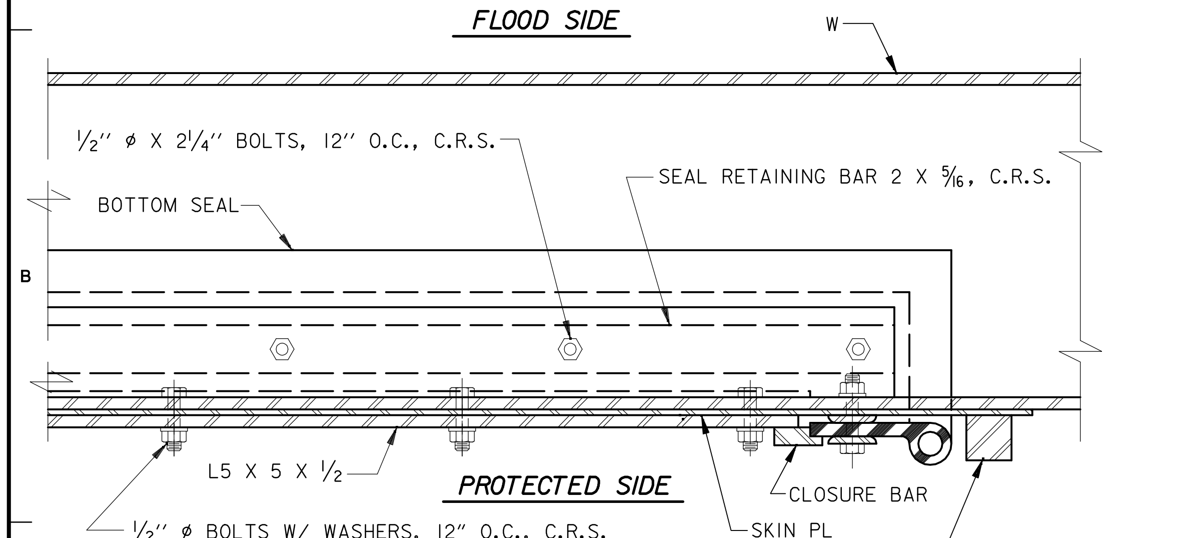
SEAL RETAINING BAR

SCALE: 12" = 1' - 0"



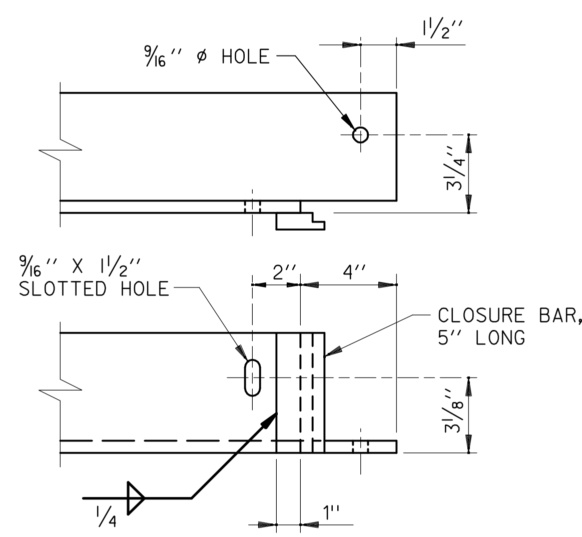
SECTION B

SCALE: 12" = 1' - 0"



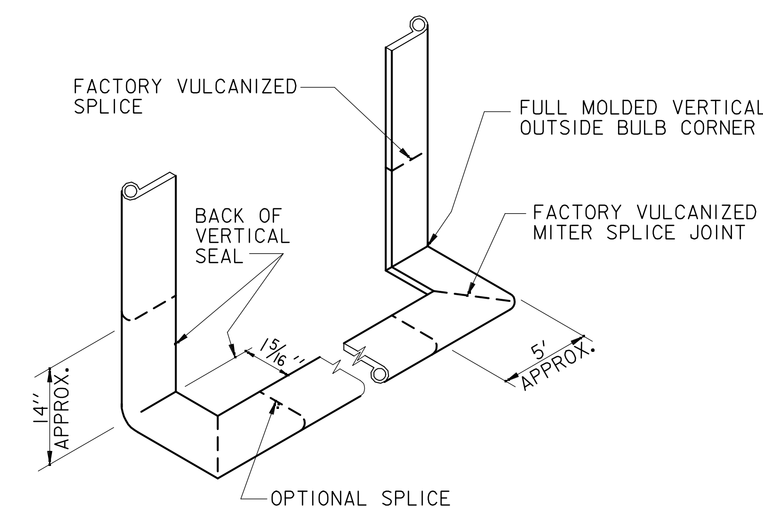
SECTION A

SCALE: 3" = 1' - 0"



END OF L 5 X 5 X 1/2

SCALE: 3" = 1' - 0"



CORNER DETAIL, TYPICAL BOTH SIDES

TYPICAL GATE SEAL

NOT TO SCALE

NOTES:

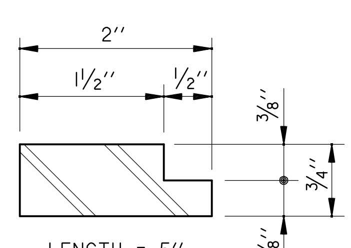
FOR GENERAL NOTES, SEE DWG. X.

ALL SPLICES WILL BE FACTORY MADE IN HEAVY STEEL PRESS TYPE MOLDS UNDER PRESSURE AND HEAT.

ALL SPLICE JOINTS MUST DEVELOP STRENGTH OF AT LEAST 50% OF THE MINIMUM TENSILE STRENGTH REQUIRED OF THE RUBBER.

SEAL CLAMP ANGLES SHALL BE PAINTED ON ALL SIDES PRIOR TO ASSEMBLY.

AFTER ASSEMBLY AND SEAL ADJUSTMENTS ARE MADE, ALL GAPS IN SEALS AND SEAL SUPPORTS SHALL BE SEALED WITH A SILICONE RUBBER SEALANT TO PROVIDE WATERTIGHT JOINTS.



CLOSURE BAR

SCALE: 12" = 1' - 0"

SCALE: 3" = 1' - 0"

12" 0 1'

SCALE: 12" = 1' - 0"

6" 3" 0

VALUE INCENTIVE CLAUSES ENGINEERING — YOUR KEY TO HIGHER PROFITS

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

MARK	DESCRIPTION	DATE	APPR

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

DESIGNED BY: | DATE: 1 Oct 07
DWN BY: | CKD BY: | SOLICITATION NO.:
SUBMITTED BY: | CONTRACT NO.:
FILE NAME: AETYP03RX.DGN | FILE NUMBER:
SIZE: | PLOT SCALE: 1 | PLOT DATE:

SWING GATE SEAL DETAILS

SHEET IDENTIFICATION NUMBER

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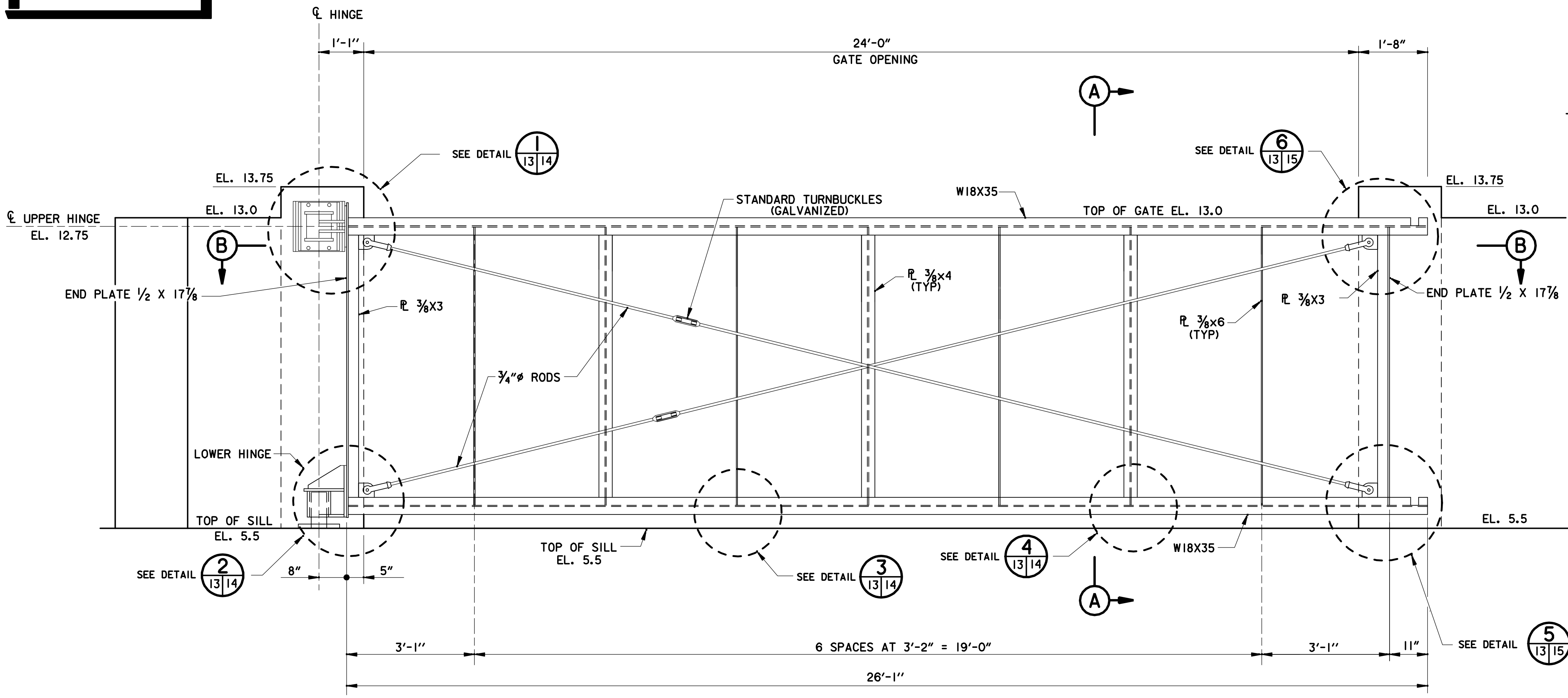
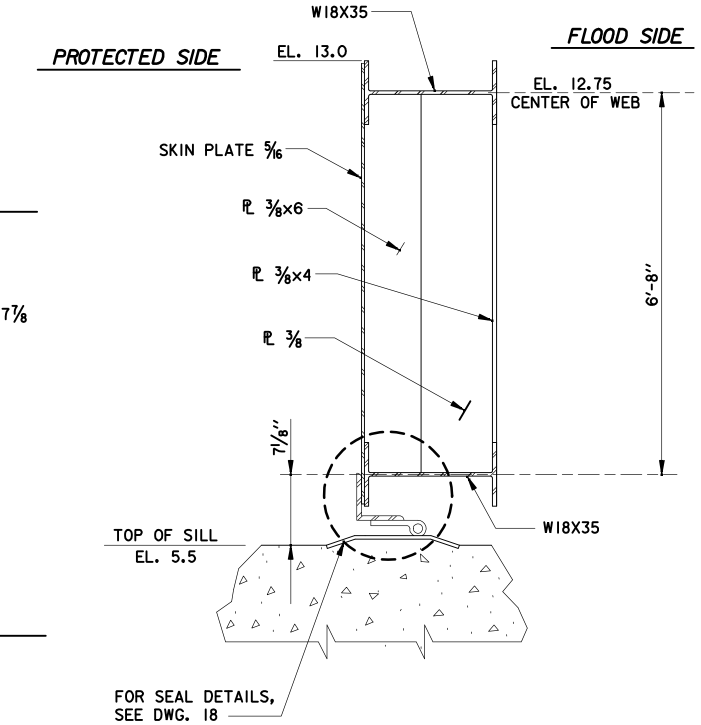


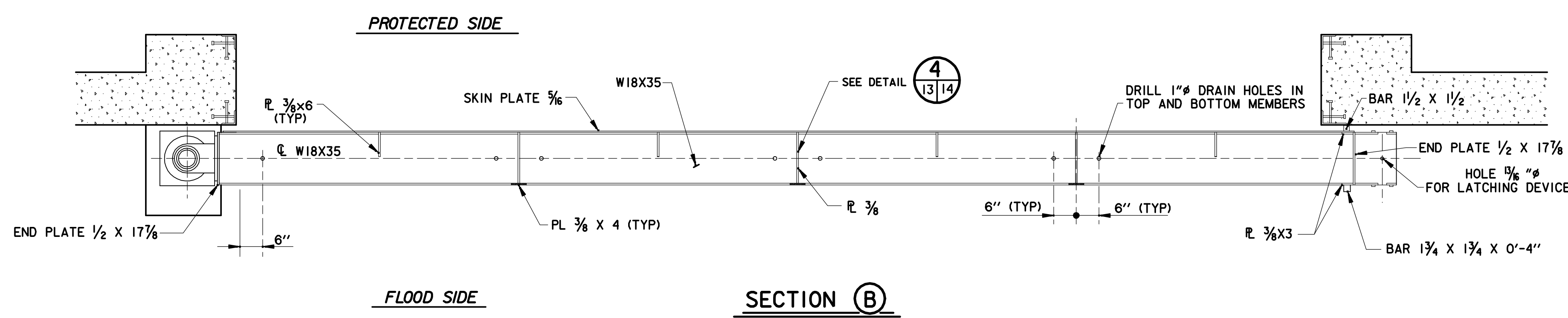
FLOOD SIDE ELEVATION

SCALE: 3/4" = 1'- 0"



SECTION A

N.T.S.



SECTION B

SCALE: 3/4" = 1'- 0"

SCALE: 3/4" = 1'- 0"

12" 0 1' 2' 3' 4' 5'

SYMBOL	DESCRIPTION	DATE	APPROVED

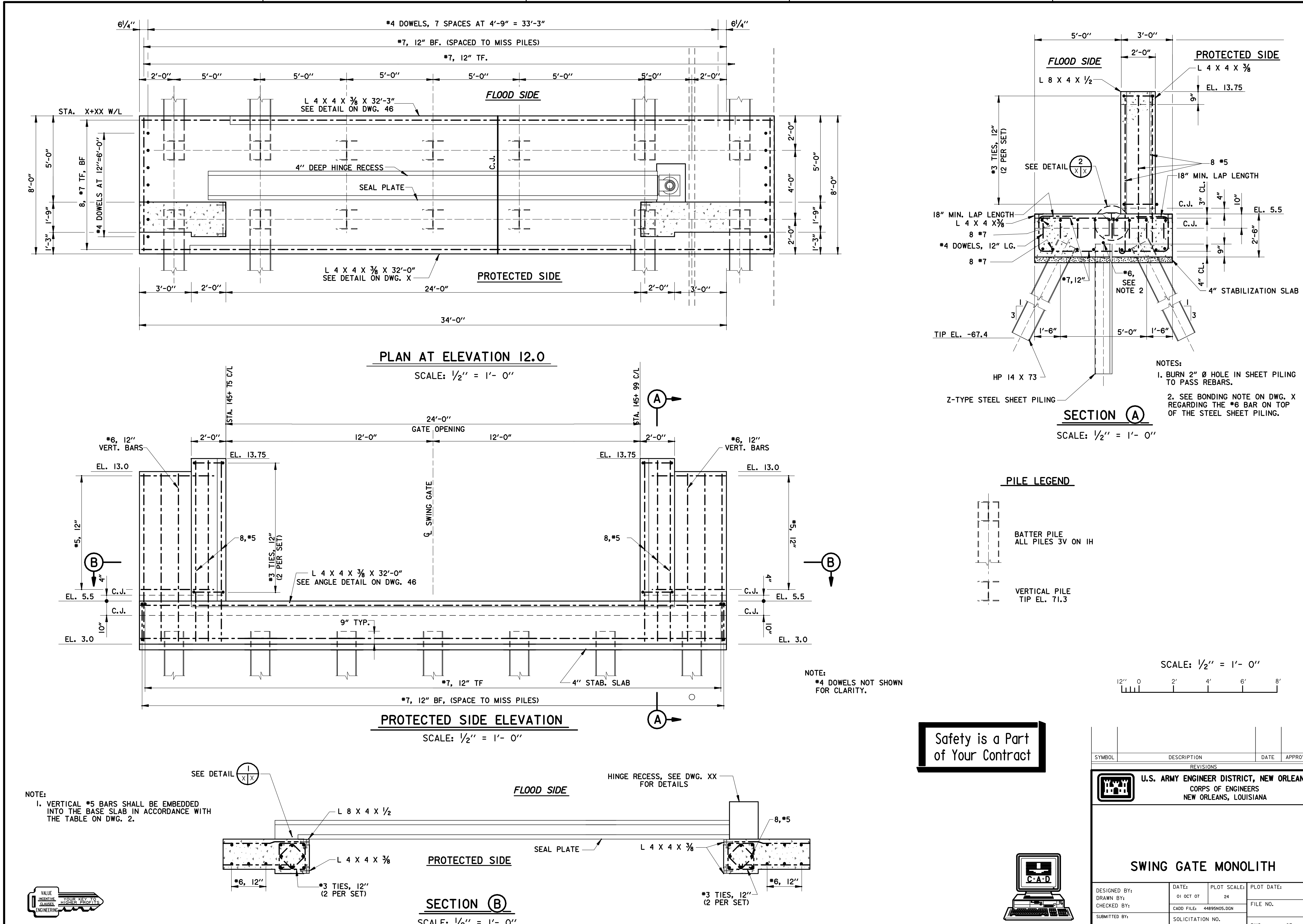
REVISIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

SWING GATE

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
DESIGN ENGINEER

DATE: 1 OCT 07
PLOT SCALE: 16
PLOT DATE:
CADD FILE: 44895N01.DGN
FILE NO.
SOLICITATION NO.
DWG. OF



SYMBOL	DESCRIPTION	DATE	APPROVED

REVISIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

SWING GATE MONOLITH

DESIGNED BY:
DRAWN BY:
CHECKED BY:
DATE: 01 OCT 07
PLOT SCALE: 24
PLOT DATE:
CADD FILE: 44895N05.DGN
FILE NO.
SUBMITTED BY:
DESIGN ENGINEER
SOLICITATION NO.
DWG. OF

5 | 4 | 3 | 2 | 1

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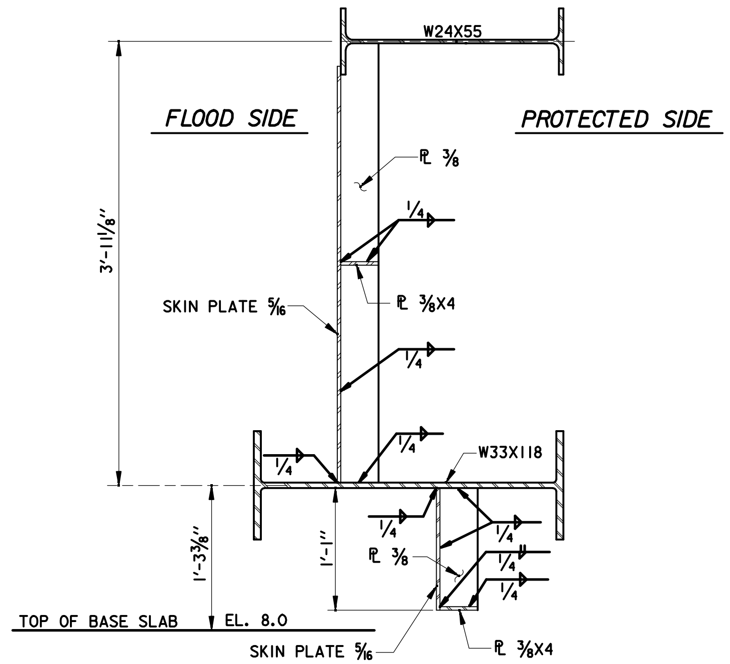
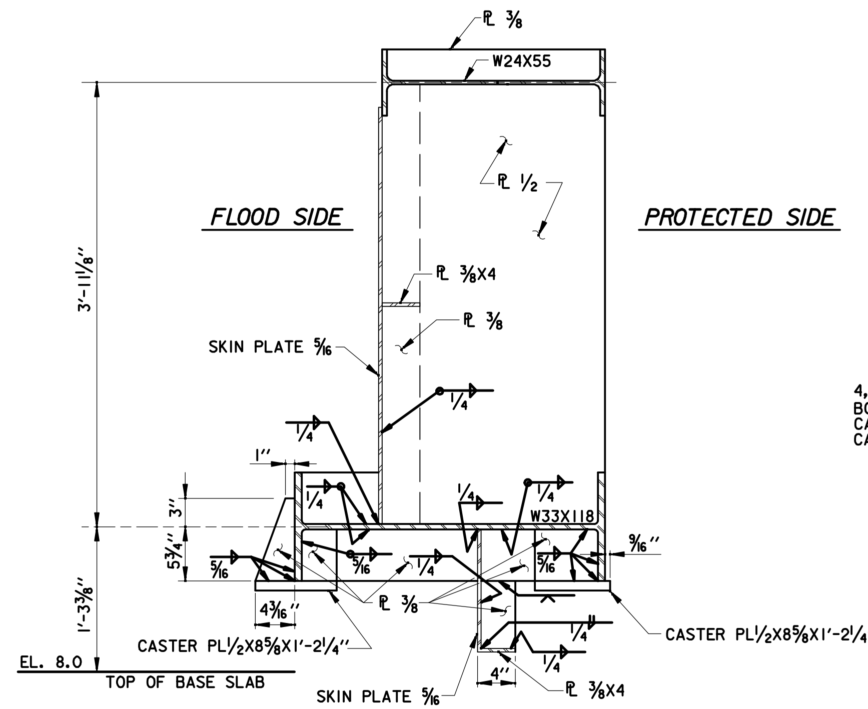
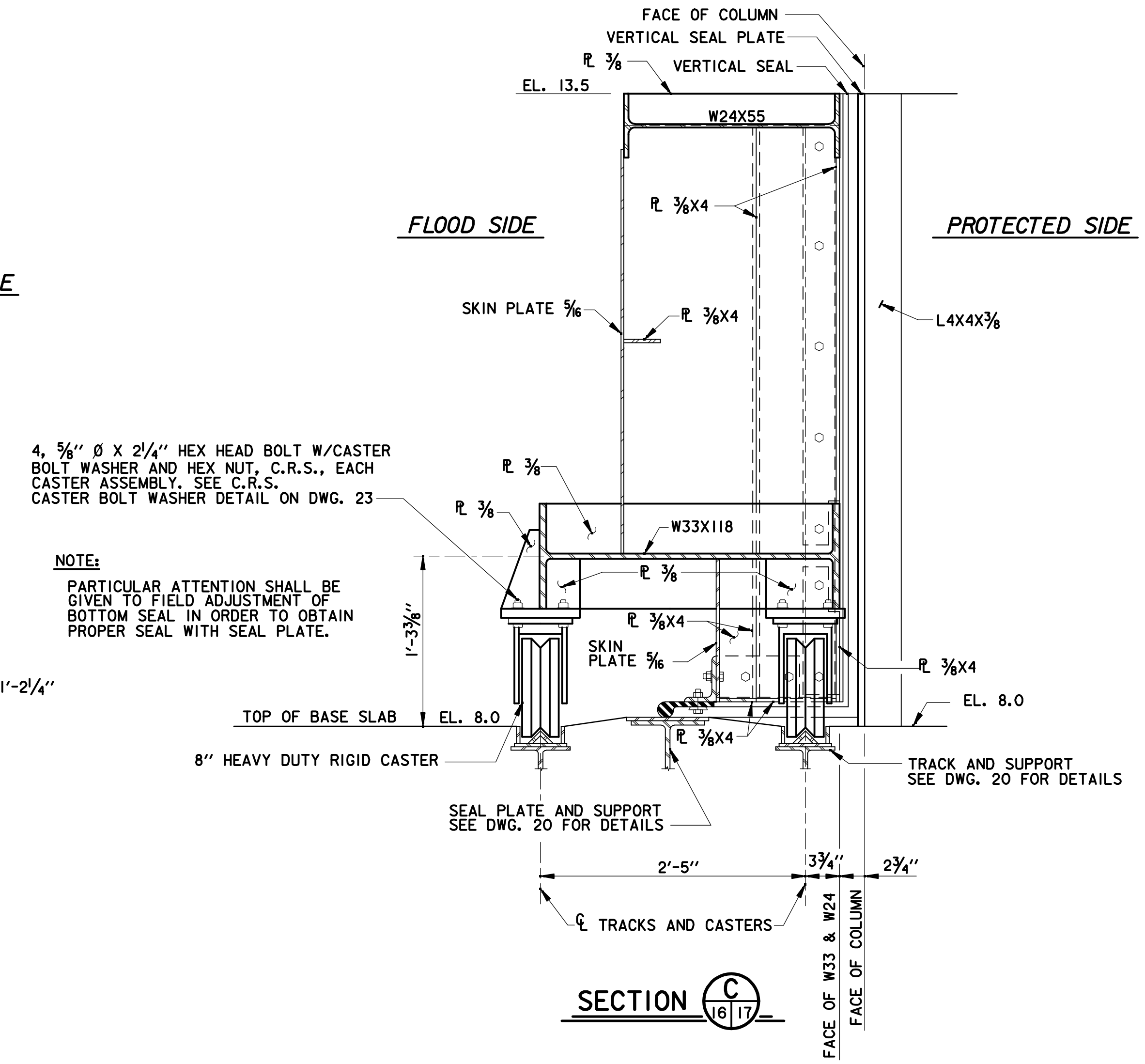


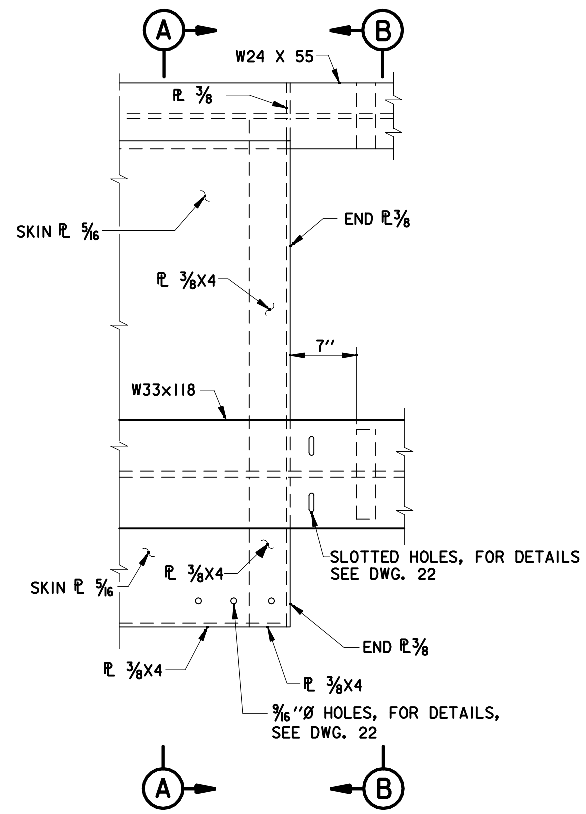
SECTION A 16|17



SECTION B 16|17

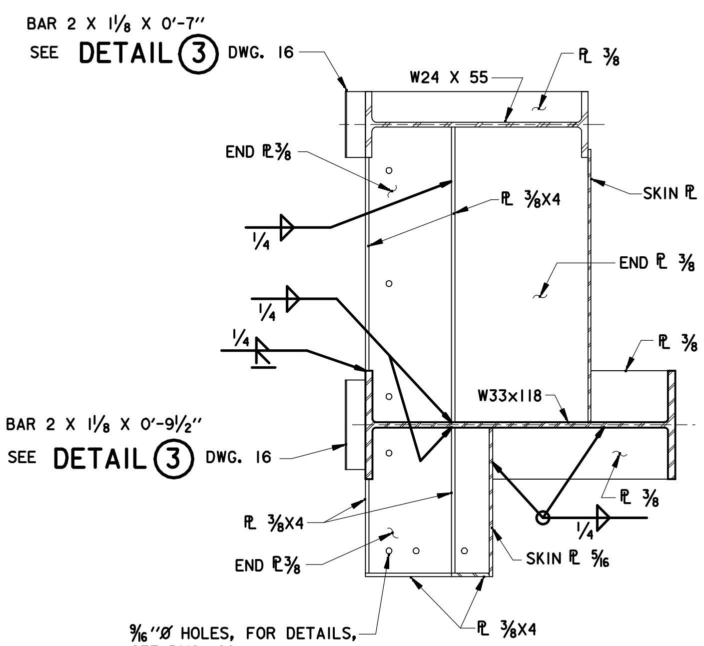


SECTION C 16|17

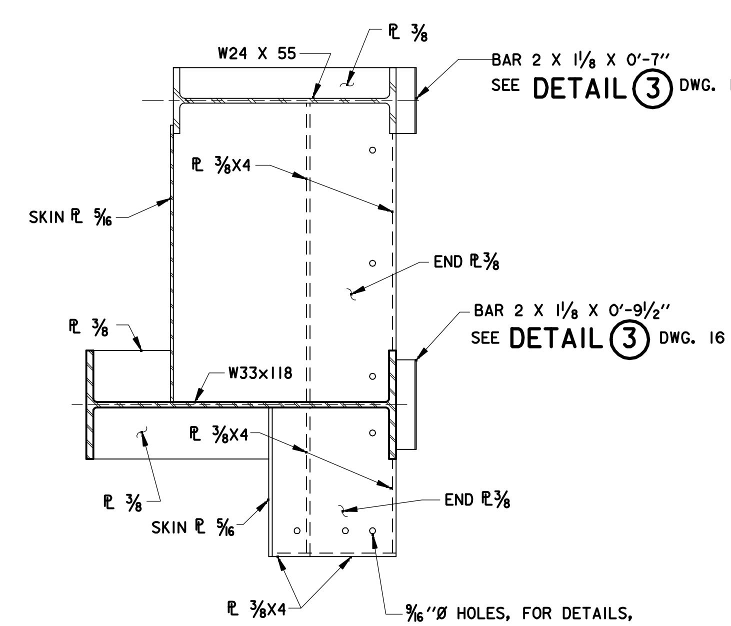


OTHER END-OPPOSITE HAND

FLOOD SIDE ELEVATION AT END OF GATE



SECTION A



SECTION B

REFERENCE DRAWINGS

FOR BOTTOM ROLLER GATE SEAL DETAILS, SEE DWG. 21.
FOR CASTER ASSEMBLY AND DETAILS, SEE DWG. 23.

SCALE: 1 ½" = 1' - 0"

12" 0 1' 2' 3'

SYMBOL	DESCRIPTION	DATE	APPROVED

REVISIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

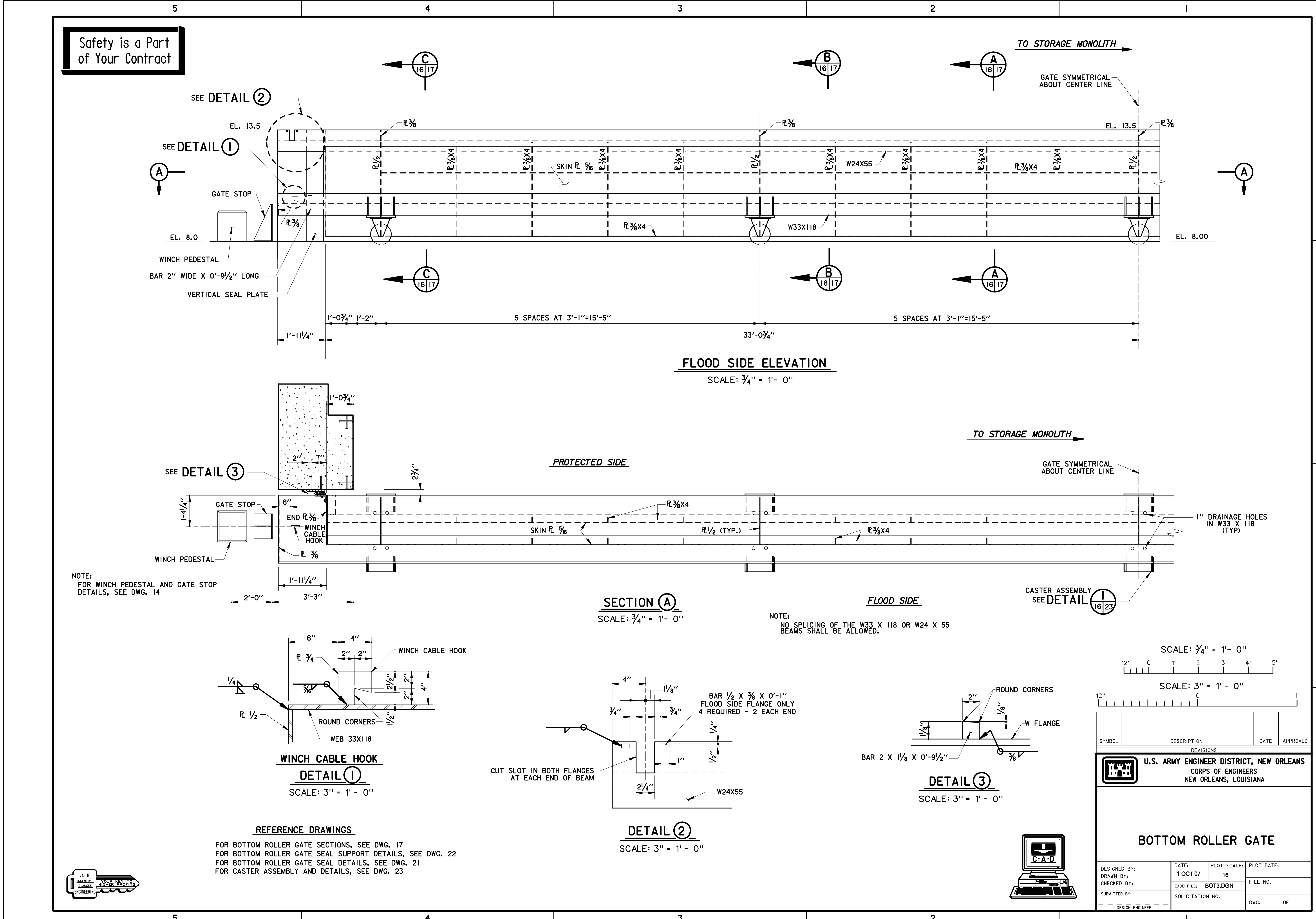
BOTTOM ROLLER GATE SECTIONS

DESIGNED BY: DRAWN BY: CHECKED BY:	DATE: 1 OCT 07	PLOT SCALE: 1	PLOT DATE:
	CADD FILE: BOT1.DGN		FILE NO.
SUBMITTED BY: DESIGN ENGINEER	SOLICITATION NO.		DWG. OF

VALUE INCENTIVE CLAUSES ENGINEERING — YOUR KEY TO HIGHER PROFITS

C·A·D

5 | 4 | 3 | 2 | 1


Safety is a Part of Your Contract
SEE DETAIL 2
SEE DETAIL 1
EL. 13.5
GATE STOP
WINCH PEDESTAL
BAR 2" WIDE X 0'-9½" LONG
VERTICAL SEAL PLATE
EL. 8.0
EL. 8.00
TO STORAGE MONOLITH
GATE SYMMETRICAL ABOUT CENTER LINE
SKIN ℞ 5⁄16
W24X55
℞⅜X4
W33X118
℞½
℞⅜
1'-0¾"
1'-2"
5 SPACES AT 3'-1"=15'-5"
1'-11¼"
33'-0¾"
FLOOD SIDE ELEVATION
SCALE: ¾" = 1'- 0"
PROTECTED SIDE
SEE DETAIL 3
END ℞⅜
WINCH CABLE HOOK
℞½ (TYP.)
1" DRAINAGE HOLES IN W33 X 118 (TYP)
NOTE:
FOR WINCH PEDESTAL AND GATE STOP DETAILS, SEE DWG. 14
2'-0"
3'-3"
SECTION A
SCALE: ¾" = 1'- 0"
FLOOD SIDE
CASTER ASSEMBLY SEE DETAIL 1 16 23
NOTE:
NO SPLICING OF THE W33 X 118 OR W24 X 55 BEAMS SHALL BE ALLOWED.
ROUND CORNERS
WEB 33X118
WINCH CABLE HOOK
DETAIL 1
SCALE: 3" = 1' - 0"
BAR ½ X ⅜ X 0'-1" FLOOD SIDE FLANGE ONLY 4 REQUIRED - 2 EACH END
CUT SLOT IN BOTH FLANGES AT EACH END OF BEAM
W24X55
DETAIL 2
SCALE: 3" = 1' - 0"
ROUND CORNERS
W FLANGE
BAR 2 X 1⅛ X 0'-9½"
DETAIL 3
SCALE: 3" = 1' - 0"
REFERENCE DRAWINGS
FOR BOTTOM ROLLER GATE SECTIONS, SEE DWG. 17
FOR BOTTOM ROLLER GATE SEAL SUPPORT DETAILS, SEE DWG. 22
FOR BOTTOM ROLLER GATE SEAL DETAILS, SEE DWG. 21
FOR CASTER ASSEMBLY AND DETAILS, SEE DWG. 23
SCALE: ¾" = 1'- 0"
SCALE: 3" = 1'- 0"
SYMBOL
DESCRIPTION
DATE
APPROVED
REVISIONS
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
BOTTOM ROLLER GATE
DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
DESIGN ENGINEER
DATE: 1 OCT 07
PLOT SCALE: 16
CADD FILE: BOT3.DGN
SOLICITATION NO.
PLOT DATE:
FILE NO.
DWG. OF
VALUE ENGINEERING INCENTIVE CLAUSES YOUR KEY TO HIGHER PROFITS

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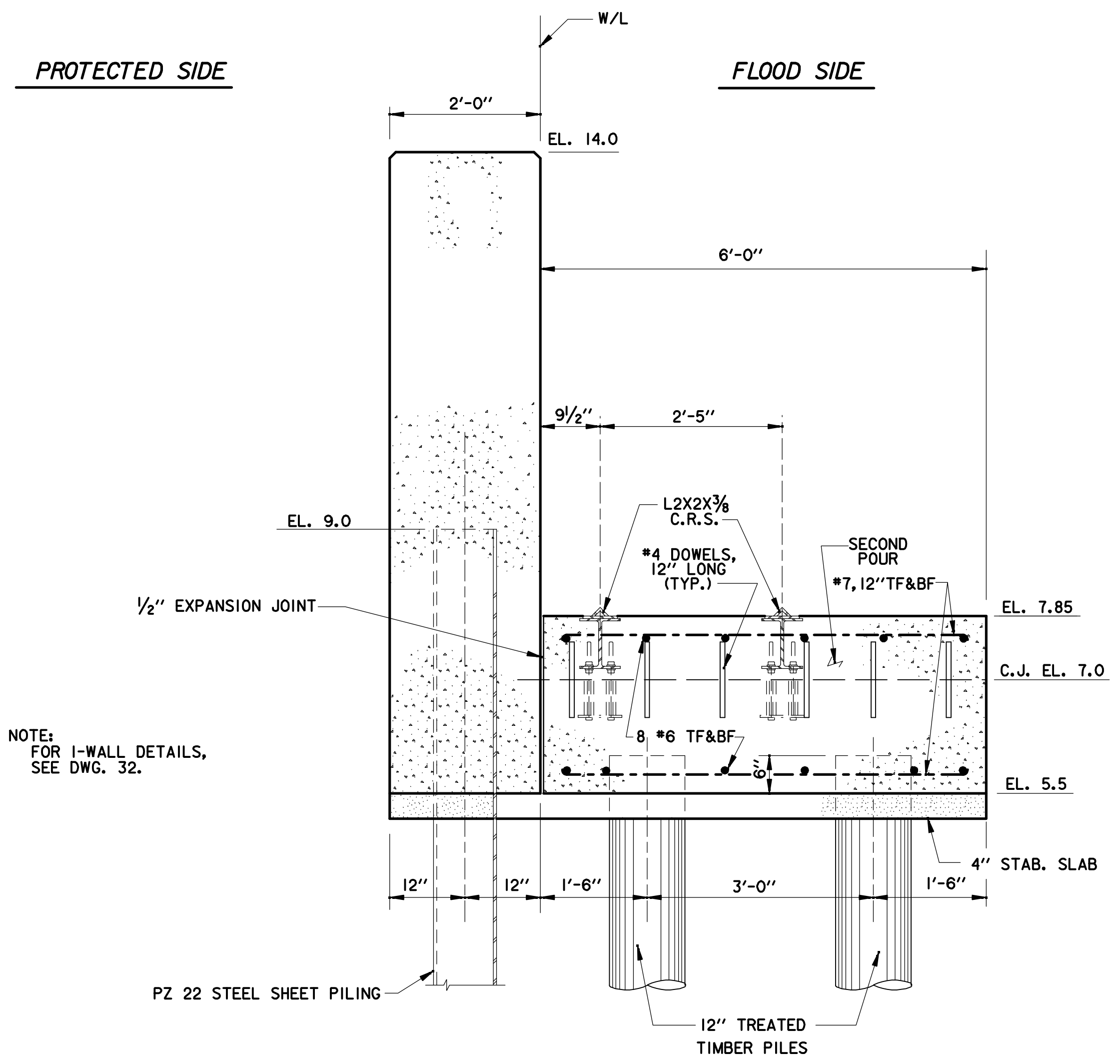
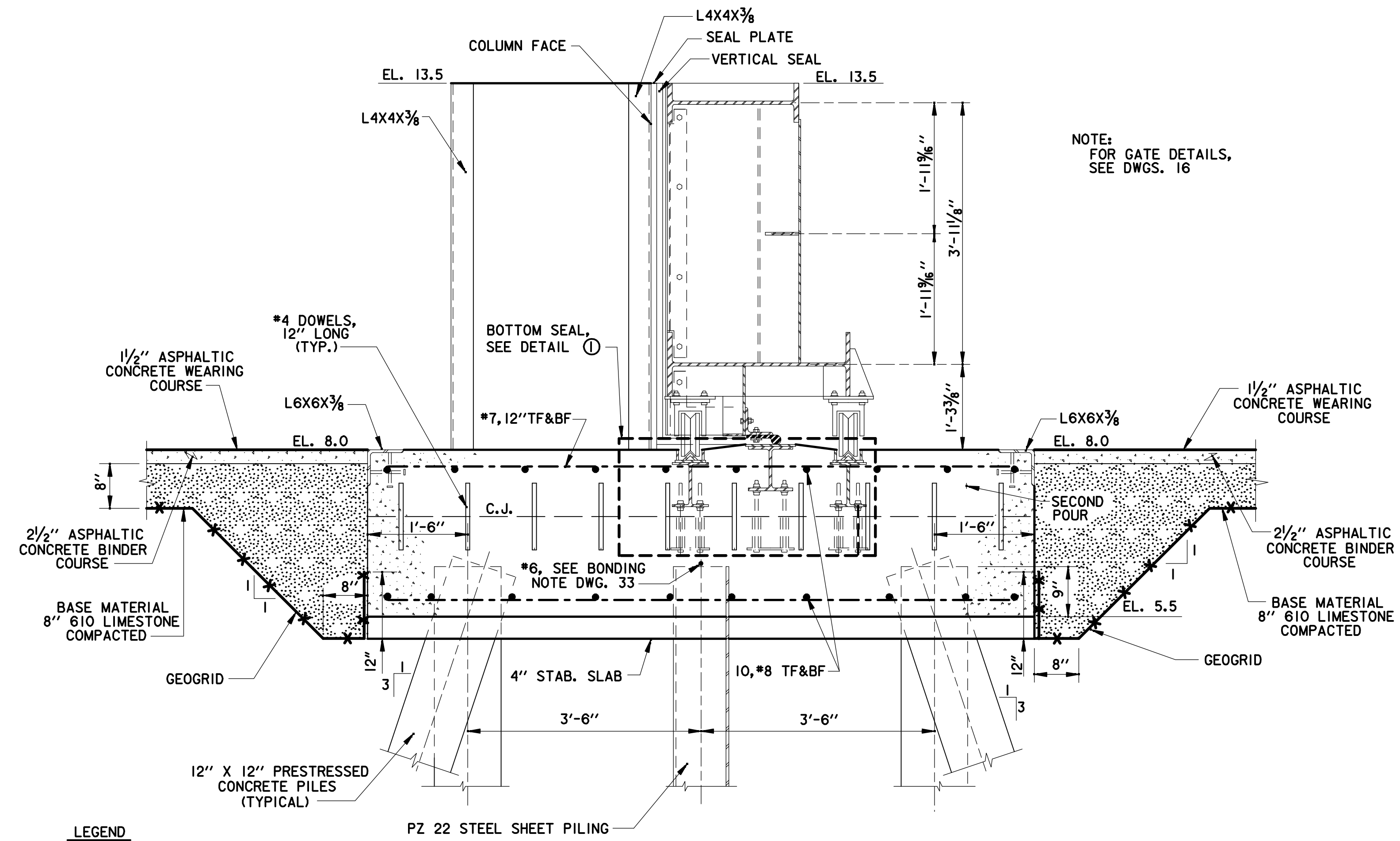


SECTION A 14 15

STORAGE MONOLITH

SCALE: 1" = 1'-0"



NOTE:
FOR GATE DETAILS, SEE DWGS. 16

LEGEND

GEOGRID —x—x—x—

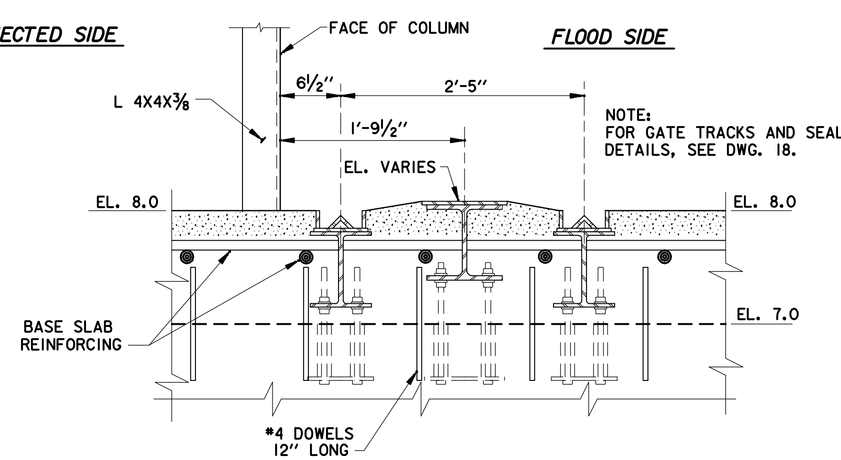
SECTION B 13 15

GATE MONOLITH

SCALE: 1" = 1'-0"

SCALE: 1 1/2" = 1' - 0"

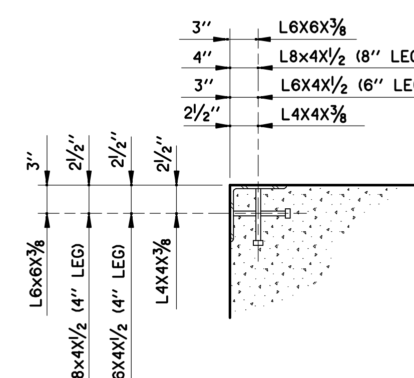
SCALE: 1" = 1' - 0"



NOTE:
FOR GATE TRACKS AND SEAL PLATE SUPPORT DETAILS, SEE DWG. 18.

DETAIL 1

SCALE: 1 1/2" = 1' - 0"



AT COLUMNS:
L4X4X3/8 W/1/2"X6" WELDED ANCHORS, 12" O.C. STAGGERED
L6X4X1/2 W/1/2"X6" WELDED ANCHORS, 12" O.C. STAGGERED
L8X4X1/2 W/1/2"X6" WELDED ANCHORS, 12" O.C. STAGGERED

AT TOP OF SILL:
L6X6X3/8 W/1/2"X6" WELDED ANCHORS, 12" O.C. STAGGERED

NOTE:
ENTIRE ANGLE ASSEMBLY TO BE HOT-DIP GALVANIZED AFTER FABRICATION.

ANGLE DETAIL

N.T.S.

SYMBOL	DESCRIPTION	DATE	APPROVED
	REVISIONS		

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

BOTTOM ROLLER GATE MONOLITH SECTIONS

DESIGNED BY: DRAWN BY: CHECKED BY:	DATE: 1 OCT 07	PLOT SCALE: 12	PLOT DATE:
	CADD FILE: BOT4.DGN		FILE NO.
SUBMITTED BY: DESIGN ENGINEER	SOLICITATION NO.		DWG. OF

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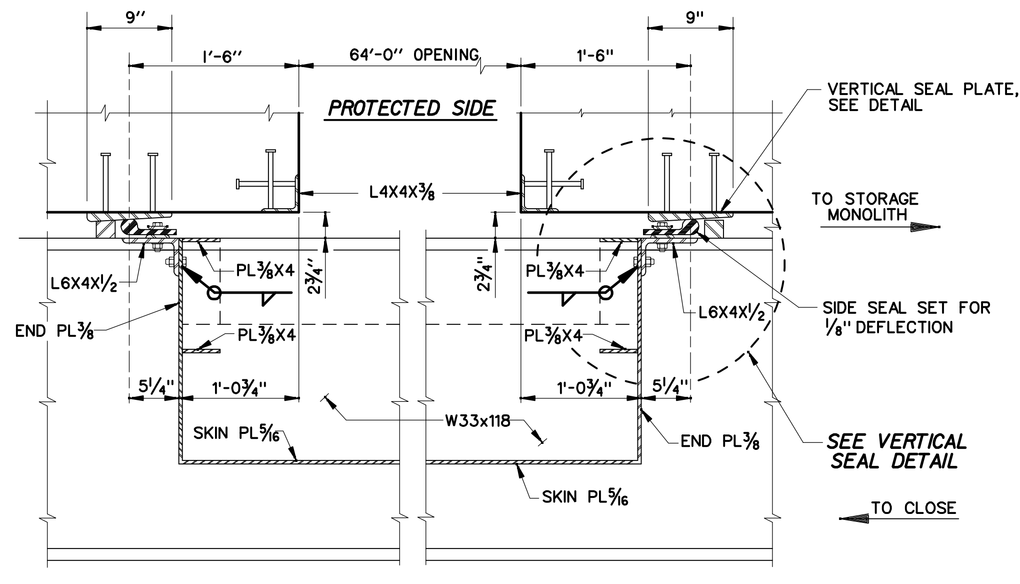
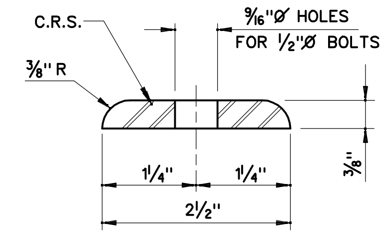


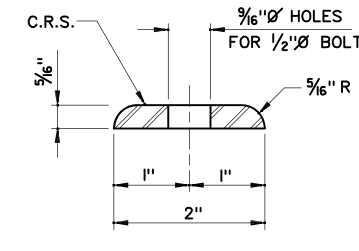
SECTION THRU ROLLER GATE

SCALE: 1 1/2" = 1' - 0"



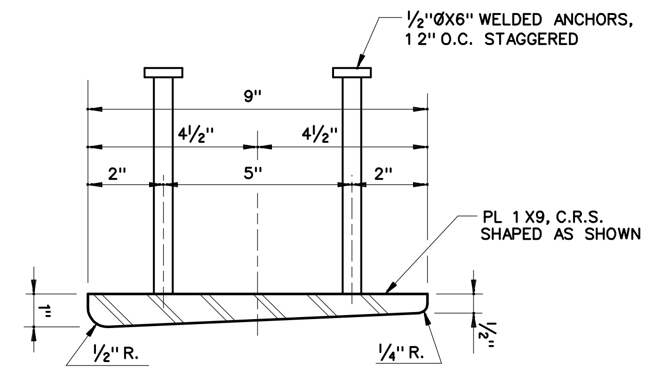
SEAL DEFLECTION BAR

SCALE: 12" = 1' - 0"



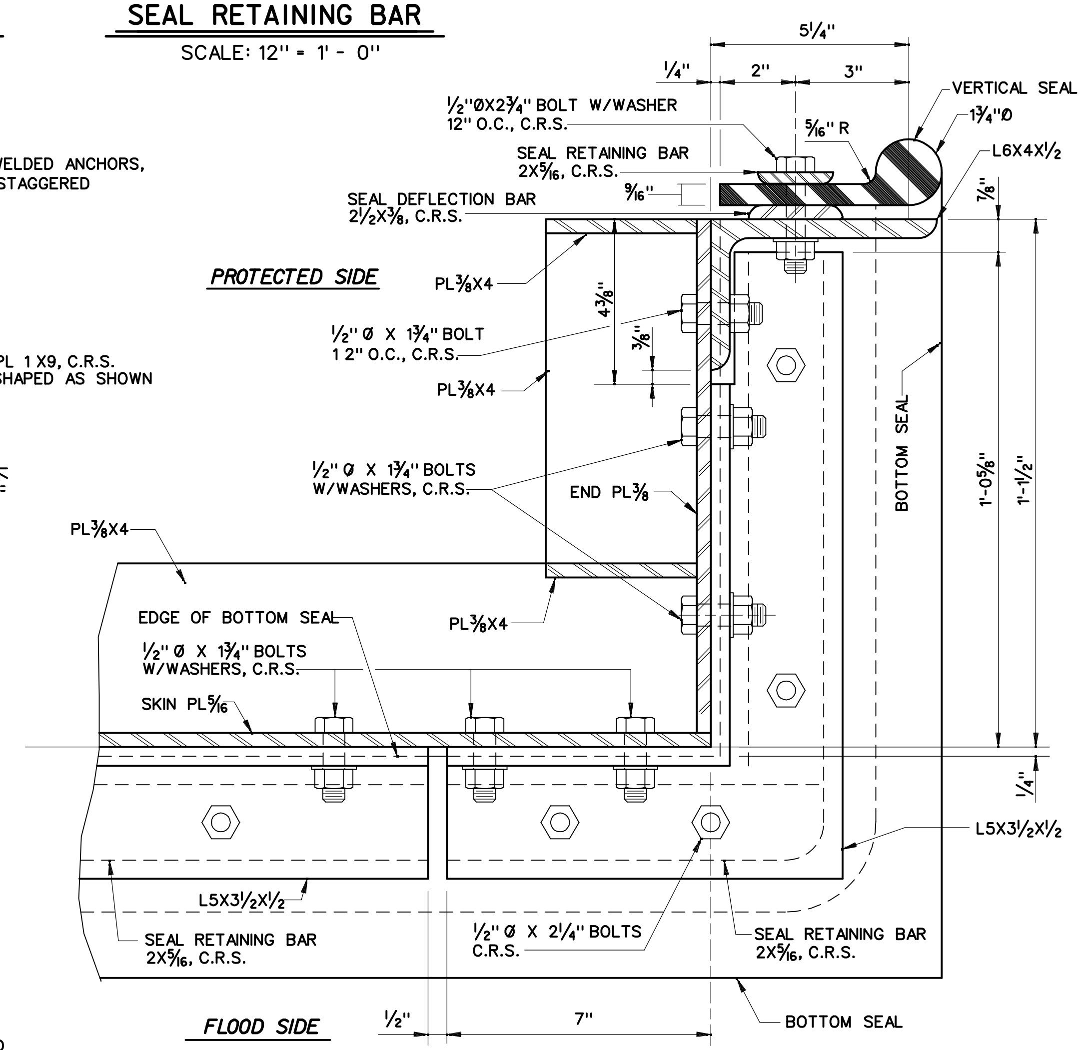
SEAL RETAINING BAR

SCALE: 12" = 1' - 0"



VERTICAL SEAL PLATE DETAIL

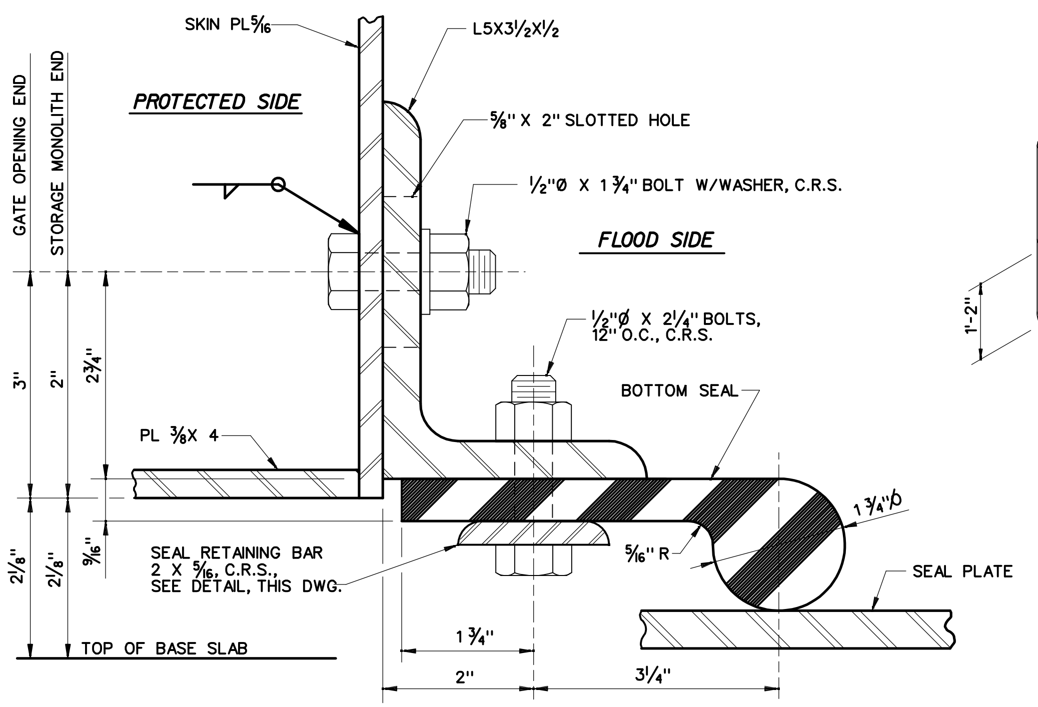
SCALE: 6" = 1' - 0"



VERTICAL SEAL

PLAN AT END OF GATE BELOW W33X118

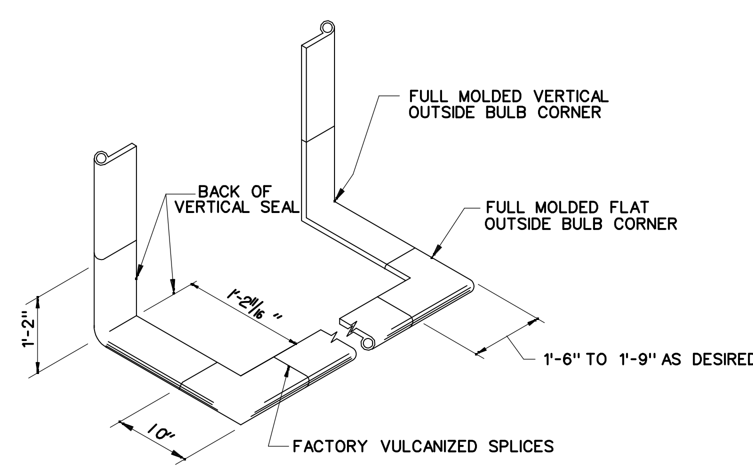
SCALE: 6" = 1' - 0"



TYPICAL SECTION THRU BOTTOM SEAL

SCALE: 12" = 1' - 0"

NOTE:
SET SEAL FOR 0" DEFLECTION. PARTICULAR ATTENTION SHALL BE GIVEN TO FIELD ADJUSTMENT OF BOTTOM SEAL IN ORDER TO OBTAIN A PERFECT SEAL WITH SEAL PLATE.



TYPICAL GATE SEAL

NOT TO SCALE

NOTE:

1. ALL INDIVIDUAL 90° CORNER INTERSECTIONS WILL BE FULLY MOLDED.
2. ALL SPLICES WILL BE FACTORY MADE IN HEAVY STEEL PRESS TYPE MOLDS UNDER PRESSURE AND HEAT.
3. ALL SPLICE JOINTS MUST DEVELOP A STRENGTH OF AT LEAST 50% OF THE MINIMUM TENSILE STRENGTH REQUIRED OF THE RUBBER.

REFERENCE DRAWINGS

FOR BOTTOM ROLLER GATE SEAL SUPPORT DETAILS, SEE DWG. 22

SCALE: 1 1/2" = 1' - 0"

12" 0 1' 2' 3'

SCALE: 6" = 1' - 0"

6" 0 6"

SCALE: 12" = 1' - 0"

6" 3" 0

SYMBOL	DESCRIPTION	DATE	APPROVED

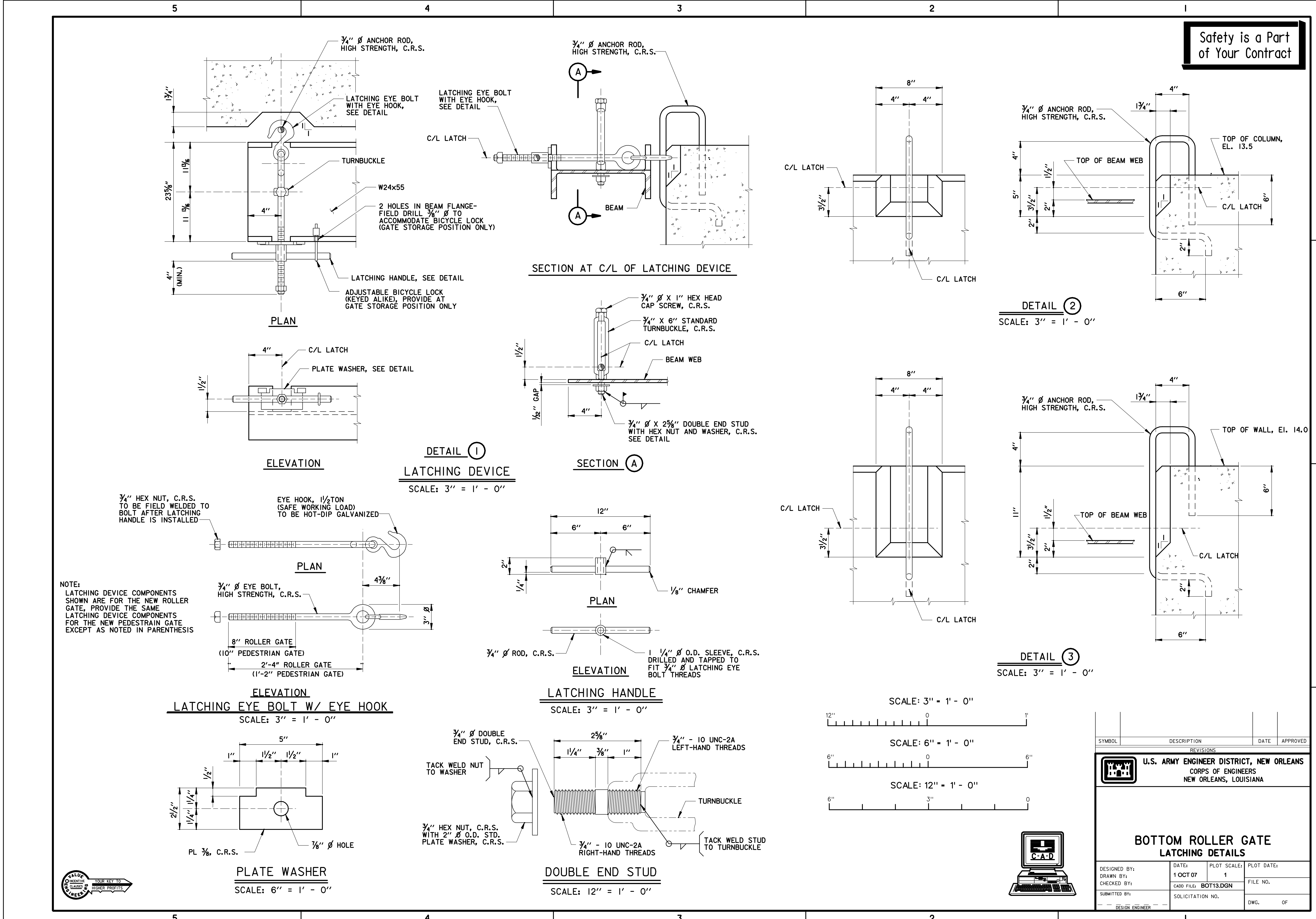
REVISIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

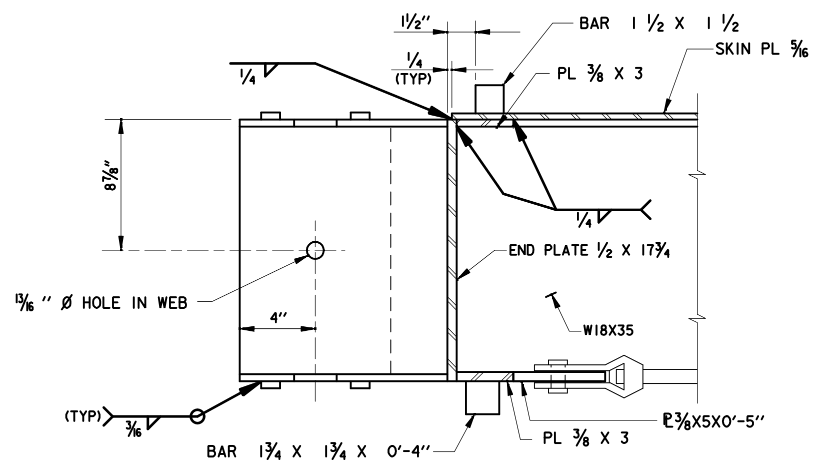
BOTTOM ROLLER GATE
SEAL DETAILS

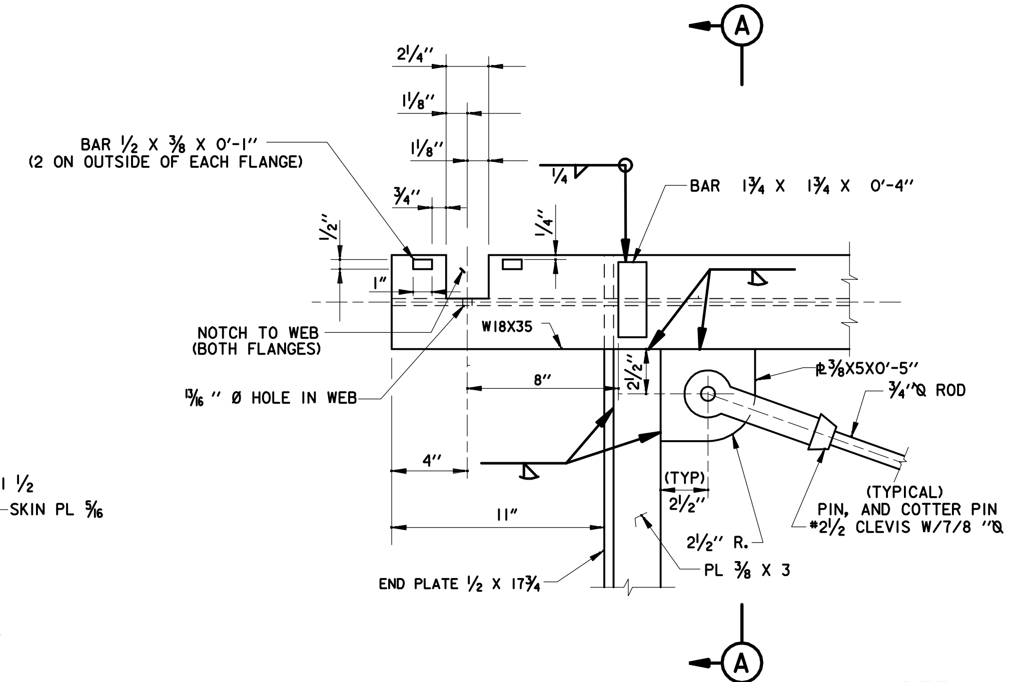
DESIGNED BY:	DATE: 1 OCT 07	PLOT SCALE: 2	PLOT DATE:
DRAWN BY:	CADD FILE: BOT5.DGN		FILE NO.
CHECKED BY:	SOLICITATION NO.		DWG. OF
SUBMITTED BY: DESIGN ENGINEER			

VALUE ENGINEERING INCENTIVE CLAUSES YOUR KEY TO HIGHER PROFITS

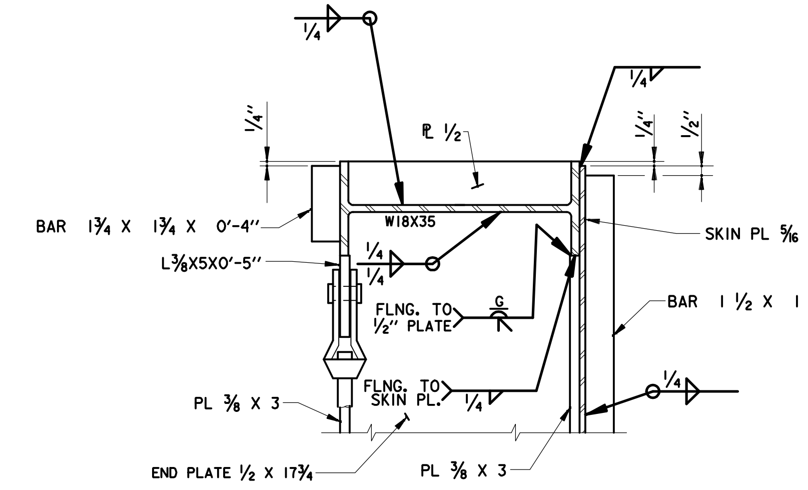

Safety is a Part of Your Contract
¾″ Ø ANCHOR ROD, HIGH STRENGTH, C.R.S.
LATCHING EYE BOLT WITH EYE HOOK, SEE DETAIL
TURNBUCKLE
W24x55
2 HOLES IN BEAM FLANGE- FIELD DRILL ⅜″ Ø TO ACCOMMODATE BICYCLE LOCK (GATE STORAGE POSITION ONLY)
LATCHING HANDLE, SEE DETAIL
ADJUSTABLE BICYCLE LOCK (KEYED ALIKE), PROVIDE AT GATE STORAGE POSITION ONLY
4″ (MIN.)
PLAN
C/L LATCH
PLATE WASHER, SEE DETAIL
ELEVATION
DETAIL 1
LATCHING DEVICE
SCALE: 3″ = 1′ - 0″
BEAM
SECTION AT C/L OF LATCHING DEVICE
¾″ Ø X 1″ HEX HEAD CAP SCREW, C.R.S.
¾″ X 6″ STANDARD TURNBUCKLE, C.R.S.
BEAM WEB
1/32″ GAP
¾″ Ø X 2⅝″ DOUBLE END STUD WITH HEX NUT AND WASHER, C.R.S. SEE DETAIL
SECTION A
¾″ HEX NUT, C.R.S. TO BE FIELD WELDED TO BOLT AFTER LATCHING HANDLE IS INSTALLED
EYE HOOK, 1½TON (SAFE WORKING LOAD) TO BE HOT-DIP GALVANIZED
PLAN
¾″ Ø EYE BOLT, HIGH STRENGTH, C.R.S.
8″ ROLLER GATE (10″ PEDESTRIAN GATE)
2′-4″ ROLLER GATE (1′-2″ PEDESTRIAN GATE)
ELEVATION
LATCHING EYE BOLT W/ EYE HOOK
SCALE: 3″ = 1′ - 0″
NOTE:
LATCHING DEVICE COMPONENTS SHOWN ARE FOR THE NEW ROLLER GATE, PROVIDE THE SAME LATCHING DEVICE COMPONENTS FOR THE NEW PEDESTRAIN GATE EXCEPT AS NOTED IN PARENTHESIS
⅛″ CHAMFER
PLAN
¾″ Ø ROD, C.R.S.
1 ¼″ Ø O.D. SLEEVE, C.R.S. DRILLED AND TAPPED TO FIT ¾″ Ø LATCHING EYE BOLT THREADS
ELEVATION
LATCHING HANDLE
SCALE: 3″ = 1′ - 0″
PL ⅜, C.R.S.
⅞″ Ø HOLE
PLATE WASHER
SCALE: 6″ = 1′ - 0″
¾″ Ø DOUBLE END STUD, C.R.S.
TACK WELD NUT TO WASHER
¾″ HEX NUT, C.R.S. WITH 2″ Ø O.D. STD. PLATE WASHER, C.R.S.
¾″ - 10 UNC-2A RIGHT-HAND THREADS
¾″ - 10 UNC-2A LEFT-HAND THREADS
TURNBUCKLE
TACK WELD STUD TO TURNBUCKLE
DOUBLE END STUD
SCALE: 12″ = 1′ - 0″
TOP OF BEAM WEB
TOP OF COLUMN, EL. 13.5
DETAIL 2
SCALE: 3″ = 1′ - 0″
TOP OF WALL, El. 14.0
DETAIL 3
SCALE: 3″ = 1′ - 0″
SCALE: 3″ = 1′ - 0″
SCALE: 6″ = 1′ - 0″
SCALE: 12″ = 1′ - 0″
SYMBOL
DESCRIPTION
DATE
APPROVED
REVISIONS
U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
BOTTOM ROLLER GATE
LATCHING DETAILS
DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
DESIGN ENGINEER
DATE: 1 OCT 07
PLOT SCALE: 1
PLOT DATE:
CADD FILE: BOT13.DGN
FILE NO.
SOLICITATION NO.
DWG. OF
VALUE ENGINEERING INCENTIVE CLAUSES YOUR KEY TO HIGHER PROFITS
C·A·D

Safety is a Part of Your Contract



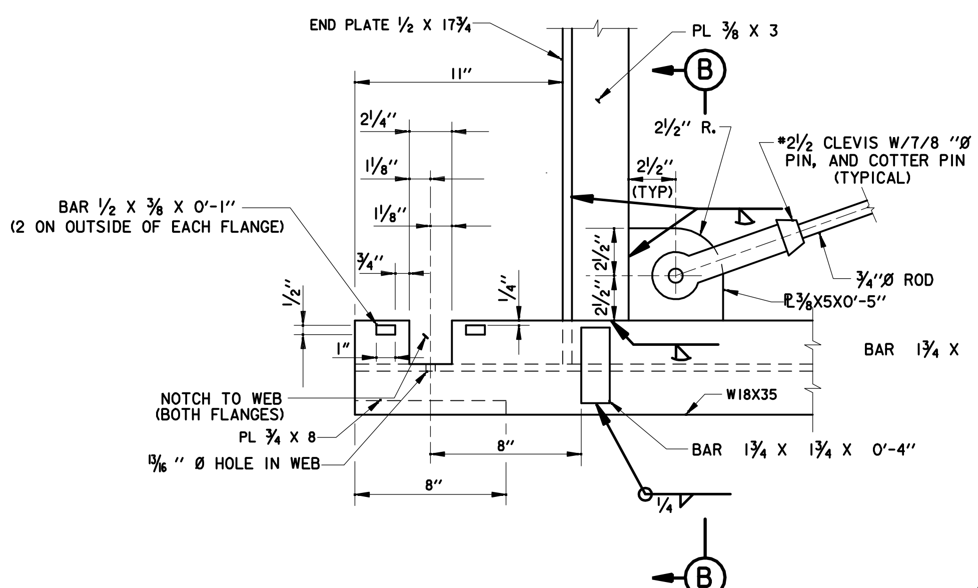


DETAIL 6 13|15

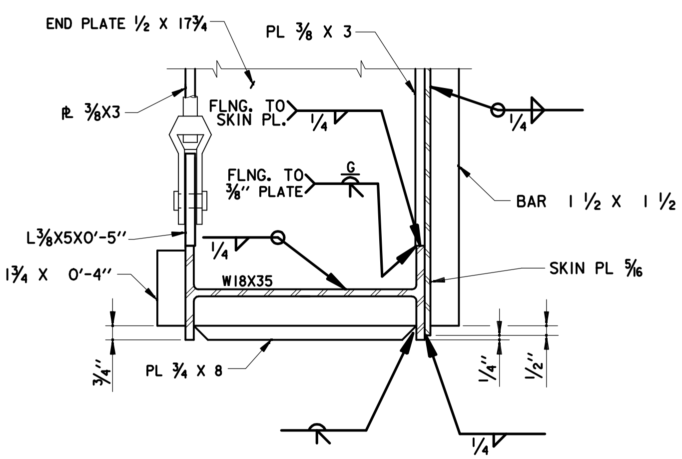


SECTION A

NOTE:
WELDS SHOWN ARE TYPICAL FOR SIMILAR JOINTS.



DETAIL 5 13|15



SECTION B

SCALE: 3" = 1' - 0"

12" 0 1'

SYMBOL	DESCRIPTION	DATE	APPROVED

REVISIONS

U.S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

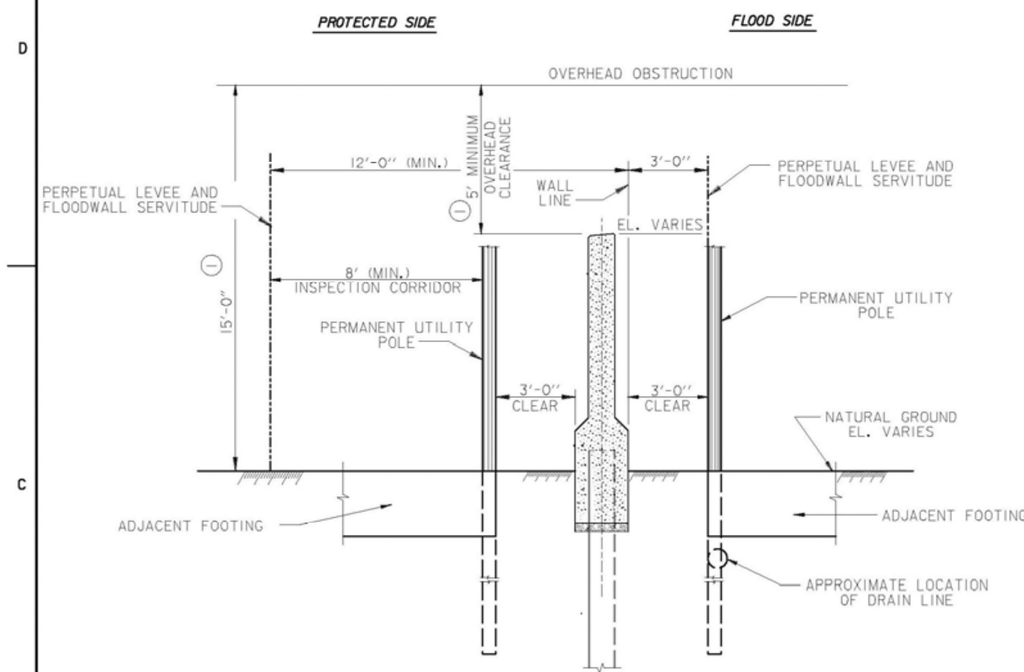
BOTTOM ROLLER GATE
DETAILS

SWING GATE
CORNER DETAILS

DESIGNED BY:
DRAWN BY:
CHECKED BY:
DATE: 01 OCT 07
PLOT SCALE: 4
PLOT DATE:
CADD FILE: 44895N03.DGN
FILE NO.
SUBMITTED BY:
DESIGN ENGINEER
SOLICITATION NO.
DWG. OF

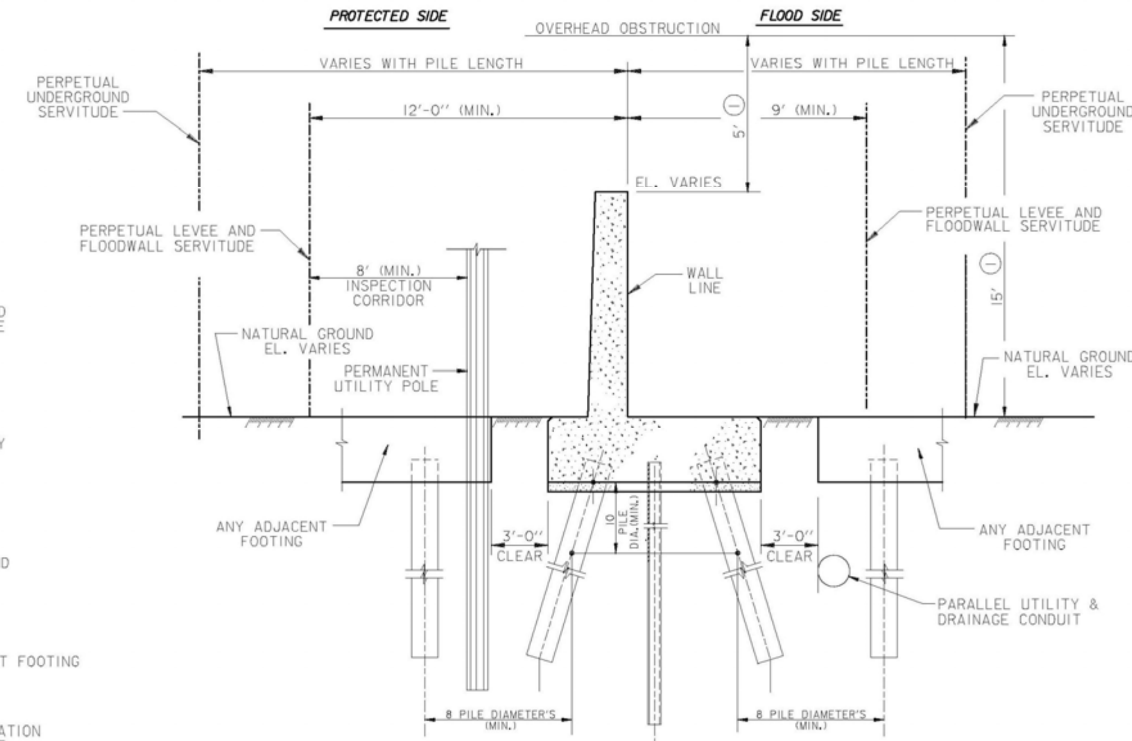
VALUE INCENTIVE CLAUSES ENGINEERING YOUR KEY TO HIGHER PROFITS

Safety is a Part of Your Contract



I-WALL CLEARANCE

SCALE: 1/2" = 1'- 0"



T-WALL CLEARANCE

SCALE: 1/2" = 1'- 0"

1. USE THE GREATER OF THE 5' CLEARANCE FROM THE TOP OF THE WALL OR THE 15' FROM THE NATURAL GROUND SURFACE. FOR ELECTRIC LINES CLEARANCE SHALL ALSO COMPLY WITH EM 385-1-1.
2. ALL POLES AND SUBSURFACE FOOTINGS PLACED WITHIN THE PERPETUAL UNDERGROUND SERVITUDE SHALL BE COORDINATED WITH A COE REPRESENTATIVE DURING DESIGN AND CONSTRUCTION SO AS TO AVOID INTERFERING WITH THE EXISTING T-WALL PILES.
3. ALL OBSTRUCTIONS OTHER THAN POLES BUILT ABOVE GROUND AND WITHIN THE EXISTING PERPETUAL LEVEE AND FLOODWALL STRUCTURES SHALL BE MADE REMOVABLE. THE CONSTRUCTION OF SUCH OBSTRUCTIONS REQUIRES A PERMIT APPROVAL ISSUED BY THE LOCAL LEVEE DISTRICT. IT SHALL BE THE RESPONSIBILITY OF THE OWNER TO REMOVE THE OBSTRUCTION AT THE REQUEST OF THE LOCAL LEVEE DISTRICT.
4. UTILITIES, BUILDINGS, ETC. ARE PROHIBITED FROM BEING ATTACHED TO ANY PART OF A FLOODWALL.
5. UTILITIES ARE PROHIBITED FROM PASSING THROUGH I-WALLS. PENETRATIONS THROUGH T-WALLS ARE ACCEPTABLE PROVIDED A USACE APPROVED SEALED SLEEVE IS USED. TYPICAL SLEEVE DETAILS ARE SHOWN ON DWG. X OF X. ALL PENETRATIONS SHALL BE APPROVED IN ACCORDANCE WITH THE LOCAL LEVEE DISTRICT PERMIT PROCESS.

SCALE: 1/2" = 1'- 0"

U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA

CLEARANCE REQUIREMENTS FOR UTILITIES AND PILE FOUNDED STRUCTURES BUILT WITHIN PERPETUAL FLOODWALL SERVITUDE

FILE NUMBER

DWG. 1 OF 2

13.0 SPECIFICATIONS

13.1 Sampling of References

- ER 1110-1-8155, Engineering and Design Specifications
- ER 1110-2-1200, Plans and Specifications for Civil Works Projects

13.2 In General

Specification preparation shall follow ECB 2006-4, dated 08JUN06, which mandates CSI MasterFormat 2004. MVN Guide Specifications are available for download from Design Guidelines page on the MVN Engineering Division web site at http://www.mvn.usace.army.mil/eng. These can be used as reference in compiling project specifications using MasterFormat 2004. Certain contracts may require MasterFormat 1995 to comply with prior agreements.

Specifications shall be prepared using the government's SpecsIntact system. SpecsIntact (Specifications-Kept-Intact), a software program copyrighted by NASA, is mandated for use in producing USACE project specifications. The program is available for free download at http://specsintact.ksc.nasa.gov/Index.asp. The SpecsIntact web page includes instructions and online help for use of the program.

APPENDIXES

A. LIST OF ACRONYMS

ACES	Automated Coastal Engineering System numerical model
ACI	American Concrete Institute
ADCIRC	ADvanced CIRCulation Multi-dimensional Hydrodynamic Model
A-E	Architect-Engineer consultant
AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
ASTM	American Society for Testing and Materials
AWS	American Welding Society
AWSE	Authorized Water Surface Elevation
BCOE	Bidability, Constructability, Operability, and Environmental Review
CADD	Computer Assisted Drafting and Design
CEDAS	Coastal Engineering Design and Analysis System
CEM	Coastal Engineering Manual, EM1110-2-1100
CFR	Code of Federal Regulations
C-Frame	Structural Analysis software
CGSI	Strength Analysis of Concrete Structural Elements software
CHAMP	Coastal Hazard Analysis Modeling Program
CIH	Certified Industrial Hygienist
CONUS	Continental United States
COR	Contracting Officer's Representative
CPGA	Pile Group Analysis software
CPT	Cone Penetration Test
CSP	Certified Safety Professional
CWALSHT	Sheet Pile Wall Design/Analysis software
DFAR	Defense Federal Acquisition Regulations
DTM	Digital Terrain Model
DWSE	Design Water Surface Elevation
ERDC	Engineer Research and Development Center
EST	Empirical Simulation Technique
ETL	Engineering Technical Letter
FDA	Flood Damage Assessment numerical model
FEMA	Federal Emergency Management Agency
FOA	Field Operating Activity
FOS	Factor of Safety
GIWW	Gulf Intracoastal Water Way Navigational Channel Project
HPO	Hurricane Protection Office at MVN
HTRW	Hazardous, Toxic, and Radioactive Waste
HURDAT	HURricane DATabase
IH	Industrial Hygienist
IHNC	Inner Harbor Navigational Channel, LA, Project
IPET	Interagency Performance Evaluation Taskforce
ITR	Independent Technical Review

JPM-OS	Joint Probability Method-Optimal Selection
LACPR	Louisiana Coastal Protection and Restoration Study
LADOTD	Louisiana Department of Transportation and Development
LCA	Louisiana Coastal Area, LA, Ecosystem Restoration Study
LIDAR	Laser Detection and Ranging
LPV	Lake Pontchartrain, LA & Vicinity Hurricane Protection Project
LRFD	Load and Resistance Factor Design
MRGO	Mississippi River Gulf Outlet, LA, Navigational Channel Project
MRL	Mississippi River Levees
MRT	Mississippi River & Tributaries Project
MTC	Materials Testing Center at ERDC
MVN	USACE New Orleans District
NAVD88	North American Vertical Datum
NDT	Non-Destructive Testing
NEPA	National Environmental Policy Act, 1969
NFPA	National Fire Protection Association
NGS	National Geodetic Survey agency
NGVD29	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
NOV	New Orleans to Venice, LA, Hurricane Protection Project
P&S	Plans and Specifications
PBL	Planetary Boundary Layer
PDA	Pile Dynamic Analysis
PDT	Product Delivery Team
POC	Point of Contact
PPC	Precast Prestressed Concrete
PRO	Protection and Restoration Office at MVN
ROE	Right of Entry
ROW	Right-of-Way
SPH	Standard Project Hurricane synthetic design storm
SSPC	Steel Structures Painting Council
STAAD	Structural Analysis and Design software
STWAVE	Steady State Irregular WAVE numerical model
SWAN	Simulating WAves Nearshore numerical model
SWL	Still Water Level
TFG	Task Force Guardian at MVN
TFH	Task Force Hope at MVN
TR4	Technical Report No. 4, “Shore Protection, Planning and Design,” Third Edition, 1966, Coastal Engineering Research Center, USACE
TRM	Turf Reinforcement Mats
TRS	Temporary Retaining Structure
UCT	Unconfined Compression Test
USACE	US Army Corps of Engineers
WAM	Water Availability Models

WBV	West Bank & Vicinity, New Orleans, LA Hurricane Protection Project

B. LINKS TO REFERENCES

US Army Corps of Engineers, engineering regulations, circulars, manuals, and other documents originating from HQUSACE
http://www.usace.army.mil/publications/

Coastal and Hydraulics Laboratory, Engineer Research and Development Center, Waterways Experiment Station, Vicksburg, Mississippi
http://chl.erdc.usace.army.mil/chl.aspx?p=Publications

Technical Report: Erosion Resistance of Grassland as Dike Covering, Technical Advisory Committee for Flood Defence in The Netherlands (TAW), Delft, Version 26 November 1997
http://www.tawinfo.nl/engels/downloads/TRGrasslandDikeCoverige.pdf

Defense Federal Acquisition Regulation, Office of the Under Secretary for Acquisition Technology and Logistics, Defense Procurement and Acquisition Policy
http://www.acq.osd.mil/dpap/

LADOTD Standard Specifications for Roads and Bridges, Louisiana Department of Transportation and Development
http://www.dotd.louisiana.gov/

SpecsIntact, Software and Instructions, NASA
http://specsintact.ksc.nasa.gov/

ECB 2006-4, Unified Facilities Guide Specifications Transition to Construction Specifications Institute MasterFormat 2004, USACE Directorate of Civil Works
http://www.wbdg.org/ccb/ARMYCOE/COEECB/ecb_2006_4.pdf

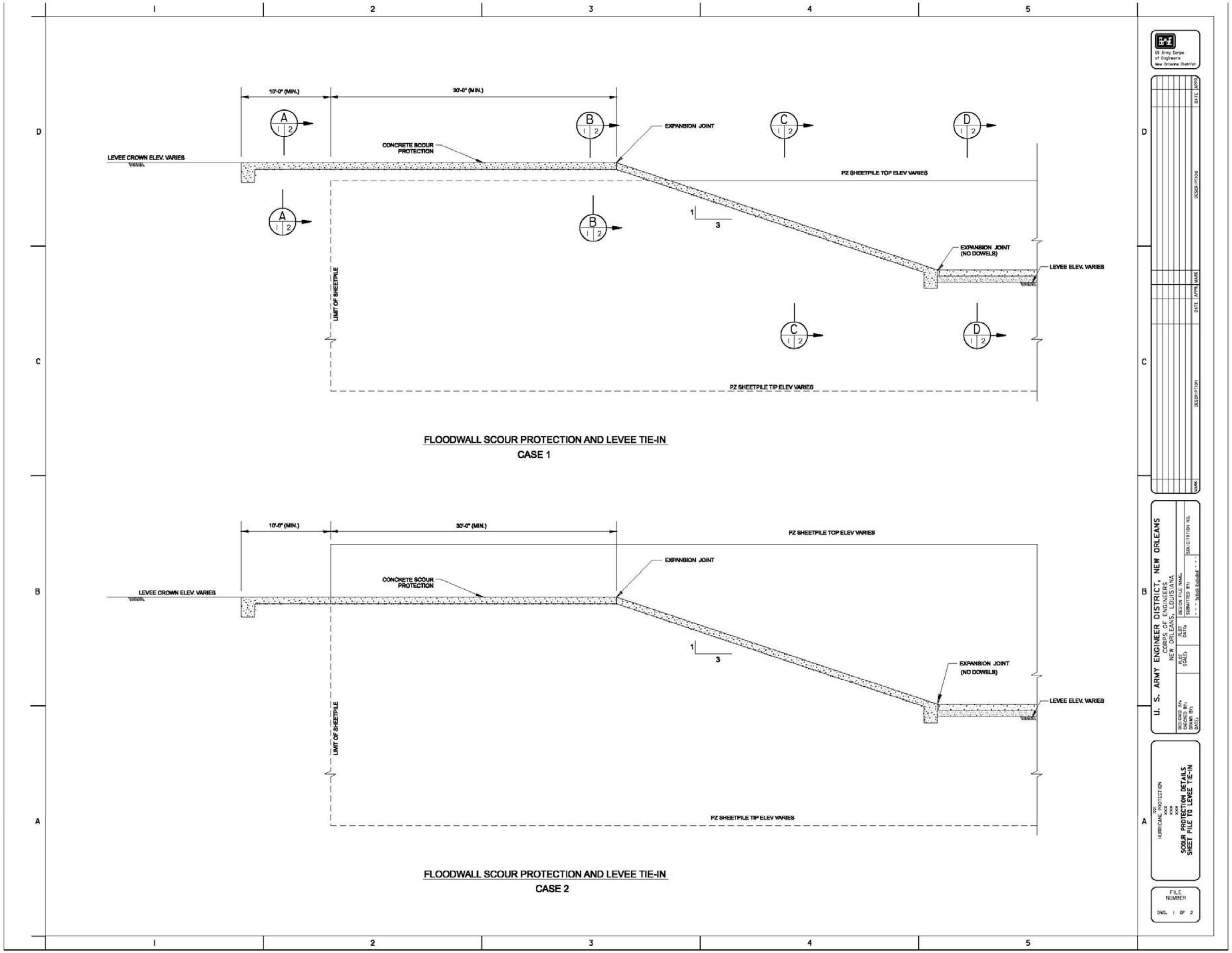
NFPA 37: Standard for the Installation and Use of Stationary Combustion Engines and Gas Turbines, National Fire Protection Association
http://www.nfpa.org

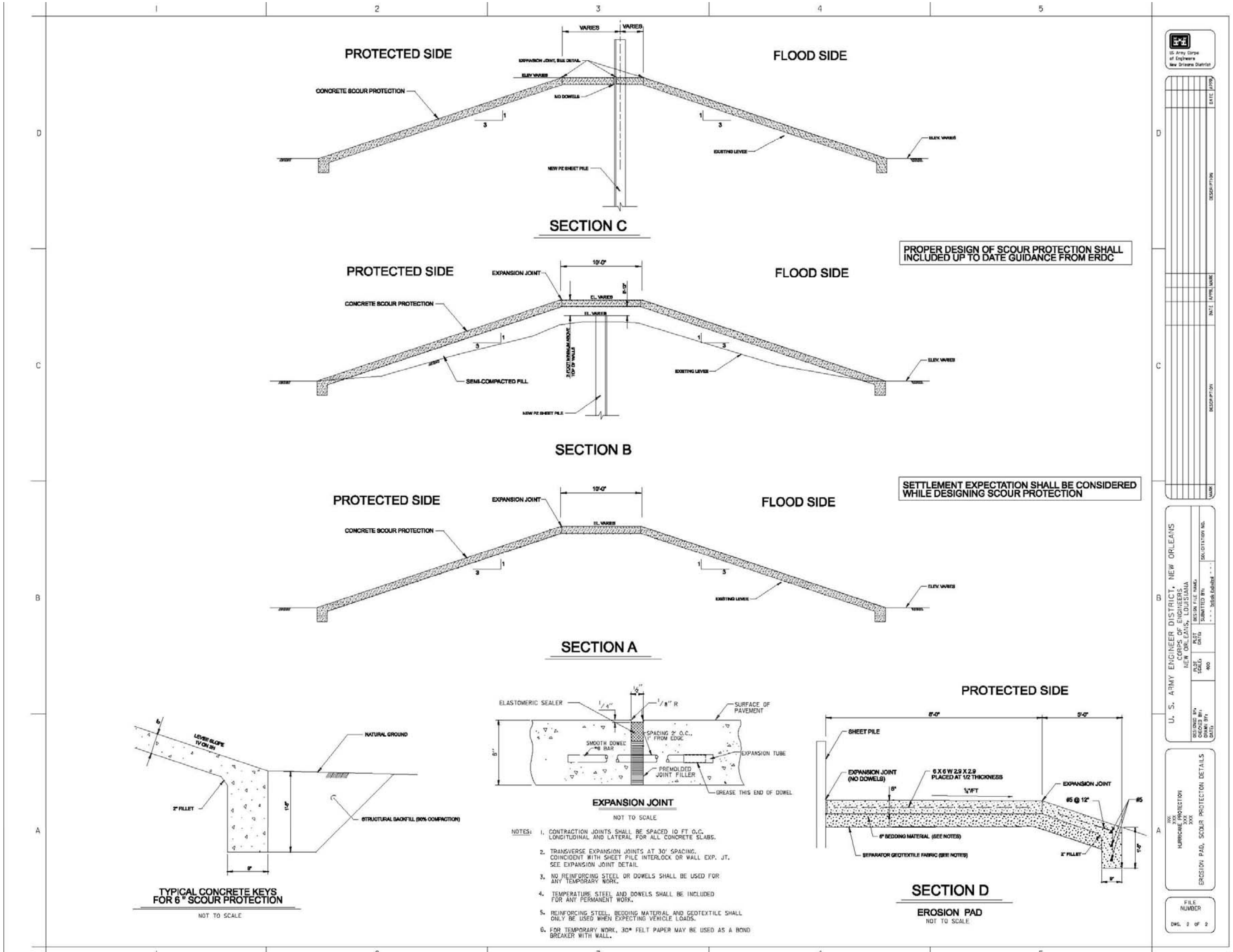
MVD CAD Resources
http://www.mvd.usace.army.mil/cad/

C. SAMPLE SCOUR PROTECTION DETAILS

Some sample details utilized by TFG are shown on the following plates. These drawings show work typical to date, however, future ERDC and IPET reports shall be used for guidance.

UPDATED 04 OCT 07




PROTECTED SIDE
FLOOD SIDE
VARIES
CONCRETE SCOUR PROTECTION
NO DOWELS
EXISTING LEVEE
NEW PZ SHEET PILE
SECTION C
PROPER DESIGN OF SCOUR PROTECTION SHALL INCLUDED UP TO DATE GUIDANCE FROM ERDC
PROTECTED SIDE
EXPANSION JOINT
10'-0"
FLOOD SIDE
CONCRETE SCOUR PROTECTION
SEMI-COMPACTED FILL
EXISTING LEVEE
SECTION B
SETTLEMENT EXPECTATION SHALL BE CONSIDERED WHILE DESIGNING SCOUR PROTECTION
PROTECTED SIDE
EXPANSION JOINT
10'-0"
FLOOD SIDE
CONCRETE SCOUR PROTECTION
EXISTING LEVEE
SECTION A
NATURAL GROUND
2" FILLET
STRUCTURAL BACKFILL (90% COMPACTION)
TYPICAL CONCRETE KEYS FOR 6" SCOUR PROTECTION
NOT TO SCALE
ELASTOMERIC SEALER
SURFACE OF PAVEMENT
SPACING 2' O.C., 1' FROM EDGE
SMOOTH DOWEL #8 BAR
EXPANSION TUBE
PREMOLDED JOINT FILLER
GREASE THIS END OF DOWEL
EXPANSION JOINT
NOT TO SCALE
NOTES: 1. CONTRACTION JOINTS SHALL BE SPACED 10 FT O.C. LONGITUDINAL AND LATERAL FOR ALL CONCRETE SLABS.
2. TRANSVERSE EXPANSION JOINTS AT 30' SPACING. COINCIDENT WITH SHEET PILE INTERLOCK OR WALL EXP. JT. SEE EXPANSION JOINT DETAIL
3. NO REINFORCING STEEL OR DOWELS SHALL BE USED FOR ANY TEMPORARY WORK.
4. TEMPERATURE STEEL AND DOWELS SHALL BE INCLUDED FOR ANY PERMANENT WORK.
5. REINFORCING STEEL, BEDDING MATERIAL AND GEOTEXTILE SHALL ONLY BE USED WHEN EXPECTING VEHICLE LOADS.
6. FOR TEMPORARY WORK, 30# FELT PAPER MAY BE USED AS A BOND BREAKER WITH WALL.
PROTECTED SIDE
SHEET PILE
EXPANSION JOINT (NO DOWELS)
EXPANSION JOINT
6" BEDDING MATERIAL (SEE NOTES)
SEPARATOR GEOTEXTILE FABRIC (SEE NOTES)
2" FILLET
SECTION D
EROSION PAD
NOT TO SCALE
U. S. ARMY ENGINEER DISTRICT, NEW ORLEANS
CORPS OF ENGINEERS
NEW ORLEANS, LOUISIANA
HURRICANE PROTECTION
EROSION PAD, SCOUR PROTECTION DETAILS
FILE NUMBER

GENERAL NOTES AND GUIDANCE FOR FLOODWALL SCOUR PROTECTION AND LEVEE TIE-IN

THIS GUIDANCE IS THE MINIMUM REQUIREMENTS NECESSARY. SITE SPECIFIC FACTORS,MUST BE INVESTIGATED TO ACCOUNT FOR GEOTECHNICAL, HYDRAULIC, AND MAINTENANCE VARIANCES.

1. FLOODWALL OVERTOPPING CONCRETE SCOUR PROTECTION:

   A. LIMITS OF SCOUR PROTECTION ON THE PROTECTED SIDE OF I-WALLS:
      MINIMUM OF 8 FEET OF TOTAL SCOUR PROTECTION
      MINIMUM OF 5 FEET DOWN SLOPING EMBANKMENTS

   B. CONCRETE PAVEMENT WITH VEHICULAR TRAFFIC (1/2 TON TRUCKS, CARS):
      6-7 INCHES LIGHTLY REINFORCED CONCRETE
      6 INCH BEDDING
      SEPARATOR GEOTEXTILE FABRIC BETWEEN BEDDING AND SOIL
      PROPER DESIGN AND LAYOUT OF EXPANSION, AND CONTROL JOINTS ARE NECESSARY FOR DIFFERENTIAL MOVEMENT

   C. CONCRETE PAVEMENT NO WITH VEHICULAR TRAFFIC:
      5-6 INCHES LIGHTLY REINFORCED CONCRETE, BEDDING AND GEOTEXTILE ARE OPTIONAL
      ENGINEERING JUDGMENT, PROPER DESIGN AND LAYOUT OF EXPANSION, AND CONTROL JOINTS ARE NECESSARY FOR DIFFERENTIAL MOVEMENT.

   D. CONCRETE KEYS:
      PROPER LOCATION OF THE CONCRETE KEYS NEEDS TO BE CONSIDERED TO PREVENT SLIDING AND POTENTIAL PIPING UNDER THE PAVEMENT.

   E. SCOUR PROTECTION MATERIALS:
      OPTIONAL MATERIALS SHOULD BE INVESTIGATED FOR SITE SPECIFIC CONDITIONS.

   F. FLOOD SIDE SCOUR PROTECTION OR WAVE RUN UP BERMS MUST BE CONSIDERED WHERE WAVE POTENTIAL EXIST.

   G. TYPICAL DETAILS ARE THE MINIMUM REQUIREMENTS FOR CONCRETE PAVING PROTECTION.

2. FLOODWALL TO LEVEE TIE-IN GUIDANCE AND DETAILS:

   A. TIE-IN DETAILS FOR I-WALLS OR T-WALLS THAT END INTO A LEVEE SECTION MUST FOLLOW THIS GUIDANCE.

   B. AS A MINIMUM, UNCAPPED SHEET PILING MUST EXTEND 30 FEET PASS THE END OF THE FLOODWALL INTO THE LEVEE SECTION.
      PROPER EARTHEN COVER AND SCOUR PROTECTION IS MANDATORY.
      FUTURE SETTLEMENT SHOULD BE ACCOUNTED FOR IN DETAILING SCOUR PROTECTION OVER SHEETING PILING AND ADJACENT FLOODWALLS.

3. TYPICAL FLOODWALLS DETAILS SHOULD BE USED FOR TRANSITIONS FROM T-WALL OR I-WALL TO THE UNCAPPED SHEET PILING (TYPICAL SLIP JOINT). LENGTHS OF I-WALL MONOLITHS, BETWEEN EXPANSION JOINTS, SHOULD GENERALLY BE 30 FEET AND END AT THE NEAREST SHEET PILE INTERLOCK.

4. TIE-IN LIMITS AND DETAILS AS SHOWN ON THE TYPICAL DETAILS ARE MINIMUMS.

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MISSISSIPPI VALLEY DIVISION

FEBRUARY 2006

LEVEE TIE-IN
GENERAL DESIGN NOTES

SHEET IDENTIFICATION NUMBER

GEOTEXTILE SEPARATOR FABRIC

1. THE GEOTEXTILE SEPARATOR FABRIC SHALL BE A WOVEN PERVIOUS SHEET MADE WITH PLASTIC YARN AS DEFINED BY ASTM D 883. THE GEOTEXTILES SHALL MEET THE REQUIREMENTS LISTED IN TABLE 1.

TABLE 1

REQUIREMENTS FOR SEPARATOR GEOTEXTILE

PROPERTY	TEST PROCEDURE	ACCEPTABLE VALUES
GRAB BREAKING LOAD	ASTM D 4632	200 POUNDS MINIMUM IN ANY PRINCIPLE DIRECTION
SEAM STRENGTH (**)	ASTM D 4632	100 POUNDS,MINIMUM
ELONGATION AT BREAK	ASTM D 4632	15 PERCENT MINIMUM IN ANY PRINCIPLE DIRECTION
APPARENT OPENING SIZE (AOS)	ASTM D 4751	NO FINER THAN THE U.S. STANDARD SIEVE NO. 50 AND NO COARSER THAN THE U.S. STANDARD SIEVE NO. 30
PERMITTIVITY	ASTM D 4491	0.35 PER SECOND MINIMUM
FLOW RATE	ASTM D 4491	MINIMUM OF 40 GALLONS PER MINUTE PER SQUARE FOOT

(*) VALUE REPRESENTS MINIMUM AVERAGE ROLL VALUE OF NEW GEOTEXTILE RECEIVED FROM THE MANUFACTURER OR DISTRIBUTOR.

(**) ALL OF THE SAMPLES SHALL YIELD TEST VALUES THAT ARE GREATER THAN THE MINIMUM VALUE THAT IS SPECIFIED.

OPTIONAL SCOUR MATERIALS:

(SCOUR PROTECTION MUST BE DESIGNED FOR THE INTENDED PURPOSE, PERFORMANCE AND QUALITY WHILE CONSIDERING LONG-TERM MAINTENANCE NEEDS.)

LIGHTLY REINFORCED CONCRETE PAVEMENT
RECOMMENDED GABIONS.
ANCHORMAT CONCRETE REVETMENTS.
ROCK FILLED GEOGRID PANELS.
ARTICULATED CONCRETE BLOCK
ASPHALT CONCRETE PAVEMENT

OTHER MATERIALS:
LIGHT WEIGHT STONE RIPRAP IS NOT RECOMMENDED (HYDRAULIC FACTORS).
LARGER STONE RIPRAP MUST BE SPECIFICALLY DESIGNED FOR SITE CONDITIONS. FOUNDATIONS SETTLEMENT MUST BE CONSIDERED.
GROUTED-IN RIPRAP OVER BEDDING MATERIALS IS AN ACCEPTABLE OPTION AS INTERIM SCOUR PROTECTION AREAS (LEVEE TIE-IN) WHERE LONG TERM EMBANKMENT SETTLEMENT IS EXPECTED.

STONE BEDDING

IF THE FOUNDATION IS ERODABLE OR THE WAVES ARE FREQUENTLY OCCURRING OR SEVERE, THEN A FILTER MAY HAVE TO BE DESIGNED. DEPENDS ON THE LEVEL OF PROTECTION AFFORDED BY THE GROUT INTO THE RIPRAP. HYDRAULICS BRANCH SHOULD BE CONSULTED. RECOMMEND A COARSE MATERIAL BE USED UNDER THE BEDDING.

NOTE: SEEPAGE CRITERIA ARE PRESENTED IN EM 1110-2-1913.
EACH BEDDING MATERIAL SYSTEM SHOULD BE DESIGNED TO BE USED WITH A SPECIFIC RANGE OF RIPRAP GRADATIONS, FOUNDATION CONDITIONS, AND CHANNEL CONDITIONS.

1. BEDDING MATERIAL

BEDDING MATERIAL UNDER RIPRAP SHALL BE 100 PERCENT CRUSHED STONE AND SHALL SHOW AN ABRASION LOSS OF NOT MORE THAN 40 PERCENT WHEN TESTED IN ACCORDANCE WITH ASTM C 131 AND A SOUNDNESS LOSS OF NOT MORE THAN 15 PERCENT WHEN SUBJECTED TO 5 CYCLES OF THE MAGNESIUM SULFATE SOUNDNESS TEST IN ACCORDANCE WITH ASTM C 88. BEDDING SHALL CONFORM TO THE FOLLOWING GRADATION WHEN TESTED IN ACCORDANCE WITH ASTM C 136:

NOTE: THE LARGEST SIZE APPROPRIATE FOR THE RIPRAP BEING SHOULD BE USED SO THAT THE MATERIAL DOES NOT GET WASHED THROUGH THE RIPRAP. DELETE THE OTHER 2 GRADATIONS.

RIPRAP BEDDING

PERCENT PASSING

US SIEVE	GRADE A	GRADE B	GRADE D
2 ½-INCH	---	---	100
2-INCH	---	100	90 – 100
1 ½-INCH	100	85-100	---
1-INCH	90–100	---	35 – 70
¾-INCH	---	35–70	---
½-INCH	25–60	---	10–30
3/8-INCH	---	10–30	---
NO. 4	0–10	0–5	0-5
NO. 8	0–5	---	---
NO. 200	0–1	0-1	0-1

STONE BEDDING UNDER CONCRETE PAVING

2. BEDDING

BEDDING MATERIAL UNDER CONCRETE PAVING SHALL SHOW AN ABRASION LOSS OF NOT MORE THAN 40 PERCENT WHEN TESTED IN ACCORDANCE WITH ASTM C 131 AND A SOUNDNESS LOSS OF NOT MORE THAN 15 PERCENT WHEN SUBJECTED TO 5 CYCLES OF THE MAGNESIUM SULFATE SOUNDNESS TEST IN ACCORDANCE WITH ASTM C 88.

BEDDING SHALL MEET ONE OF THE FOLLOWING:

2.1 CRUSHED STONE

CRUSHED STONE SHALL CONSIST OF 100 PERCENT STONE AND SHALL MEET THE FOLLOWING REQUIREMENTS WHEN TESTED IN ACCORDANCE WITH ASTM C 136 AND ASTM C 117:

US SIEVE	PERCENT PASSING
1 ½-INCH	100
1-INCH	90 – 100
¾-INCH	70 - 100
NO. 4	35 - 65
NO. 40	12 – 32
NO. 200	0 – 8

THE FRACTION OF STONE PASSING THE NO. 40 SIEVE SHALL COMPLY WITH THE FOLLOWING REQUIREMENTS:

LIQUID LIMIT (MAX.) 25
PLASTICITY INDEX (MAX.) 4

2.2 RECYCLED PORTLAND CEMENT CONCRETE:
RECYCLED PORTLAND CEMENT CONCRETE SHALL BE 100 PERCENT CRUSHED PORTLAND CEMENT CONCRETE OR WILL BE PERMITTED IN COMBINATION WITH AN APPROVED STONE FOR BASE COURSE. AFTER BEING CRUSHED, THE RECYCLED PORTLAND CEMENT CONCRETE OR THE COMBINATION OF STONE AND RECYCLED PORTLAND CEMENT CONCRETE SHALL COMPLY WITH THE FOLLOWING GRADATION.

US SIEVE	PERCENT PASSING
1 ½-INCH	100
1-INCH	90 – 100
¾-INCH	70 - 100
NO. 4	35 - 65
NO. 40	12 – 32
NO. 200	0 – 8

THE FRACTION OF RECYCLED PORTLAND CEMENT CONCRETE PASSING THE NO. 40 SIEVE SHALL BE NON-PLASTIC.

NOTES:
FOR SLOPE PAVEMENT WHERE TRAFFIC MAY PASS USE PARAGRAPHS UNDER 2.
FOR SLOPE PAVEMENT WHERE TRAFFIC WILL NOT PASS, BEDDING IS NOT RECOMMENDED. IF ENGINEER ELECTS TO USE IT, USE THE PARAGRAPHS UNDER 2.
FOR GROUTED RIPRAP, USE PARAGRAPH 1.

US ARMY CORPS OF ENGINEERS
NEW ORLEANS DISTRICT

U. S. ARMY ENGINEER DISTRICT
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SHEET IDENTIFICATION NUMBER

D. EXTRACT FROM DRAFT SCOUR STUDY

Following is an extract from “Protection Alternatives for Levees and Floodwalls in Southeast Louisiana: Phase One Evaluation,” a report prepared by ERDC Coastal and Hydraulics Laboratory. The document is still a draft and is marked “Intended for internal Corps use only.”

The extract included here is Chapter 4, “Protection for Overtopped Floodwalls.” This information is provided to designers to illustrate some of the design issues to be addressed when designing scour protection. This extract is provided for information only.

4 Protection for Overtopped Floodwalls

Failure Modes of Concrete and Sheetpile Floodwalls

Floodwall failures can be broadly grouped into two categories: (a) structural failure of the vertical wall due to applied hydrodynamic pressure forces; and (b) foundation failure due to seepage and liquefaction, slip surface or shear plane failures, and loss of lateral support due to erosion. This chapter focuses only on protection from loss of foundation support due to the erosive impact of falling water that has overtopped the floodwall.

Floodwalls that might be overtopped by rising water should be designed with erosion protection on the protected (dry) side capable of resisting the force of the free-falling water jet. Figure 4.1 illustrates flow overtopping a floodwall and plunging (in this case) into standing water on the protected side of the floodwall. The plunging jet penetrates the water and creates large eddies that erode material from the unprotected soil surface. The same mechanism will scour bed material when there is not standing water on the protected side of the floodwall.

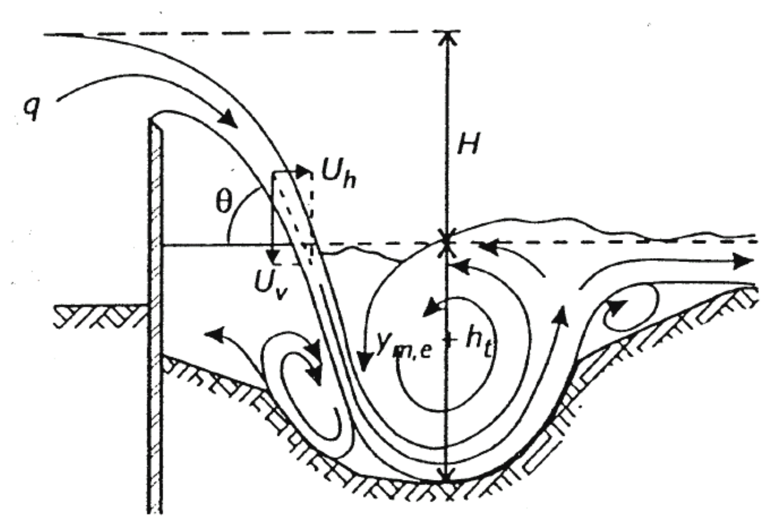

Figure 4.1. Scour hole formation by overtopping jet (from Hoffmans and Verheij 1997)

Eroded material is thrown into suspension and carried away by the turbulent flow. This scouring action removes material that may be providing critical lateral support pressure against the protected side of the vertical floodwall. Failure occurs if the remaining, undamaged portion of the foundation adjacent to the wall cannot withstand either the shear force or the overturning moment exerted on the floodwall by the elevated water on the flood side of the wall.

Total collapse of a section of the floodwall allows a large volume of water to flow into the protected region through the resulting breach, and this may cause adjacent wall sections to fail and enlarge the breach. Localized partial failure includes tilting of the floodwall so gaps open up between the dislodged section and adjacent undamaged floodwall. Provided the wall does not tilt farther, it still affords some degree of flood protection. However, the wall top elevation is deceased slightly by tilting, and the overflowing water jet will be directed on foundation soil farther away from the wall that could increase the scour hole width.

Figure 4.2 shows scour on the protected side of an I-wall adjacent to the Lakefront Airport. A deep trench was scoured by the overflowing jet, but in this case the floodwall does not appear to be affected by the loss of lateral support at the base.

Figure 4.2. Scour trench formed by overtopping flow at I-wall adjacent to the Lakefront Airport (photograph by Peter Nicholson from Seed, et al. (2005)).

Figure 4.3 shows the I-wall along the east side of the IHNC at approximate B/L Sta 11+00 (DM3 Chalmette Area Plan), looking toward the Claiborne Avenue bridge. Depth of scour was to the bottom of the I-wall concrete cap (2 ft), and scour trench width was approximately 7 ft. The I-wall top elevation was designed to a height of 15 ft above mean sea level, the bottom of the concrete cap was at elevation 7 ft, and the earthen levee crown was at elevation 9 ft. Actual wall height was reported to be 12.5 ft when converted to local mean sea level, and the storm surge height was reported to be up to 15 ft. As an approximation of the overtopping water impact, a surge crest 2.5 ft above the floodwall impacted the earthen levee crown from a height of 6 ft. Using procedures developed in the following section, the falling jet of water was estimated from Figure 4.12 to have an impact velocity of about 23 ft/sec, and the impact force was estimated from Figure 4.13 to be about 700 lb/ft. The water impact removed a portion of the earthen levee crown, including all of the structural backfill zone adjacent to the concrete wall.

Figure 4.3. Scour trench on the east side of the IHNC

Soil scour within the structure backfill zone is also evident at other locations such as the T-wall on the north side of Gate 13E on the east side of the IHNC near Lakefront Airport at approximate W/L Sta 61+38 (DM2 Supplement 8 IHNC Remaining Levees). The top of T-wall elevation was 13.25 ft (MSL) and the existing top of ground elevation was 0.1 ft (MSL), from drawing file H-2- 24111, plate IV-20. Figure 4.4 shows a scour trench with depth of 30 in. and trench width of approximately 8 ft. Overtopping water dropped 13 ft before impacting the levee. Figures 4.12 and 4.13 were used to estimate an impact velocity of about 30 ft/sec and an impact force over 700 lb/ft.

Figure 4.4. Scour trench at a T-wall on the east side of the IHNC

Reaches along the MRGO protected by exposed sheetpile floodwalls experienced scouring on the backside, and breaches occurred at several locations. Figure 4.5 shows a section with 4300 ft of sheetpile damage along MRGO between Bayous Bienvenue and Dupre, St. Bernard Parish. The damaged sheetpile section is near utility crossings, with scour on the protected side and levee crown. B/L Sta 590+70 is centerline of the two pipelines.

Larger breaches along sheetpile reaches were evident on the north bank of the GIWW, including the Bulk Loading Facility, the Michoud Canal (Air Products plant), and pump station 15. Figure 4.6 shows the Air Products plant breach near Sta 772+00 B/L (New Orleans East Back Levee). Scour depths were 10 to 12 ft on both the floodside and protected side of the sheetpile wall. Nearest borings on either side of the failure, 5-E and 6-E (from plate 5, DM2 Supp 4, March 1971) shows CH material with sand / silt lenses in the pre-existing (1965) levee at crown elevation ~12 ft, prior to construction of the sheetpile wall. The storm surge in the GIWW was at an approximate elevation of 15 to 17 ft, and Figures 4.12 and 4.13 indicate the estimated overtopping jet impact velocity ranged up to about 23 ft/sec, and the impact force ranged up to about 700 lb/ft. Note that the breach occurred in the sheetpile reach, and not along the adjacent transitions to earthen levee on the east side and connection to the T-wall on the west side.

Figure 4.5. Overtopping scour at sheetpile floodwall along the MRGO

Figure 4.6. Sheetpile floodwall breach on the New Orleans East Back Levee

Several other vertical structures (mostly I-walls) were catastrophically breached along the 17th St. and London Avenue Canals and the IHNC (East and West sides). Investigations are ongoing, but it appears that failure modes other than erosion and scour caused by overtopping water may have played larger roles at those locations.

Design Physical Parameters

Scour protection placed on top of the foundation soil on the protected side of floodwalls must be able to withstand a free-falling jet of water that overtops the wall. This condition could persist for a prolonged period. Protection coverage must extend away from the wall sufficient distance to assure complete protection from both the direct plunging water jet, and also from the resulting ground-parallel supercritical flow and eventual hydraulic jump that forms some distance from the wall. Important design parameters related to the flow hydrodynamics are floodwall height and height of the storm surge level relative to the floodwall top elevation.

Under the assumption that robust structural foundation protection is necessary, geotechnical design parameters are somewhat limited to the requirement that foundation soil must support the overlain protection without significant differential settlement. Also important is the possibility of soil erosion at the boundaries of the overtopping protection. Geotechnical considerations related to proper foundation design to resist the applied lateral loading on the floodwall and to prevent seepage underneath the wall are not included in this chapter.

Surge Overtopping

Storm surge overtopping of a floodwall having constant top elevation along the wall is well approximated by the classic hydraulics problem of flow over a sharp-crested weir. Assuming no viscous energy dissipation occurs over the short crest width of the vertical floodwall shown in Figure 4.7, and there are no lateral contraction effects (i.e., constant wall top elevation), discharge per unit wall length is given by the expression (e.g., Henderson 1966)

$$q = C_d \, \frac{2}{3} \sqrt{2g} \; h_1^{3/2} \tag{4.1}$$

The discharge coefficient, C_d, is given by the expression

$$C_d = 0.611 \left[\left(1 + \frac{v_1^2}{2gh_1} \right)^{3/2} - \left(\frac{v_1^2}{2gh_1} \right)^{3/2} \right] \tag{4.2}$$

where g is the acceleration of gravity, h_1 is height of the surge above the wall, and v_1 is the upstream velocity as shown on Figure 4.7. The above discharge formulation was referred to as the “Weisbach extention of the Poleni formula” by Rouse (1961) with the addition of C_d in Eqn. 4.1 and the definition of C_d (Eqn. 4.2) being Weisbach’s contribution.

Experimental work provided a simple approximation for C_d expressed as

$$C_d = 0.611 + 0.08 \left(\frac{h_1}{h} \right) \tag{4.3}$$

where h is the depth of the water as defined in Figure 4.7. For small values of h_1/h, the discharge coefficient approaches $C_d = 0.611$. Figure 4.8 presents discharge per unit length of floodwall as a function of surge elevation above the wall for values of h = 4, 6, 8, and 10 ft. For these cases the discharge curves do not have much variation until the ratio h_1/h approaches unity.

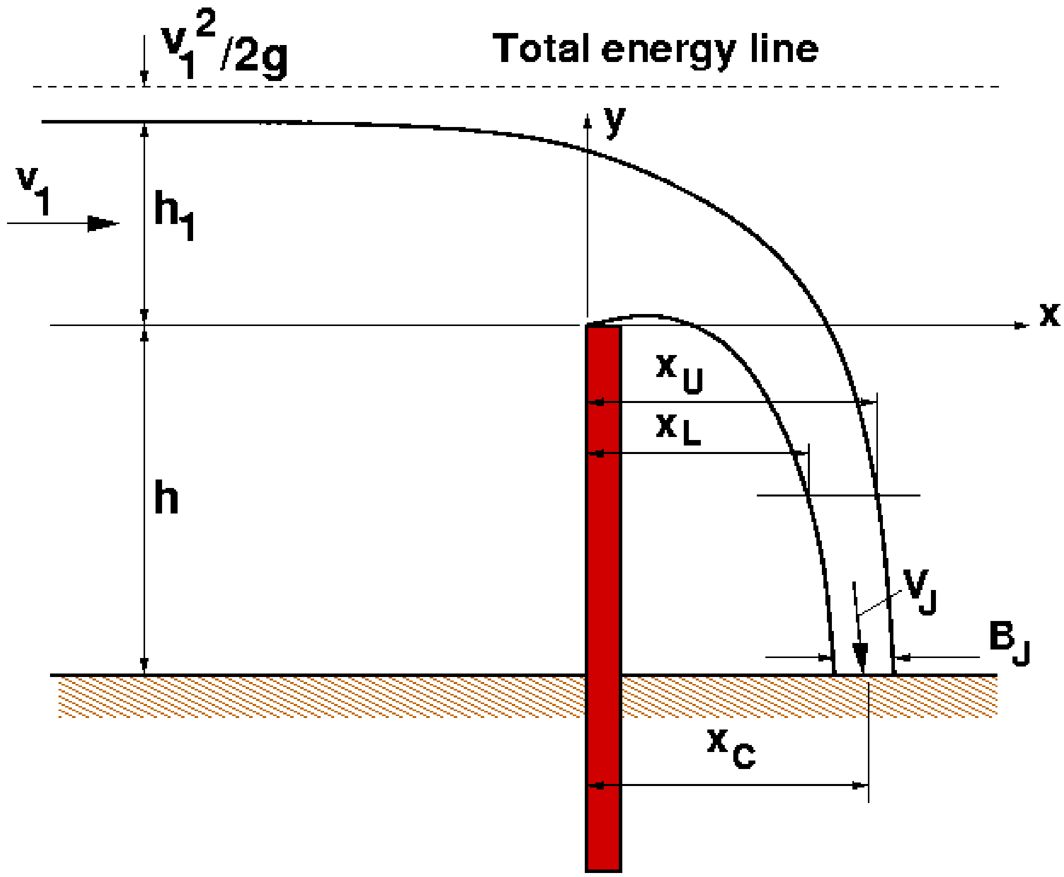

Figure 4.7. Flow over a sharp-crested weir

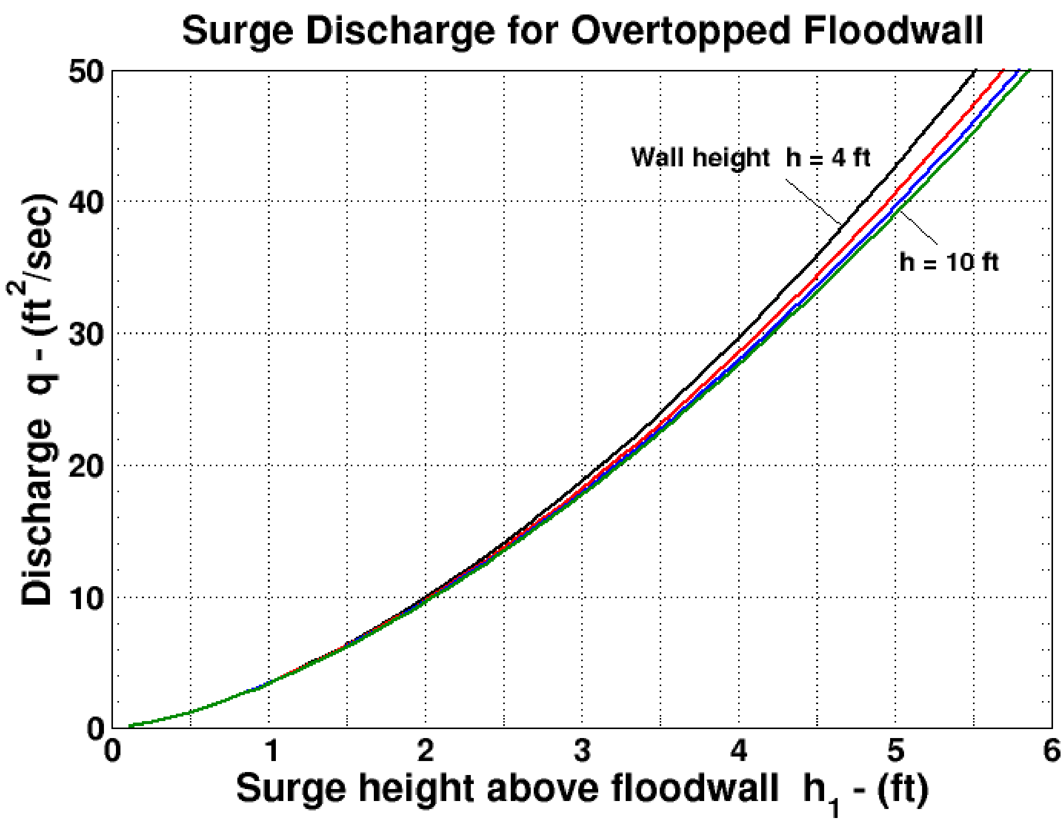

Figure 4.8. Discharge per unit floodwall length for values of h = 4, 6, 8, and 10 ft

The jet of water passing over the vertical floodwall has two surface profiles referred to as "nappes" (a French word meaning "a continuous surface"). The lower nappe is closest to the backside of the floodwall, and the upper nappe is the extension of the flow free surface as it spills over the wall. The trajectories of the lower and upper nappes are given in most open channel flow books (e.g., Chow 1959, Morris 1963). In dimensionless form, the equations are as follows with the x-y coordinate system as defined in Figure 4.7

$$\frac{y}{H} = A\left(\frac{x}{H}\right)^2 + B\left(\frac{x}{H}\right) + C \qquad \text{(lower nappe)} \qquad (4.4)$$

$$\frac{y}{H} = A\left(\frac{x}{H}\right)^2 + B\left(\frac{x}{H}\right) + C + D \qquad \text{(upper nappe)} \qquad (4.5)$$

where H is the total head above the weir crest, i.e.,

$$H = h_1 + \frac{v_1^2}{2g} \qquad (4.6)$$

and

$$A = -0.425 + 0.25\,G \qquad (4.7)$$

$$B = 0.411 - 1.603\,G - \sqrt{1.568\,G^2 - 0.892\,G + 0.127} \tag{4.8}$$

$$C = 0.150 - 0.45\,G \tag{4.9}$$

$$D = 0.57 - 0.02\left[10\left(G - 0.208\right)\right]^2 \exp\left[10\left(G - 0.208\right)\right] \tag{4.10}$$

with

$$G = \frac{v_1^2}{2gH} \tag{4.11}$$

For high weirs, $v_1 \approx 0$, and $H \approx h_1$, and the nappe equations reduce to the forms

$$\frac{y}{h_1} = A\left(\frac{x}{h_1}\right)^2 + B\left(\frac{x}{h_1}\right) + C \qquad \text{(lower nappe)} \tag{4.12}$$

$$\frac{y}{h_1} = A\left(\frac{x}{h_1}\right)^2 + B\left(\frac{x}{h_1}\right) + C + D \qquad \text{(upper nappe)} \tag{4.13}$$

with

$$A = -0.425$$
$$B = 0.055$$
$$C = 0.150$$
$$D = 0.559$$

Equations 4.12 and 4.13 are quadratic equations that can be solved to give values of the nappe profile x-values in terms of the vertical distance from the top of the floodwall. There are two solutions that satisfy each quadratic equation. The equations given below are the appropriate solutions yielding positive values of x.

$$\frac{x_L}{h_1} = \frac{-B - \sqrt{B^2 - 4A\left(C - y/h_1\right)}}{2A} \qquad \text{(lower nappe)} \tag{4.14}$$

$$\frac{x_U}{h_1} = \frac{-B - \sqrt{B^2 - 4A\left(C + D - y/h_1\right)}}{2A} \qquad \text{(upper nappe)} \tag{4.15}$$

The intersection points of the lower and upper nappes with the horizontal ground level on the protected side of the floodwall are found by setting $y = -h$ in the above equations. The horizontal width of the overtopping jet at impact is given by

$$B_X = x_U\,(y = -h) - x_L\,(y = -h) \tag{4.16}$$

and the distance from the flood side of the wall to the center of the jet at impact is given as

$$x_C = \frac{x_U(y=-h) + x_L(y=-h)}{2} \tag{4.17}$$

Figure 4.9 shows the variation of jet impact location distance, x_C, from the floodwall front face as a function of surge elevation above the wall crest and the vertical plunge distance. Horizontal width of the plunging jet at impact is given as a function of the same parameters in Figure 4.10.

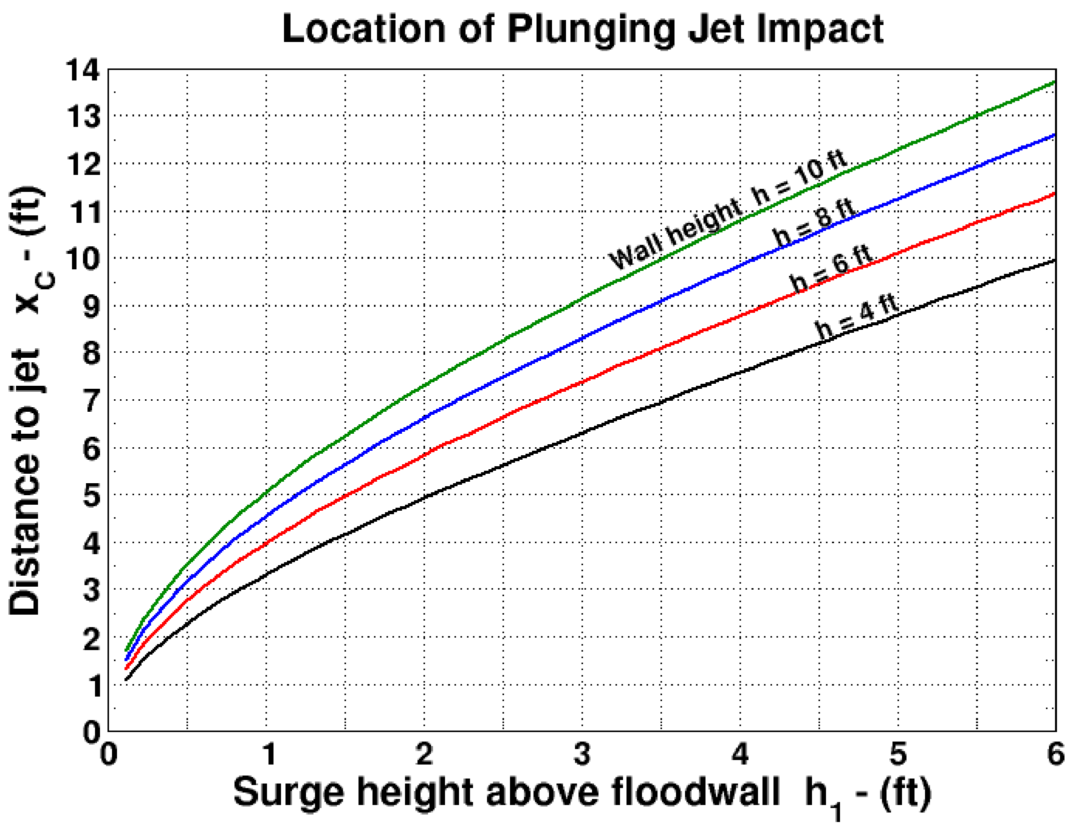

Figure 4.9. Horizontal distance between the floodwall front face and the center of the plunging jet at impact

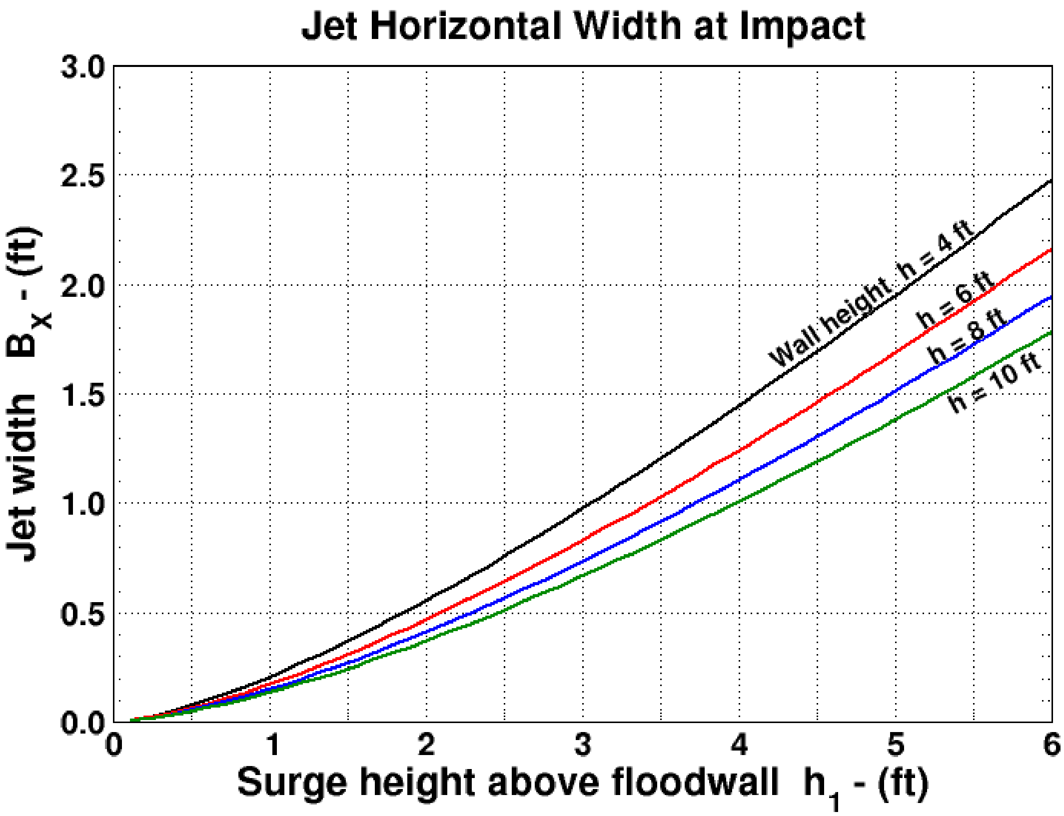

Figure 4.10. Horizontal width of the plunging jet at impact

If there is no venting, the air pressure in the space between the floodwall and lower nappe may become less than atmospheric as air is entrained into the jet during sustained overtopping. The decreased pressure will draw the plunging jet closer to the wall; however, this decrease in plunge point location away from the vertical wall is difficult to predict. This is likely not a problem because the scour protection will probably cover the entire region from the base of the wall out well past the location of jet impact.

The overtopping jet impacts the ground at an angle less than vertical (which is given by –90 deg in the coordinate system defined in Figure 4.7). The jet entry angle is well approximated by the average of the angles of the lower and upper nappe profiles when they intersect the horizontal ground level. The entry angles of the nappe profiles are found by taking the derivative of Eqns. 4.12 and 4.13 and evaluating the result at $x = x_L$ and $x = x_U$, respectively, to get

$$\theta_L = \tan^{-1}\left(\frac{dy}{dx}\right)_L = \tan^{-1}\left(\frac{2\,A\,x_L}{h_1} + B\right) \tag{4.18}$$

$$\theta_U = \tan^{-1}\left(\frac{dy}{dx}\right)_U = \tan^{-1}\left(\frac{2\,A\,x_U}{h_1} + B\right) \tag{4.19}$$

where $A = -0.425$ and $B = 0.055$. The jet entry angle is estimated as

$$\theta_J = \frac{(\theta_L + \theta_U)}{2} \tag{4.20}$$

Overtopping jet entry angles are shown on Figure 4.11 as a function of surge height above the floodwall for a variety of wall heights.

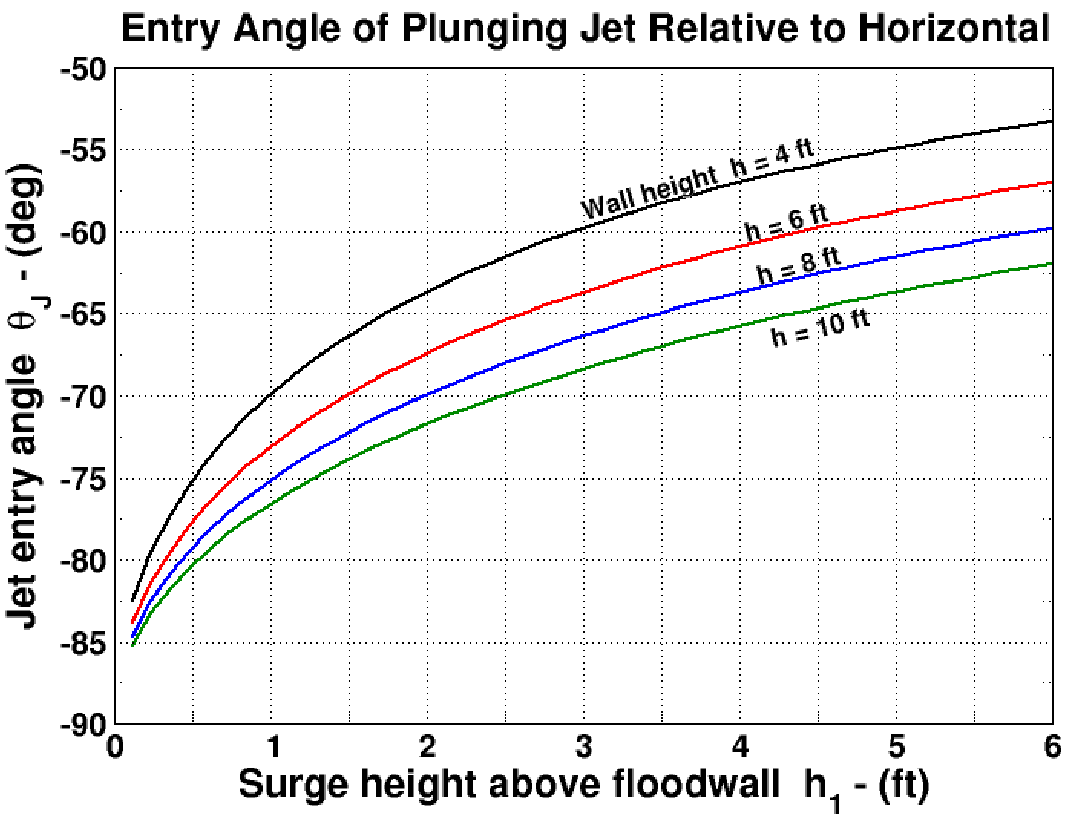

Figure 4.11. Overtopping jet entry angle relative to the horizontal ground level

From geometric considerations the width of the impinging jet normal to the flow streamlines can be estimated with reasonable accuracy by the formula

$$B_J = B_X \sin(-\theta_J) \tag{4.21}$$

Discharge over the floodwall remains constant for steady flow, and the discharge per unit length of the plunging jet at impact with the ground surface is given simply as the jet velocity parallel to the flow streamlines times the width of the jet normal to the flow. Thus, the jet entry velocity can be estimated as

$$V_J = \frac{q}{B_J} \tag{4.22}$$

Figure 4.12 shows jet impact velocities as a function of surge height above the floodwall and vertical distance to the ground level.

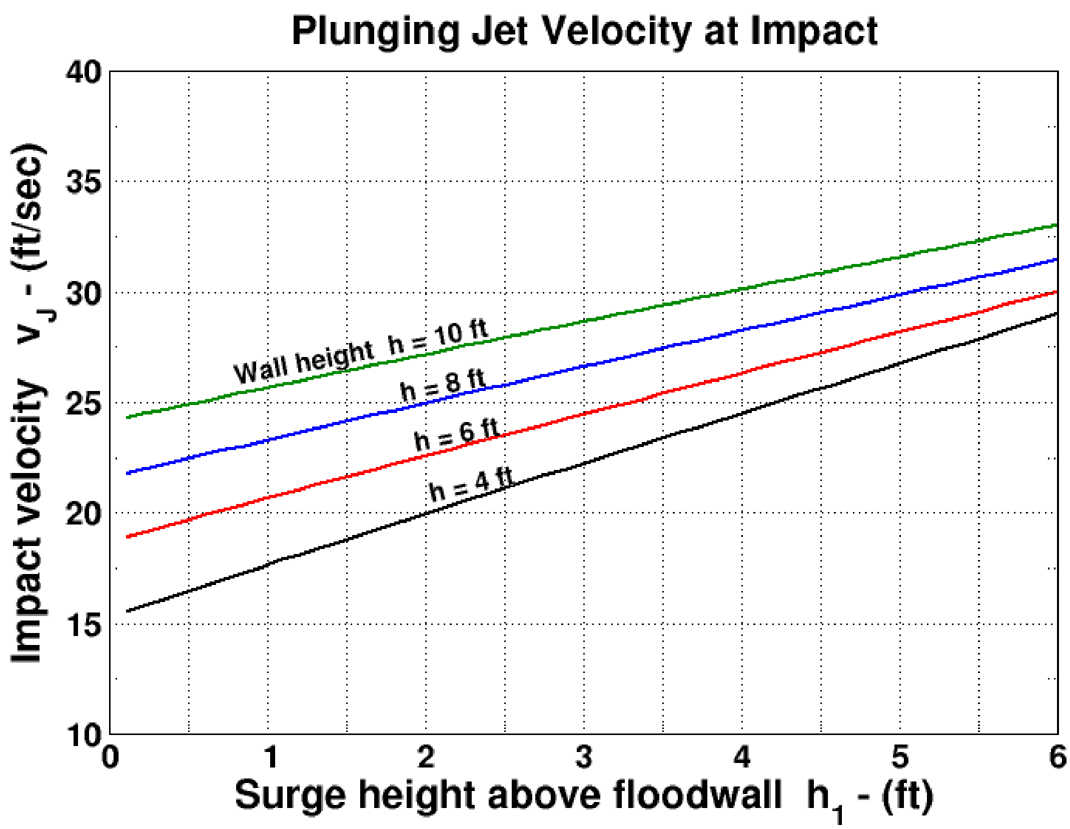

Figure 4.12. Overtopping jet velocity at impact with the ground

Finally, the total force (thrust) exerted by the overtopping jet on the scour protection per unit length along the wall is given in inviscid jet theory (e.g. Milne-Thompson 1960) as

$$F_J = \rho B_J \left(v_J\right)^2 \tag{4.23}$$

where ρ is water density. This equation is an expression of the momentum flux of the jet, and the force is directed parallel to the jet streamlines.

Figure 4.13 presents force magnitude estimates based on Eqn. 4.23. As shown on Figure 4.13, the lines for the different fall distances h are quite close because the range of fall distance is not too large. However, the impact force increases substantially with overtopping elevation h_1, that is directly related to total discharge per unit length of wall. The convergence of the lines at the higher values of h_1 is not physically correct. This convergence is most likely caused by the empirical approximations for discharge coefficient C_d (Eqn. 4.3) and jet width B_J (Eqn. 4.21).

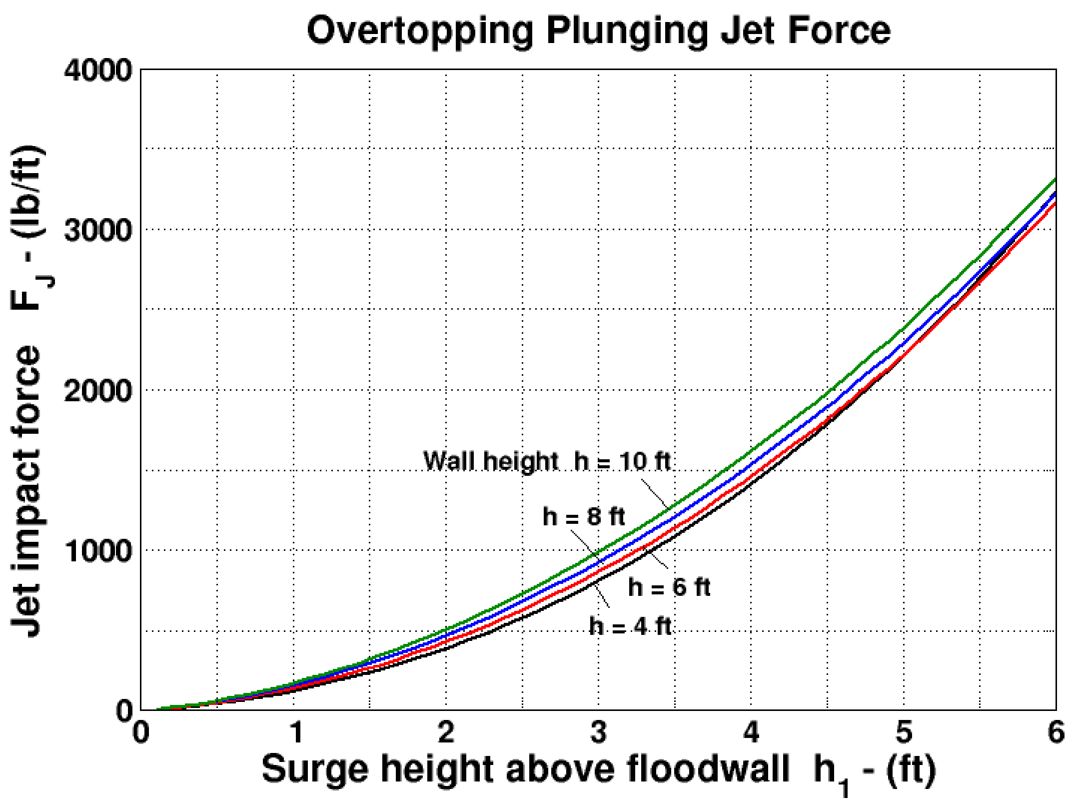

Figure 4.13. Overtopping jet impact force on the ground

The force of the overtopping jet at impact creates high pressures because the jet width is narrow (see Figure 4.10). The impact force given in Figure 4.13 can be resolved into vertical and horizontal components using the estimated jet entry angle given on Figure 4.11. Thus, the apportioning of force between vertical and horizontal components will vary with overtopping condition, and successful scour protection must be able to resist the expected range of vertical and horizontal forces. For high discharges over low walls, the jet entry angles are far from vertical, and the water after impact will retain a substantial horizontal velocity as it flows down the protected side of the earthen levee.

Depending on the elevation of the adjacent land on the protected side of the floodwall, there may be standing water at the base of the wall. The impact force of an overtopping jet will be dissipated to some degree as it enters the standing water, but it still retains sufficient force to erode unprotected foundation soil. Scour protection that relies on self-weight for stability will be less stable when submerged, and the overtopping jet may be able to dislodge submerged components of the protection.

Wave Overtopping

Waves can overtop a vertical floodwall even when the storm surge elevation is below the top elevation of the wall as illustrated by Figure 4.14. That portion of the wave above the floodwall will tumble over the wall and plunge to the ground under the force of gravity. The quantity of water will vary in time, and the unsteady discharge will be a function of wave height, wave period, and surge elevation relative to the wall. Erosion of unprotected soil will occur as the waves cascade over the wall, but the unsteadiness of the process, coupled with the variation of impact point due to irregular waves, makes scour estimation difficult, if not impossible.

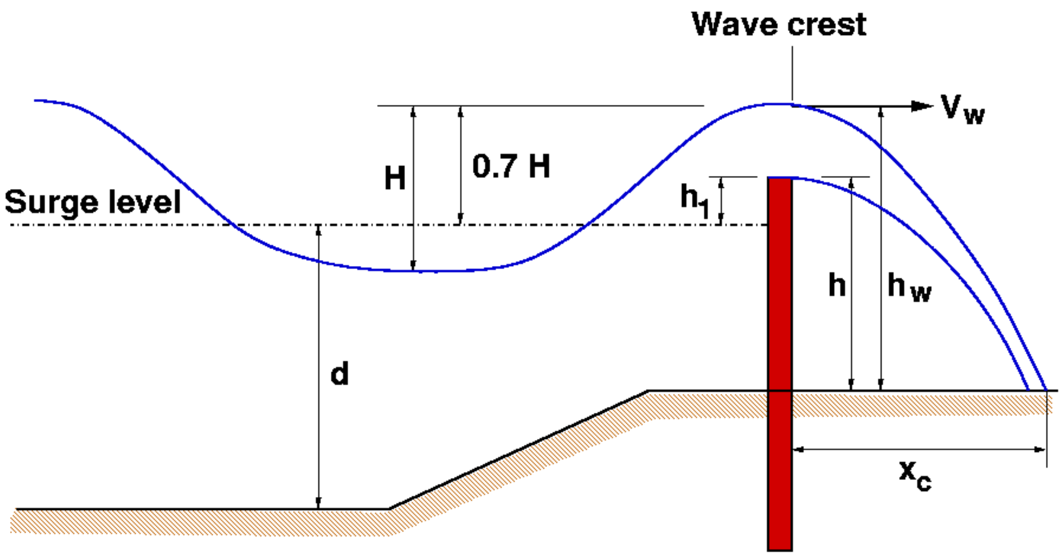

Figure 4.14. Definition sketch of wave overtopping floodwall

The hydrodynamics of this phenomenon is quite complex because a substantial portion of the incident wave is reflected by the floodwall, and the reflected wave will interact nonlinearly with the incident wave. Therefore, a few simplifying assumptions are necessary for the approximation given here.

Assume the incident waves are reasonably approximated as shallow water waves. Furthermore, assume the incident wave crest height reaches the floodwall without being modified by the reflected wave. In other words, there is no nonlinear interaction between the incident and reflected wave. Waves in deeper water are symmetrical about the still water level (swl) with the vertical distance between the wave crest and swl is the same as the vertical distance between the wave trough and swl. However, in shallow water the wave crests become more peaked and the troughs become flatter, and the vertical distance between the wave crest and the swl becomes proportionally larger. For this simple development, assume the distance of the wave crest above the swl is 70% of the wave height, H, as shown in Figure 4.14.

As the wave crest passes over the floodwall, the orbital velocity of water particles at the free surface will be nearly the same as the wave celerity. Using the expression for wave celerity given by third-order theory for nonlinear, shallow water waves, the horizontal velocity V_w is given by

$$V_w = C = \sqrt{g(d+H)} \tag{4.24}$$

where g is gravity, d is water depth, and H is incident wave height. Note that wave celerity is independent of wave period in shallow water, and instead depends only on water depth and wave height.

The distance from the wall to where the plunging wave crest impacts the ground level is found using the formulas for an object in free fall having an initial horizontal velocity of V_w and falling a vertical distance h_w. The total vertical fall distance is given as

$$h_w = h + 0.7\,H + h_1 \tag{4.25}$$

where h is the vertical distance between the top of the flood wall and the ground level, and h_1 is the distance between the top of the wall and the surge level. If the surge level is lower than the floodwall, h_1 is negative. When the surge overtops the floodwall, h_1 is positive.

The vertical fall distance is a function of fall time and gravitational acceleration, i.e.,

$$h_w = \frac{1}{2} g\, t^2 \tag{4.26}$$

Thus, the fall time for a water particle at the wave crest free surface to fall to the ground level is given by

$$t_f = \sqrt{\frac{2\,h_w}{g}} \tag{4.27}$$

The horizontal distance traversed by the water particle during this free-fall time is simply

$$x_C = V_w\, t_f \tag{4.28}$$

Substituting Eqn. 4.24 for V_w and Eqn. 4.27 for t_f into Eqn. 4.28 yields

$$x_C = \sqrt{2\left(d + H\right)\left(h + 0.7\,H + h_1\right)} \tag{4.29}$$

Figure 4.15 shows the variation in impact distance from the floodwall as a function of surge elevation relative to floodwall elevation for different floodwall heights above the ground level. These curves were calculated using Eqn. 4.29 with a wave height of $H = 4$ ft, and a water depth of $d = 16$ ft. Different curves should be generated for other values of H and d.

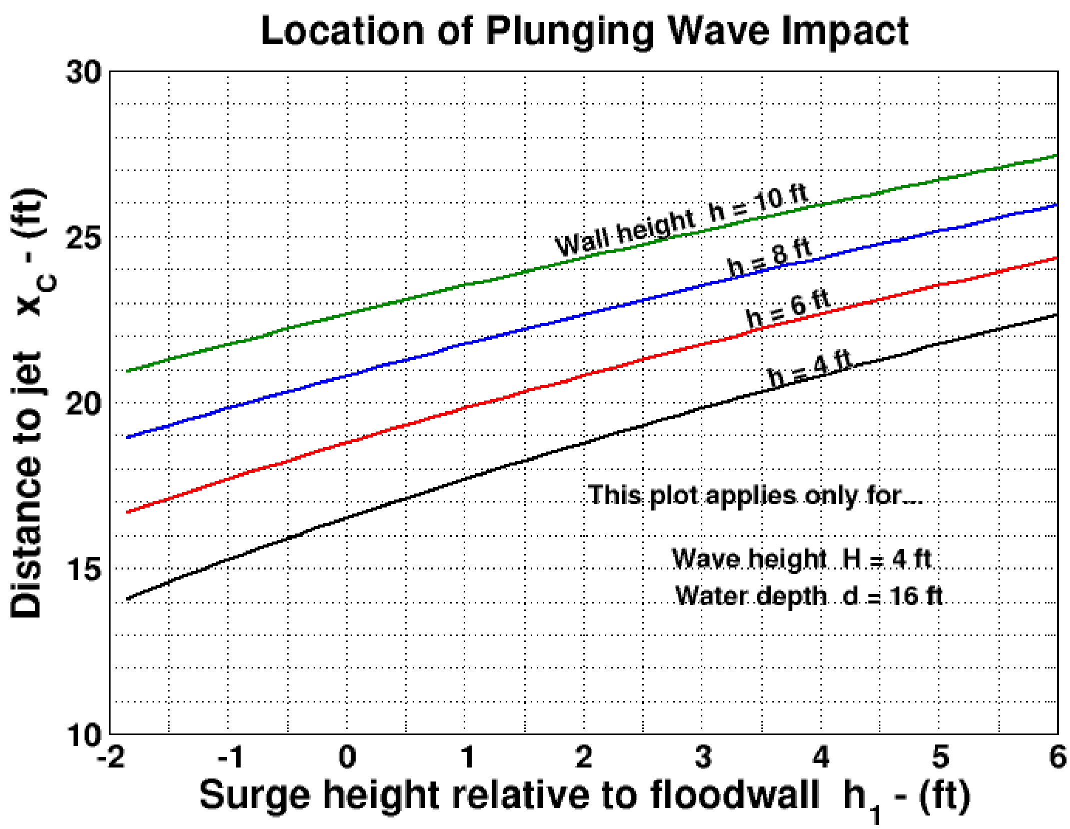

Figure 4.15. Horizontal distance between the floodwall and approximate impact point of plunging wave crest

The horizontal distance between the floodwall and the plunging wave impact point is appreciably farther than corresponding distances for surge overtopping without waves as estimated from Figure 4.9. This difference is due to the forward speed of the wave crest, which is greater than the fluid velocity of the overtopping surge. If the elevation of the surge level is substantially below the floodwall top elevation, only the highest waves will overtop the wall, and the quantity of overtopped water will be relatively small. As the surge level rises, more of the wave crests will topple over the wall, and the likelihood of scour damage increases.

Depending on the cross section of the earthen levee supporting the floodwall, the horizontal projection of the overtopping jet may over-shoot the crown of the earthen levee and impinge on the protected side slope. It this case it is a simple matter to continue the parabolic trajectory used in this analysis to estimate the point of impact on the rear slope. The easiest procedure is trial and error solution of Eqn. 4.29 until values of x_C and h correspond to the surface of the levee protected side slope.

Wave and Surge Overtopping

Where both waves and storm surge overtop the floodwall the hydrodynamics are complex, and the simple methods provided here are less valid. More research is needed to establish accurate hydrodynamic design criteria. Steady overflow associated with the storm surge elevation above the top of the floodwall is combined with the unsteady waves propagating on top of the surge. This results in a pulsating unsteady flow over the wall with larger discharge when the wave crest passes over the wall, and decrease discharge when the wave trough is at the wall. This pulsating action affects the location of the free-falling water jet in time with the jet landing farther from the floodwall with greater flow volume when the wave crest overtops. Consequently, scour protection for the case of wave and surge overtopping must be more robust then needed for surge overtopping alone, and the protection must extend a greater distance from the protected side of the floodwall.

A first approximation of the maximum jet impact horizontal distance from the wall can be estimated using Eqn. 4.29 with h_1 specified as the distance between the surge elevation and the top of the floodwall (positive value). The actual impact distance may be slightly farther because the overtopping flow could add to the initial horizontal velocity (V_w) of the wave. The maximum impact force of the falling jet will be greater than that estimated for surge overtopping alone (see Figure 4.13).

Performance Criteria

Below are listed the key performance criteria pertinent to protection for vertical floodwalls and sheetpile walls. Many of the criteria given below are nearly identical to those given in earlier chapters. At this stage much of the performance criteria are in the form of questions related to various aspects of armor and protection performance. Some question responses may yield specific answers based on test results and/or previous experience, whereas answers to other questions may result in assigning a value such as poor, fair, good, excellent, or unknown. Performance criteria will continue to evolve as additional information is gathered.

Survivability Considerations

Survivability of floodwall toe protection on the protected side can be divided into two categories. The first category is survivability of the protection over the relatively short duration of a major hurricane event when the floodwall is overtopped and large quantities of free-falling water impact the ground with substantial force. Wave and water overtopping will cause maximum destructive loading on the protective system, and thus, constitute the critical design condition.

Evaluation of potential armoring or protection alternatives for the overtopping category should determine which of the following scenarios best describes how the system will respond to a major overtopping event where the storm surge level exceeds the top of the floodwall for as much as three hours, and the flow parameters are within the ranges estimated in the previous section.

a) The protection system is expected to survive intact with only minor damage that does not endanger the floodwall's integrity and does not result in a significant loss of foundation material that provides lateral support. Repairs may be needed, but the repairs are not urgent and can be scheduled as resources allow.

b) The protection system suffers damage; but the damage is progressive in time, and more importantly, the loss of foundation material does not ultimately result in loss of lateral support and floodwall displacement or collapse. In other words, the floodwall has sustained some damage to the scour protection and considerable loss of foundation material, but the wall remains intact through the duration of the event. Immediate repairs must be undertaken as soon as feasible.

c) The protection system holds for a while, but then fails in a catastrophic manner with nearly complete loss of protective functionality. Foundation soil will erode as if unprotected, and the floodwall is at risk as lateral supporting soil is removed. The floodwall must be repaired, and a nearly complete reinstallation of the protection is required.

The second survivability category pertains to long-term deterioration of the scour prevention system or some of its components, even in the case where the floodwall is not exposed to overtopping flow for many years. Factors that may be important include material degradation, adaptation to differential settlement of the earthen levee supporting the floodwall, gradual stress loading on components during settlement, burrowing by small mammals, and tolerance to unintended plant growth within the protective system.

Long-term survivability assumes necessary monitoring and maintenance is performed as recommended (see below).

Geotechnical Considerations

Armoring is the only practical solution for preventing scour caused by water and waves overtopping a vertical floodwall. Soil strengthening techniques and some products designed to help soil embankments resist lateral flow will most likely not withstand the direct nearly-vertical impact of the overtopping water jet. The following are the main geotechnical considerations related to armoring the levee crest on the protected side of vertical floodwalls and sheetpile walls.

a) Bearing capacity. The soil must have adequate bearing capacity to support the overlaying scour protection without significant differential settlement.

b) Soil retention. The scour protection system must be designed to prevent foundation soil from leeching out between voids in the protective layers. Excessive loss of soil could result in localized collapse of the scour protection that might rapidly spread. If a geotechnical filter fabric is placed under the protection system, it must relieve any built-up pore pressure.

c) Erosion at protection termination and tie-in locations. Ideally, the scour protection will continue some distance farther away from the wall and eventually either terminate or tie into slope protection. Where no tie in to slope protection exists, the soil abutting the protection must have sufficient strength to resist the erosive effect of the overtopping water runoff.

Construction/Installation Considerations

The following list provides the more important considerations related to installation of scour protection systems on the protected side of existing undamaged and repaired vertical floodwalls. The items are not listed in any particular order of importance.

a) Design modification. Does the scour protection method require modifying the floodwall design to accommodate the armoring system? For example, is the added weight of the protection system such that underlying soils will compact resulting in loss of levee height through settlement.

b) Site access. Some portions of the existing levee and floodwall system may have limited access for heavy equipment, or for transporting materials to the work site. What site access and maneuverability are required to install a particular protection system?

c) Equipment requirements. Are there any special equipment requirements to install a particular system that might be considered out of the ordinary? If so, how might this impact construction schedule and cost per installed area?

d) Installation skills. Are there any particular or unusual skills required to install a particular system successfully? If so, what are these skills, how can these skills be obtained by the work force, and what construction monitoring and oversight are needed to assure competent installation?

e) Installation tolerances. Does successful system installation depend on precise placement of system components? If so, what are the tolerances, what methods are used to attain accurate placement, what onsite oversight and inspection are required, and what are the consequences if tolerances are not met?

f) Rate of installation. How much scour protection can be installed in an average work week, and what are the parameters associated with this rate (personnel, equipment, etc.)?

g) Protection termination and tie-in locations. The peripheral boundaries where the protection system terminates or joins with some other form of protection are often where initial damage occurs. Scour protection must extend away from the floodwall a sufficient distance to cover the region where direct water jet impact is expected. However, overtopping water will flow laterally after impact, most likely flowing down the earthen levee slope on the protected side. This flow will have high velocities, and some of the same slope protection concepts discussed in Chapter 3 apply here. Relatively light-weight scour protection systems should be affixed to the side of the floodwall to prevent possible dislocation by uplift forces. How does the particular protection system deal with transition points? What is the recommended extent of protection coverage to assure no problems will arise at the transition between protection and no protection? Is it possible and advisable to reinforce the boundaries with a more robust form of armoring (e.g., at the toe where head cutting is likely to initiate)?

h) Immersion effects. Are there any adverse consequences arising from immersion of the scour protection? If local topography is such that overtopping water can pond immediately behind the floodwall, the immersed weight of the scour protection will be considerably less than the dry weight (less than half for concrete). The impinging jet will have reduced impact force, but the capability of the protection to resist the force by self-weight is significantly reduced.

i) Construction staging. What is the construction sequence for a particular scour protection system, and does this have any effect on installation. For example, can some system components be prefabricated offsite, and then transported to the construction site by the most economical means?

j) Safety. What are the safety concerns and issues associated with a particular protection system? Will special precautions or training be needed, and what is the plan to assure all safety measures will be strictly implemented and enforced?

k) Removable and reinstallation. It is anticipated that some levee crest elevations and associated floodwalls may need to be increased by addition of earthen material to compensate for settlement or to increase the level of protection. In such an event, can a particular scour protection system be removed and reinstalled? The reinstalled protection must provide the same level of protection afforded by the original installation, and most of the cost will be associated with re-handling the armoring and not purchase of large quantities of new armoring materials

Maintenance and Repair Considerations

Long-term maintenance of the floodwall foundation scour protection is paramount for assuring continual integrity of the southeast Louisiana levee system. Without proper inspection, maintenance, and repair of deteriorated or damaged sections of floodwall scour protection, risk of damage from hurricanes weaker than the design storm increases. Below are evaluation considerations related to maintenance and repair of armoring systems.

a) Maintenance requirements. What are the specific maintenance requirements for a particular scour protection system? Is special equipment or specific skills required for ongoing maintenance?

b) Timing for maintenance. Is maintenance for a particular system performed at regular intervals, or only when needed as determined by inspection?

c) Inspection. How often is inspection recommended for a particular protection system? What is the recommended inspection technique? Which aspects of the system should be inspected? How much of the protection can be inspected in a day? Are any special tests or testing apparatus required to conduct inspections?

d) Signs of deterioration. What are the signs that a protection system is deteriorating, and can these signs be readily detected during inspection? What are the indicators that maintenance needs to be performed?

e) Maintenance costs. What costs are associated with maintenance beyond personnel time? For example, does usual maintenance require a significant mobilization of equipment?

f) Damage repair procedures. After episodes resulting in significant or wide-spread damage, what are the repair procedures? How is the repaired section tied into the adjacent undamaged protection? Can small sections of isolated damage be repaired by a small crew using readily obtained equipment?

g) Robustness of repair. Will repaired sections of damaged protection retain the full strength and resistance to damage as the original installation, or will the repair section represent a weakened area that may require additional strengthening?

h) Safety during inspection, maintenance, and repair. Are there any safety concerns or safety procedures specific to a particular protection system? Are there any additional risks working near a damaged portion of the protection beyond those that could be reasonably identified or anticipated?

Environmental Considerations

The following is a list of considerations related to environmental consequences that might apply to some scour protection alternatives.

a) Environmentally sensitive areas. Are there any aspects of the scour protection system that might make it difficult to deploy on floodwalls located in environmentally sensitive areas?

b) Toxic materials. Does the protection system contain any toxic materials or chemicals that might be released into the environment either during installation or over time due to deterioration? Are there any special treatments or handling considerations to assure no toxics are released?

c) Endangering animals or plant species. Are there any aspects of the protection system that might be considered detrimental or dangerous to local plant and animal species?

Overview of Concrete and Sheetpile Floodwall Protection

The forceful, near vertical, impact of falling water due to surge and wave overtopping at vertical floodwalls imposes loads on the protection system that are vastly different than loads exerted by water flowing parallel to the protection surface. As a consequence, armoring systems fully capable of protecting backside slopes of levees and earthen levee transitions may not be appropriate for protecting the levee crown soil on the protected side of an overtopped floodwall. For example, individual stones will be dislodged in riprap protection, turf reinforcement mats might not withstand forces applied perpendicular to the mat, and soil or small stones used as geocell fill will be flushed out by the water.

This section briefly overviews four protection alternatives that have sufficient strength, rigidity, and robustness to withstand high impact loads from overtopping water jets without loss of functionality. All the options have the disadvantage of adding significant weight to the levee foundation, and this could be problematic where soil is weak. The following are considered to be viable alternatives for armoring floodwalls on the protected side:

a) Poured-in-place reinforced and non-reinforced concrete

b) Grouted stone riprap

c) Rock-filled mattresses

d) Articulated concrete mats

Below are brief generic descriptions of these protection systems.

Poured-in-place reinforced and non-reinforced concrete. Levee soil can be protected by an impermeable, continuous, reinforced concrete slap containing light reinforcement mesh. Alternately, the concrete slab can be made thicker without reinforcing. The slab is formed, and concrete is poured in place to cover the area from the base of the floodwall protected side out a distance beyond the expected splash-down point of the overtopping jet. The slab can be tied into the floodwall using a variety of techniques. This provides a rigid horizontal surface that can absorb the impact of falling water and divert the overtopping jet toward the backside slope of the earthen levee. Advantages include high strength and durability, readily available materials, and flexibility to vary project dimensions as needed. Where appropriate, the concrete apron can be designed as a roadway for vehicular traffic. The main disadvantage of reinforced and non-reinfoced concrete is its relative intolerance to differential settlement. Where future plans call for addition of levee height, concrete aprons cannot be easily removed and re-used.

Grouted stone riprap. This protection method consists of conventional riprap armoring placed on top of a bedding layer and then filled with a concrete grout mixture. The purpose of the grout is to solidify the riprap protection into a solid, continuous, impermeable structure and to prevent loss of individual stones when impacted by the falling water jet. Because the grout mixture has minimal strength in tension, grouted stone riprap will have little tolerance for differential settlement of the underlying levee crown. Once the bond between adjacent stones is broken, riprap stones can be dislodged by the overtopping flow. Advantages of grouted stone riprap are ease of installation, capability to protect varying terrain, and ease of removal for future increases in levee height. However, the removed riprap is not readily re-usable because much of the grout will remain intact. The main disadvantage of grouted stone riprap is the uncertainty associated with the long-term integrity of the grout/stone bonds if there is any ground settlement.

Rock-filled mattresses. Rock-filled mattresses are containers fabricated of geogrid material and filled with small rocks varying in size from 2 inches up to about 5 inches. Mattresses are placed directly on top of a geotextile filter cloth or conventional gravel filter layer. Rock-filled mattresses are flexible, and they can adapt to terrain changes easily. They are also tolerant of differential settlement, and they will continue to be fully functional if the ground settles beneath them. Overtopping water landing on the mattress fills the voids between stones and helps reduce the flow energy. Soil could be placed over the mats to support vegetative growth. For application at the base of floodwalls, special attention is needed to assure mattresses are placed with minimal gaps between adjacent units. Gaps between mattresses are weak points that could allow soil to escape if the geotextile is punctured. Advantages of rock-filled mattresses include lower cost for smaller stone, rapid installation, off-site fabrication, and the capability to remove the protection and re-use the mattresses if the levee needs to be raised. Disadvantages of rock-filled mattresses include the need for heavy equipment to lift and place the mats, potential gaps between adjacent mats and next to the floodwall, and long-term durability of the geogrid material when subjected to UV radiation. Whereas the mats could support vehicular traffic, there is a risk of damaging the geogrid material or the lacing that holds the mats together.

Articulated concrete mats. Articulated concrete mats consist of concrete block units linked together with cables made of metal or other high-strength material. Blocks can be solid or open, with gaps between adjacent blocks. Articulated concrete mats are fabricated off-site and rapidly installed using heavy lifting cranes. The concrete blocks have sufficient strength to resist the battering of overtopping jets of water, but the gaps between the blocks could allow underlying soil to erode. Therefore, these mats will be most effective if placed over a stone or gravel bedding layer sized to prevent movement of the gravel through the gaps in the mat. Articulated concrete mats are flexible and very tolerant of differential settlement. The mats are easily removed and re-used without any loss of effectiveness, and they have no problem supporting low-speed vehicular traffic. Advantages of articulated concrete mats include off-site fabrication, rapid placement, capability to cover irregular terrain, tolerance to differential settlement, and long service life. Disadvantages include the need for heavy-lift cranes during installation and providing adequately-sized gravel underlayers to prevent loss of material through gaps.

Alternative: Poured-in-Place Reinforced and Non-Reinforced Concrete

1. Manufacturer.

   No specific manufacturer.

2. Product Description.

   Poured-in-place concrete provides effective armoring of the levee crown soil on the protected side of a vertical floodwall. The concrete apron is formed, and concrete is poured in place to cover the area from the base of the floodwall protected side out a distance beyond the expected splash-down point of the overtopping jet. Concrete offers great flexibility for protecting odd-shaped areas, gaps between the floodwall and existing structures as shown in Figure 4.16, and around corners in the floodwall protection. Reinforced concrete slabs can be thinner because the reinforcing mesh resists tension loads. The slab can be tied into the floodwall using a variety of techniques. Details of reinforced and non-reinforced concrete aprons are shown on Figures 4.16 and 4.17, respectively. These specific plans are being implemented by Task Force Guardian.

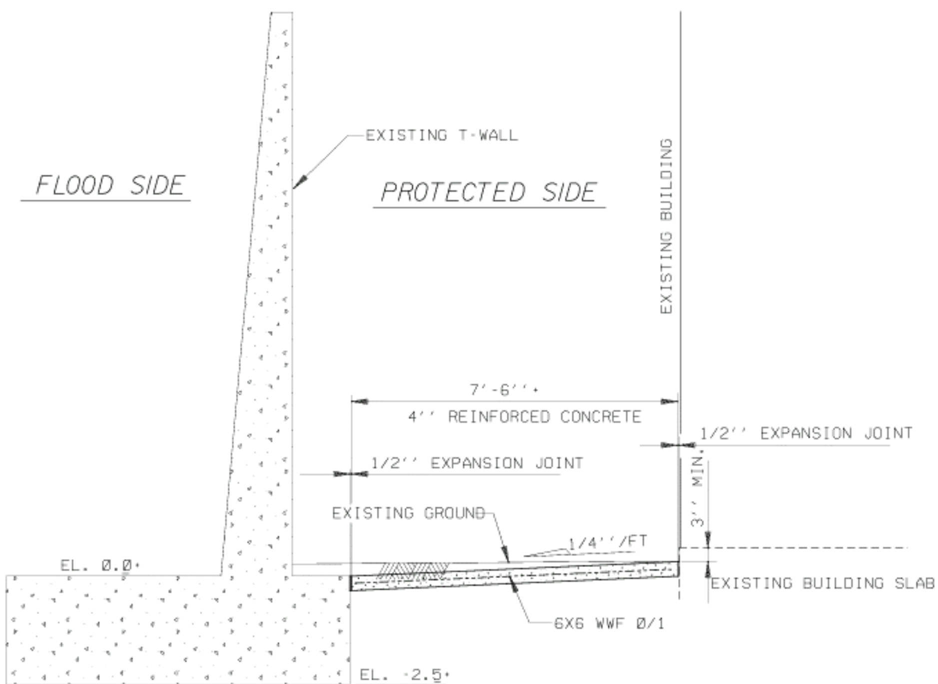

Figure 4.16. Detail of 4-inch-thick reinforced concrete apron (from URS drawing for IHNC West side)

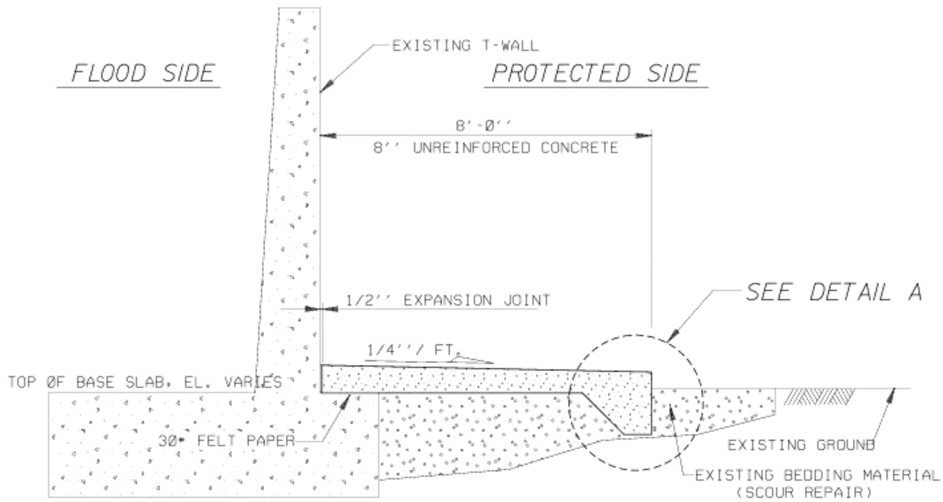

Figure 4.17. Detail of 8-inch-thick unreinforced concrete apron (from URS drawing for IHNC West side)

3. Product Functionality.

   Levee soil is protected by an impermeable, continuous, concrete slap with or without light reinforcement mesh. This provides a rigid, nearly horizontal surface that can absorb the impact of falling water and divert the overtopping jet toward the backside slope of the earthen levee.

4. Stated Applications.

   Applications of formed and poured-in-place concrete to control flow and prevent scour are wide spread and very successful. Implementations illustrated in Figures 4.16 and 4.17 are most appropriate, and these designs should be fully successful under design load conditions.

5. Potential Failure Modes and Mechanisms.

   The loads to which the concrete slab might be subjected are not well defined, and this makes design of the slab difficult. If the slab remains on firm footing with no loss of underlying material, loads generated by the falling jet of water should be transferred to the foundation. However, if the ground beneath the slab settles, there may be locations where the slab spans a void and must function like a beam. The slab will crack if the reinforcement mesh is not near the bottom, and this could lead to partial breakup of the slab. Alternately, if a portion of the slab is cantilevered by loss of supporting material at the outer edge, the reinforcement mesh is then needed near the top surface of the slab.

6. Application Limitations.

   There are few limitations on poured-in-place concrete slabs. Near full strength is attained in about one month, and strength continues to increase slowly for some time.

7. Documented Applications.

   Numerous.

8. Costs.

   Cost is a function of project location, site accessibility, coverage, slab thickness, and reinforcement. Preparation costs will vary. The experience of Task Force Guardian should provide an idea of installed costs.

9. Technical Evaluation Relative to Performance Criteria.

   a) Survivability Criteria. Concrete has excellent survivability characteristics. Properly designed slabs should withstand the dynamic forces, and the relatively short duration of overtopping events precludes erosion of the concrete surface. Properly prepared concrete is durable, and it weathers well.

   b) Geotechnical Criteria. Concrete provides an impermeable barrier, so any loss of underlying soil will be at the slab boundaries or perhaps through the activities of burrowing animals. The underlying soil must provide adequate bearing capacity for the slab (and any anticipated vehicular traffic) without differential settlement.

   c) Construction/Installation Criteria. Cracks will form during the concrete curing process, so steel mesh must have sufficient coverage so corrosion does not occur. Steel corrosion will cause spalling and a reduction in slab width. Usual practices must be followed as with any poured concrete slab, e.g., water should not be added to increase the concrete flow characteristics during placement, etc. Expansion/contraction joints are necessary, and it may be advisable to tie the slab into the existing floodwall.

   d) Maintenance and Repair Criteria. Concrete requires little maintenance. If inspection indicates an area of deteriorating concrete due to corrosion of reinforcement or spalling due to poor quality materials, those sections should be cut out and replaced with new concrete.

   e) Environmental Criteria. Concrete slabs do not cause any environmental problems. Site access may disrupt the local ecology temporarily.

   f) Design Requirements. Conventional concrete slab design for typical dead and live loads is well understood and dictated by building codes. Slab resistance to the impact loading of falling water caused by wave and surge overtopping is not as well understood. An initial estimate of the total force in the water jet (per unit length along the floodwall) is provided by Figure 4.13 for the case of surge overtopping. The associated bearing pressure can be estimated using Figure 4.10 to find the jet thickness at impact. Apply the resulting pressure as a live load. It might be prudent to include a factor of safety given the uncertainty of wave overtopping loads.

10. Summary of Poured-in-Place Concrete Alternative.

a) Advantages. Advantages of poured-in-place reinforced and non-reinforced concrete include high strength and durability, readily available materials, and flexibility to vary project dimensions as needed. Where appropriate, the concrete apron can be designed as a roadway for vehicular traffic. Where site access is limited, concrete can be placed using a crane bucket or by pumping short distances.

b) Disadvantages. The main disadvantage of reinforced and non-reinfoced concrete is its relative intolerance to differential settlement. Buckled sections of the paved area are more apt to allow leaking of underlying soil. Where future plans call for addition of levee height, concrete aprons cannot be easily removed and re-used.

c) Risk and uncertainties. The suggested method for estimating the live loads due to overtopping water are approximate, and wave overtopping has not been included. The estimated load is considered a live load, but the impact force created by initial splash-down of the jet is not included in the force estimate.

Alternative: Grouted Stone Riprap

1. Manufacturer.

   No specific manufacturer.

2. Product Description.

   This protection method begins with conventional riprap armoring placed on top of a bedding layer and geotextile filter fabric. The voids in the riprap are then filled with a concrete grout mixture. The final protection is a solid, impermeable protection layer. Figure 4.18 below illustrates typical project dimensions for rehabilitation of scour holes caused by floodwall overtopping during Hurricane Katrina.

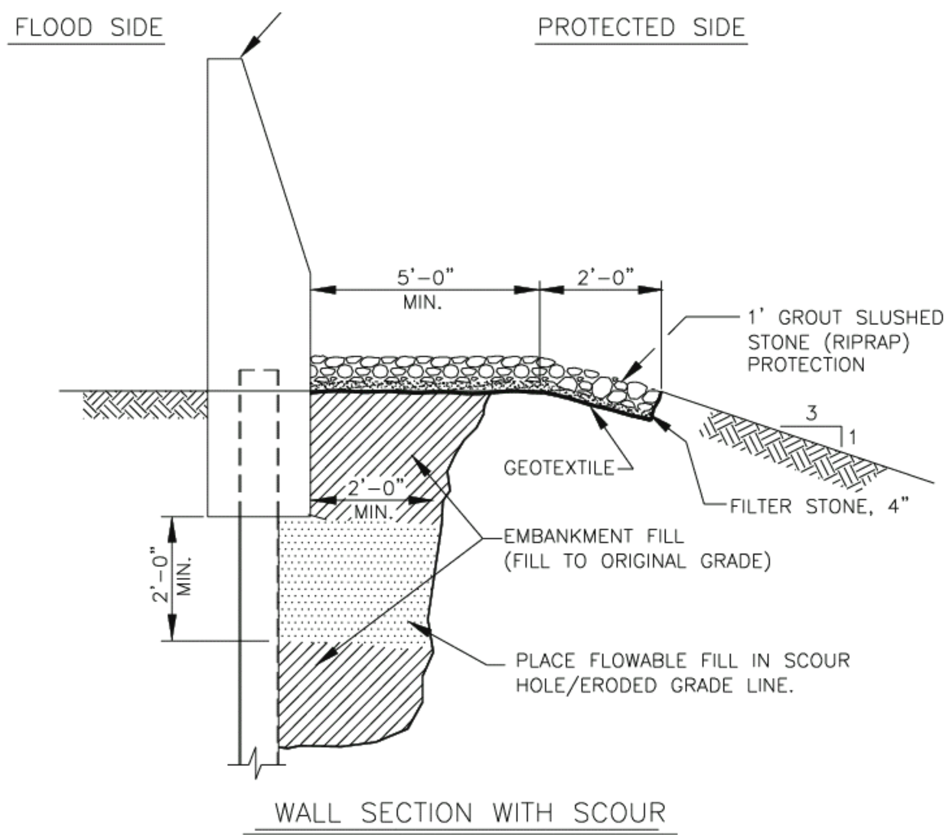

Figure 4.18. Detail of grouted stone riprap floodwall apron (from URS drawing for IHNC East side)

3. Product Functionality.

   The purpose of the grout is to solidify the riprap protection into a solid, continuous, impermeable structure, and to prevent loss of individual stones when impacted by the falling water jet. Whereas the grouted riprap might support vehicular traffic, the risk of damage is too great, and vehicles should be banned from driving on the protection. The underlying soil is shielded from the forces of falling water, and the only loss of soil might occur at the project boundaries if steps are not taken to prevent erosion.

4. Stated Applications.

   Grouted riprap has been used successfully at numerous locations as protection against water flowing parallel to the armoring. It is not known whether or not grouted riprap has been used where high quantities of overtopping water are expected to impact with forces normal to the slope.

5. Potential Failure Modes and Mechanisms.

   Because the grout mixture has minimal strength in tension, grouted stone riprap will have little tolerance for differential settlement of the underlying levee crown soil. Once the bond between adjacent stones is broken, riprap stones can be dislodged by the overtopping flow, and this could start an unraveling of the protection. Poor quality grout will be ineffective and easily broken by the force of water impact. Deterioration of grouted riprap is expected to occur more rapidly than for concrete slabs. Grouted riprap will not expand and contract with temperature change as much as concrete, but expansion and contraction might cause the grout to crack and break.

6. Application Limitations.

   Grouted riprap should not be used where foundation conditions cannot support the weight of the protection or where different soil types might cause differential settlement of the monolithic protection. It would be advisable to have expansion/contraction joints between the riprap and the floodwall, and expansion/contraction joints perpendicular to the floodwall at given spacing.

7. Documented Applications.

   The report authors are not aware of documented cases of grouted riprap used where the protection must resist high volumes of falling water, but that does not mean such applications do not exist. Grouted riprap has been successful in numerous other applications where water flows parallel to the protection.

8. Costs.

   Costs for grouted riprap are unknown, but the experience of Task Force Guardian's implementation of similar protection in the reconstruction of damaged levees and floodwalls in New Orleans should provide sufficient cost guidance.

9. Technical Evaluation Relative to Performance Criteria.

   a) Survivability Criteria. Survivability of grouted riprap to protect foundation soils against surge and wave overtopping is not proven. The main weakness is the inability of the grout to withstand tensile stresses, and the possibility of individual stones breaking free and becoming dislodged. The long-term durability of grouted riprap will be a function of foundation stability and quality of the cement grout.

b) Geotechnical Criteria. The foundation soil must be strong and well compacted to prevent differential settlement. Steps must be taken at the protection boundaries to prevent erosion of supporting soil. This is critical where the riprap ends on the protected side of the earthen levee. Water flowing down the slope will erode the soil as it passes over the terminus of the grouted riprap.

c) Construction/Installation Criteria. Dumped riprap must be checked for good distribution of riprap material sizes. Avoid hotspots where there is a congregation of smaller stones. Grout must be of high quality and only fluid enough to assure that all the voids in the riprap are filled.

d) Maintenance and Repair Criteria. Sections of grouted riprap can be repaired by replacement of the damaged section. However, this patched area will not be well tied into the neighboring intact section, and this might cause a weakness in the protection.

e) Environmental Criteria. Grouted riprap does not cause any environmental problems.

f) Design Requirements. Guidance on the design and construction of grouted riprap revetments is given in the Corps of Engineers' Technical Letter, "Design and Construction of Grouted Riprap" (Corps of Engineers, 1992).

10. Summary of Riprap Alternative.

a) Advantages. Advantages of grouted stone riprap are ease of installation, capability to protect varying terrain, and easy removal for future increases in levee height. However, the removed riprap is not readily re-usable because much of the grout will remain intact. Grouting provides increased stability for riprap that would be dislodged by the overtopping flow.

b) Disadvantages. The main disadvantage of grouted stone riprap is the uncertainty associated with the long-term integrity of the grout/stone bonds if there is any ground settlement. Also, cracks will form around larger stones, and this could lead to gradual deterioration of the grout bonding.

c) Risk and uncertainties. The main uncertainty of grouted riprap is its resistance to large impact forces associated with overtopping jets of water. There is little evidence of grouted riprap being used for this particular application.

Alternative: Rock-Filled Mattresses

1. Manufacturer.

   Marine Mattress
   Tensar Earth Technologies, Inc.
   5883 Glenridge Drive
   Suite 200
   Atlanta, GA 30328-5363
   (888) 828-5126 Toll Free
   (404) 250-1290 International
   (404) 250-0461 Fax
   www.tensarcorp.com

2. Product Description.

   Rock-filled mattresses, often referred to as marine mattresses, are containers fabricated of geogrid material and filled with small rocks varying in size from 2 inches up to about 5 inches. Mattresses are placed directly on top of a geotextile filter cloth or conventional gravel filter layer. Rock-filled mattresses can be fabricated and filled off-site and transported by truck or barge to the job site. Mattresses dimensions are typically 5-ft wide and up to 35 ft long. Depending on the application, mattress thickness can be as little as 4 inches or as large as 2 ft. For application as floodwall overtopping protection mattress thickness should probably be at least 6 inches thick.

3. Product Functionality.

   Rock-filled mattresses are flexible, and they can adapt to terrain changes easily. They are also tolerant of differential settlement, and they will continue to be fully functional if the ground settles beneath them. Overtopping water landing on the mattress fills the voids between stones and helps reduce the flow energy. Soil could be placed over the mattresses to support vegetative growth. The surface of a rock-filled mattress is not intended for vehicular traffic, and the surface may become a slipping hazard if placed on a slope.

4. Stated Applications.

   Rock-filled mattresses have been used as revetments, scour protection, foundation mats, and for protection at culverts and bridge abutments. The writers are not aware of any applications where rock-filled mattresses were intended to resist the forces of water impacting normal to the mattress.

5. Potential Failure Modes and Mechanisms.

   Rock-filled mattresses fail if the supporting container is breached either by failure of the geogrid material or by failure of the lacing and connectors used to construct the cage. The geosynthetic materials used to construct the mattresses are treated against UV radiation, but the long-term (tens of years) durability of the material is unknown.

Mattress protection can also fail if the mattress is lifted by the hydrodynamic forces and displaced laterally as a unit. This might occur if the mattress is too thin relative to the lifting force. Erosion might occur at the mattress boundaries, but the flexible nature of the mattress allows it to slump into any scour hole and continue to provide a reasonably high degree of functionality.

6. Application Limitations.

Rock-filled mattresses add a considerable weight to the levee crown, and they should not be used where foundation soils cannot bear the additional weight. Heavy equipment is required for installation, so site access is a critical issue.

7. Documented Applications.

Numerous field applications including USACE applications as breakwater and revetment foundation support, contaminated sediment cap, and streambank protection. See U.S. Army Engineers Technical Note ERDC/CHL CHETN-III-72, "Uses of Marine Mattresses in Coastal Engineering" (available at http://cirp.wes.army.mil/cirp/cetns/chetn-iii-72.pdf). There are no documented applications where mattresses were expected to resist the impact forces of falling water. However, marine mattress have been reported to be stable as revetments in waves as high as 8 ft. This condition could have generated breaking wave impacts similar to the impact of surge overtopping a floodwall.

8. Costs.

Initial cost estimates can be derived from the table below that was reproduced from the above-cited Technical Note. Installed costs for rock-filled mattresses depend on such factors as application, proximity and cost of rock-fill material, site accessibility, placement method (land-based or from barge), availability of equipment, and project size.

Table 1 Installed Mattress Cost per Square Foot

Application	**Mattress Placement**	**Mattress Thickness**	**Cost per square foot**
Breakwater construction	In water	12 in.	$15
Riverbank revetment	On land	12 in.	$10
Revetment foundation	In water	6 in.	$13

9. Technical Evaluation Relative to Performance Criteria.

a) Survivability Criteria. Rock-filled mattresses should be capable of withstanding the forces of surge and waves overtopping a vertical floodwall; however, this aspect has never been tested to the knowledge of the report authors. The one weakness might be where adjacent mattresses abut if any gaps are allowed. Water hitting any gaps could rupture the underlying geotextile filter fabric and allow soil to erode. Long-term durability depends on the effectiveness of the geogrid and lacing material UV resistance. Mattresses covered with a layer of vegetated soil should have excellent service life.

b) Geotechnical Criteria. Rock-filled mattresses are heavy, and the levee soil must be able to support the weight of the armoring system. However, the system will probably weigh less than comparable grouted riprap solutions. The flexible nature of the mattress allows them to adapt to differential settlement or local losses of underlying soil. Mattress deployment requires minimal compacting of soil, and soil surface preparation requirements are minimal beyond grooming of the soil in preparation for covering with filter cloth.

c) Construction/Installation Criteria. For application at the base of floodwalls, special attention is needed to assure mattresses are placed with minimal gaps between adjacent units. Gaps between mattresses are weak points that could allow soil to escape if the geotextile is punctured. Mattresses are placed by heavy cranes, and adequate site access is needed. Placement from barges is also an option.

d) Maintenance and Repair Criteria. Ruptures to the mattress containers can be repaired in-situ using a patching technique. Extensive mattress damage is repaired by removing the entire mattress and replacing with a new unit.

e) Environmental Criteria. There are no environmental impacts associated with rock-filled mattresses.

f) Design Requirements. There is ample guidance related to mattress fabrication for best service life, but no design guidance exists suggesting appropriate mattresses thicknesses to resist a given overtopping water force load.

10. Summary of Rock-Filled Mattress Alternative.

a) Advantages. Advantages of rock-filled mattresses include lower cost for smaller stone, rapid installation, off-site fabrication, and the capability to remove the protection and re-use the mattresses if the levee needs to be raised.

b) Disadvantages. Disadvantages of rock-filled mattresses include the need for heavy equipment to lift and place the mats, potential gaps between adjacent mats and next to the floodwall, and long-term durability of the geogrid material when subjected to UV radiation.

c) Risk and uncertainties. Behavior of rock-filled mattresses when subjected to the forces of overtopping water is largely unknown. Whereas the mats could support vehicular traffic, there is a risk of damaging the geogrid material or the lacing that holds the mats together if vehicular traffic is allowed.

Alternative: Articulated Concrete Mats

1. Manufacturer.

   Several commercial manufacturers. For example…

   ARMORTEC
   Mid-South Regional Manager
   301 Pascoe Boulevard
   Bowling Green, KY 42104
   Phone: 270-843-4659
   Mobile: 270-535-3539
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   The Mat Sinking unit of the Corps of Engineers produces articulated concrete mats annually for bank protection on the Mississippi River.

2. Product Description.

   Articulated concrete mats consist of concrete block units linked together with cables made of metal or other high-strength material. Mattress thickness varies between manufacturer and intended application with the thickness range between about 5 to 12 inches. Articulated concrete mats are fabricated off-site and rapidly installed using heavy lifting cranes. Mattresses are laid over a filter layer, typically a geotextile fabric, and adjacent mattresses are interlocked or cabled together to form continuous coverage.

3. Product Functionality.

   The cabling between blocks serves two purposes: (1) the cabling holds the blocks together so they can be lifted as a unit for placement, and (2) the cabling provides additional mattress stability and prevents loss of individual blocks. The concrete blocks have sufficient strength to resist the battering of overtopping jets of water, but the gaps between the blocks could allow underlying soil to erode. Therefore, these mats will be most effective if placed over a stone or gravel bedding layer sized to prevent movement of the gravel through the gaps in the mat. The mats are strong, durable, and they have no problem supporting low-speed vehicular traffic.

4. Stated Applications.

   Articulated concrete mats have been used in a wide variety of applications related to protecting soils from flowing water. They are even appropriate as protection against small waves. It is not readily apparent if concrete mats have been used specifically to resist the forces of overtopping water impact normal to the mat. For use as foundation armoring near the protected-side base of vertical floodwalls, perhaps the most appropriate mat would be similar to those constructed by the Corps' mat-sinking unit. These mats have larger rectangular concrete blocks with fewer gaps. The mats are not as flexible as some of the commercial mats, but this particular application is mostly flat, narrow areas without terrain variation (in contrast to the need for articulation at levee transitions).

5. Potential Failure Modes and Mechanisms.

   Concrete mats should have sufficient self-weight to prevent lifting and lateral shifting. Anchoring is an option for the mats. The main concern is loss of underlying soil through gaps, even if covered with a geotextile that could be breached by the falling water impact. For this reason it is advisable to use mats with larger concrete area and smaller gap area. Mats should be placed over a gravel filter layer with stone sizes greater than the gap width. Cable breakage could result in block displacement and erosion of soil in a localized area, but the damage is not likely to spread without wholesale cable breakage.

6. Application Limitations.

   Foundation soils must be able to support the additional weight of the mats. Coverage pattern (long dimension parallel or perpendicular to the wall) will be dictated by the particular mat geometry.

7. Documented Applications.

   There are numerous successful applications of articulated concrete mattresses used to protect against flow parallel to the mat, including the experience of the Corps of Engineers' Mat Sinking Unit. Experience related to water forces applied normal to the mats is limited to breaking of small waves. Very heavy mats may have been used to prevent scour at dam spillways.

8. Costs.

   Typical costs were unavailable at the time of this writing.

9. Technical Evaluation Relative to Performance Criteria.

   a) <u>Survivability Criteria</u>. Articulated concrete mats are expected to have good survivability characteristics during short-term overtopping events. Even if some of the underlying soil is lost during an extreme event, the mattress protection retains most of its functionality. The mats are very durable over the long term with corrosion of the cabling being the only concern.

b) Geotechnical Criteria. The underlying soil must be able to support the mattress weight without undue differential settlement, and the geotextile filter fabric must provide continuous coverage to retain the soil while relieving built-up pore pressure. The smallest stones in the bedding layer must be larger than the gaps between the concrete blocks.

c) Construction/Installation Criteria. Mattresses are fabricated off-site and delivered by flatbed trucks (or barges) to the site. The mattresses require heavy equipment for installation. When placing the mattresses special attention should be given to minimizing gaps between adjacent mats so bedding stone is not lost.

d) Maintenance and Repair Criteria. Generally, articulated concrete mats require no maintenance. If differential settlement becomes problematic, individual mats can be lifted out, and fill soil can be added and compacted before replacing the mat. If mattress cabling corrodes, the entire mattress can be replaced.

e) Environmental Criteria. Installation of articulated concrete mattresses does not cause any adverse environmental consequences. Mattresses do have aesthetic appeal versus riprap protection.

f) Design Requirements. Individual manufacturers provide design information and installation guidelines. The most important parameter is appropriate mattress thickness because this influences the installed cost of the protection. Unfortunately, no guidance exists at present to make this determination.

10. Summary of Articulated Concrete Mat Alternative.

a) Advantages. Advantages of articulated concrete mats include off-site fabrication, rapid placement, capability to cover irregular terrain, tolerance to differential settlement, and long service life. The mats are easily removed and re-used without any loss of effectiveness.

b) Disadvantages. Disadvantages of articulated concrete mats include the need for heavy-lift cranes during installation and providing adequately-sized gravel underlayers to prevent loss of material through gaps.

c) Risk and uncertainties. As with all the alternatives for protecting the base of floodwalls, the greatest unknown is how the system responds to high impacts of overtopping surge and waves.

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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.

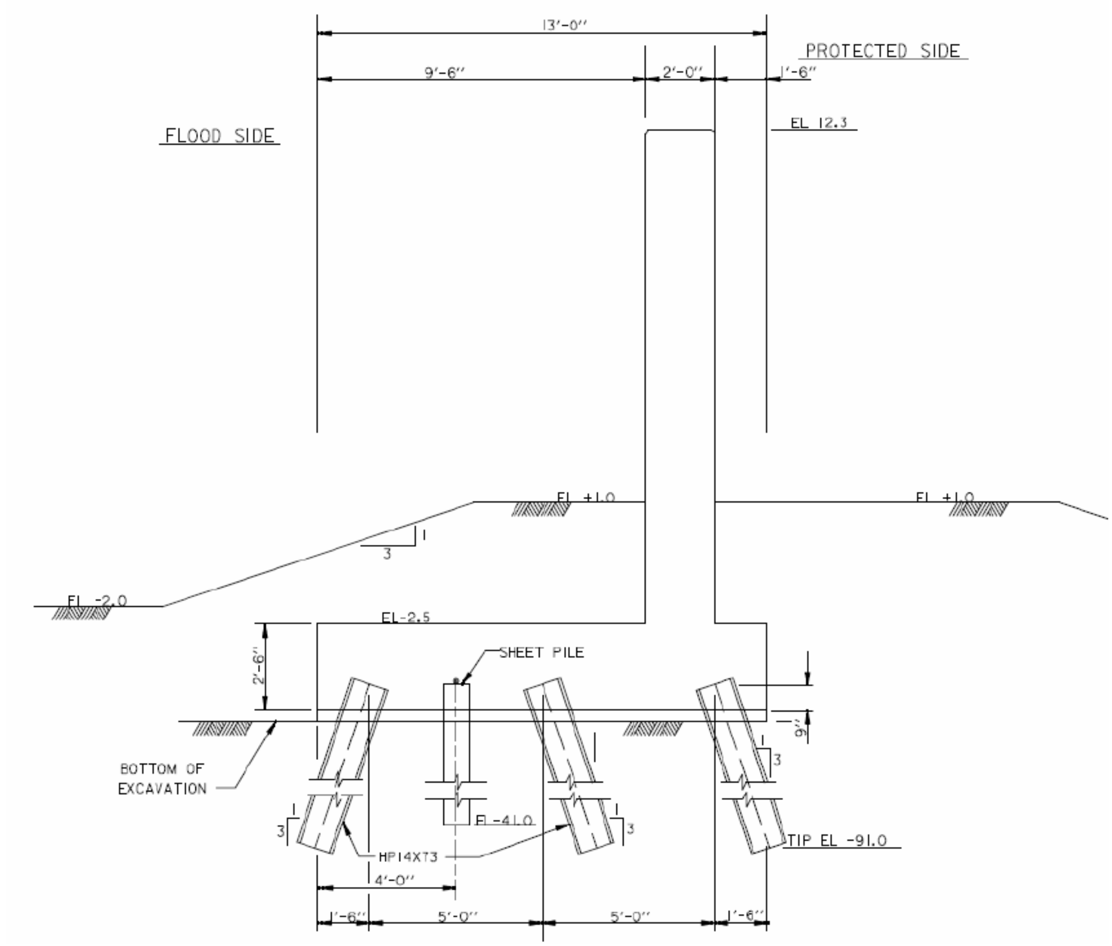


Figure 1. Wall Geometry.

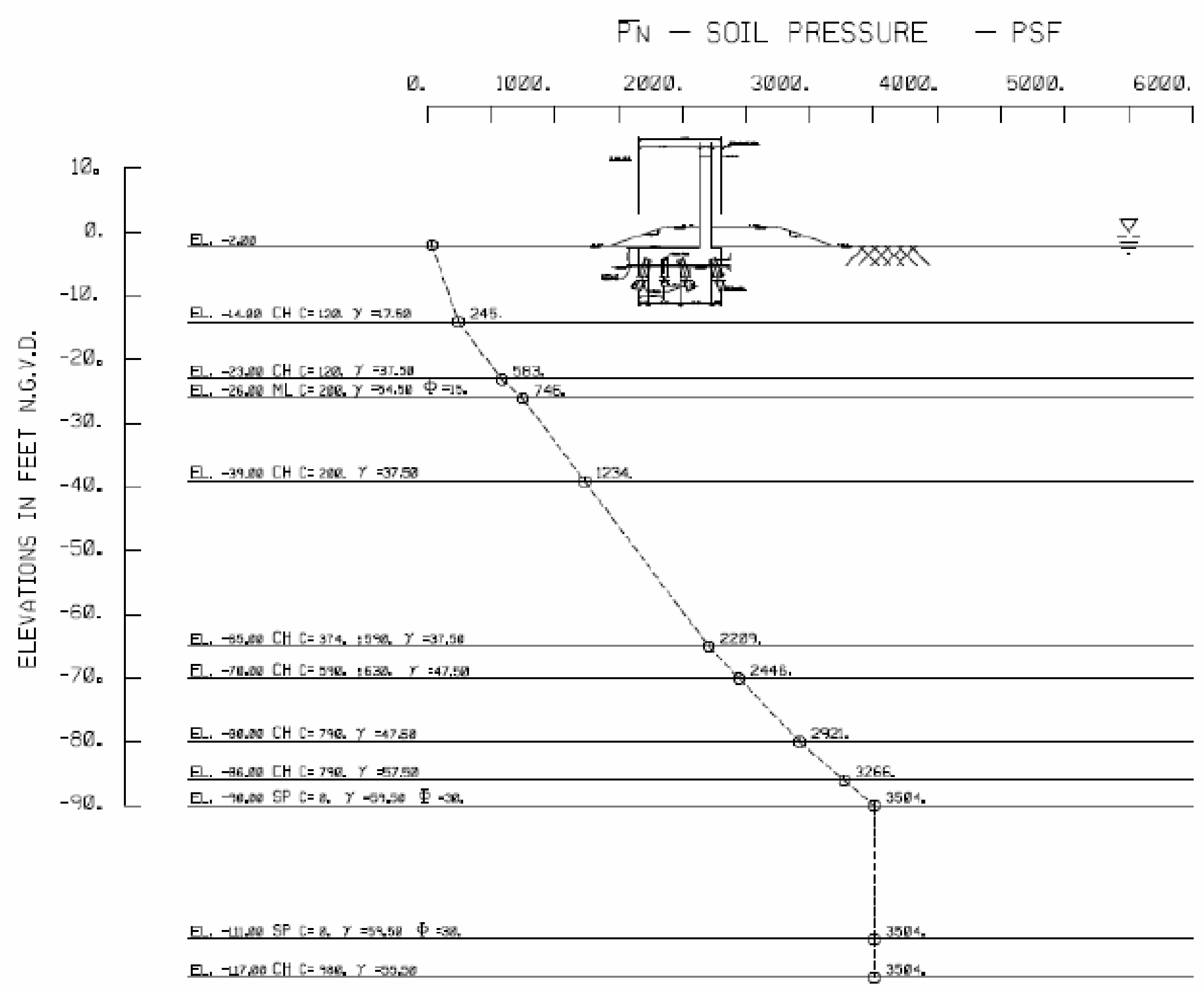


Figure 2. Soil Profile.

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.

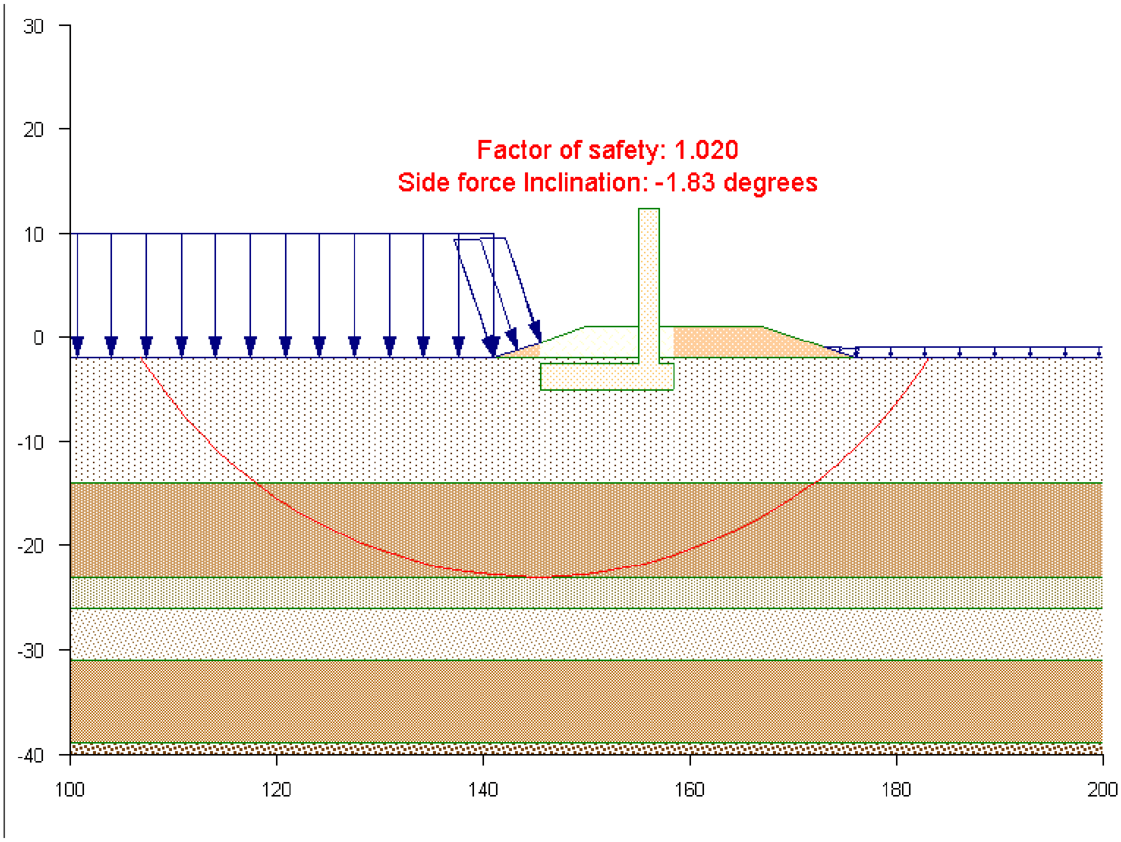


Figure 3. Spencer's analysis of the T-Wall without piles.

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.

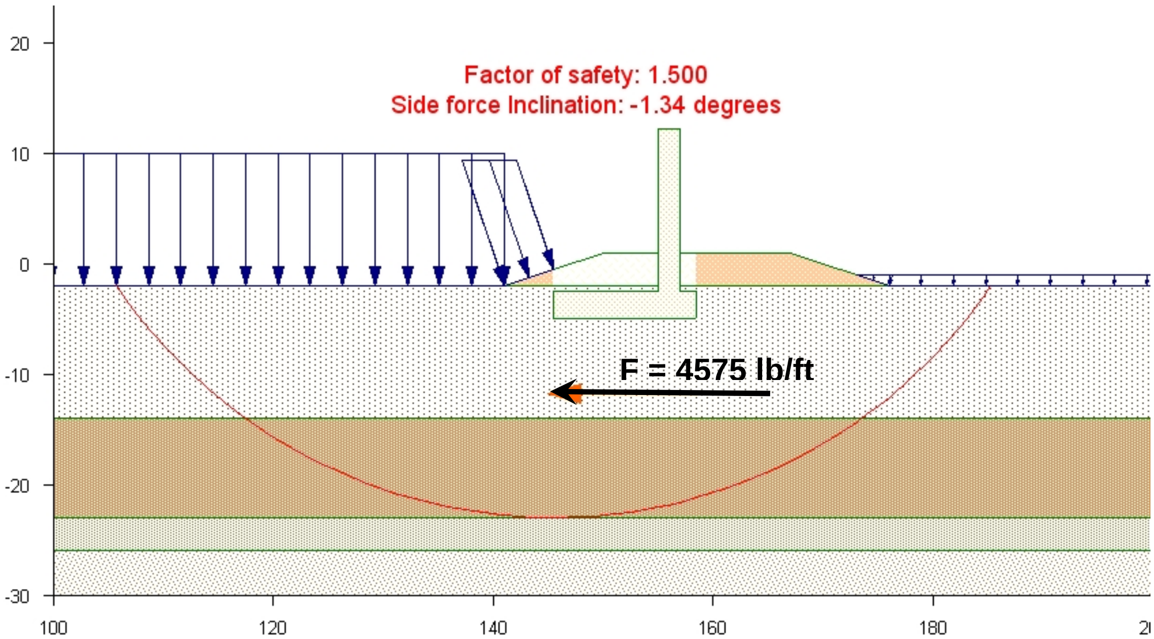


Figure 4. Spencer's analysis of the T-Wall with an unbalanced load to increase global stability.

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.

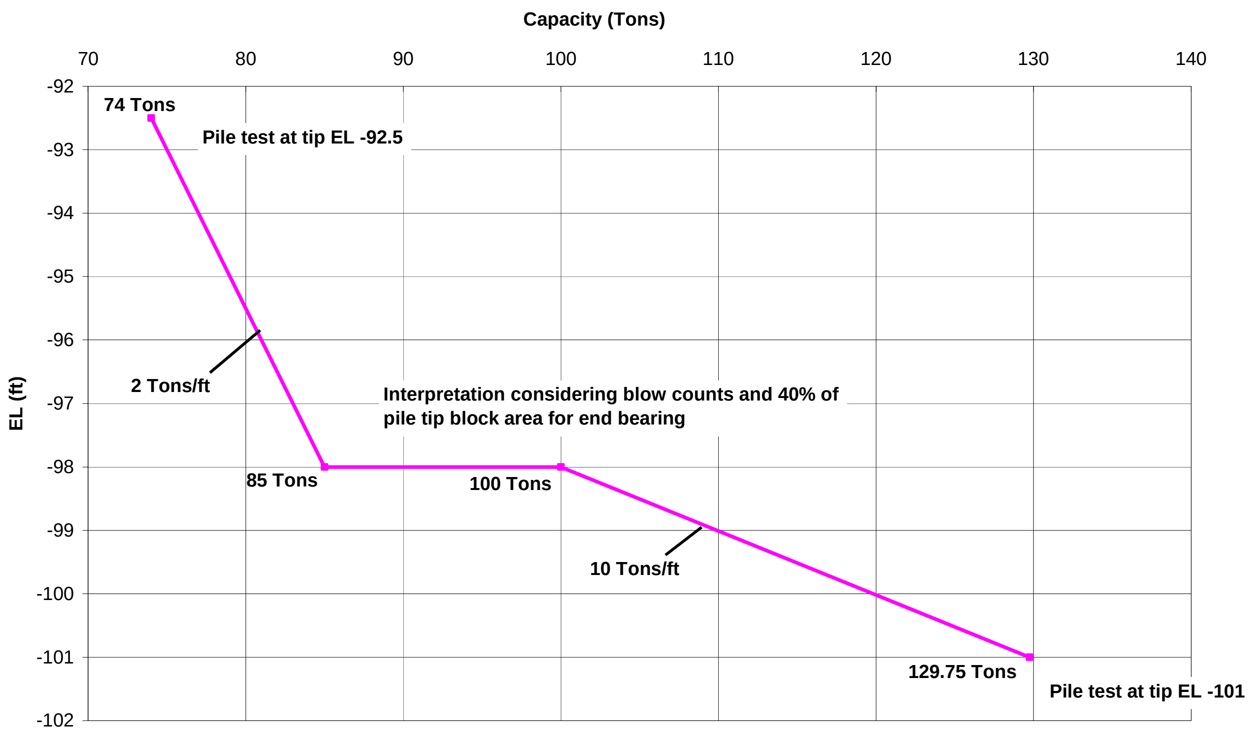


Figure 5. Pile Load Test Data

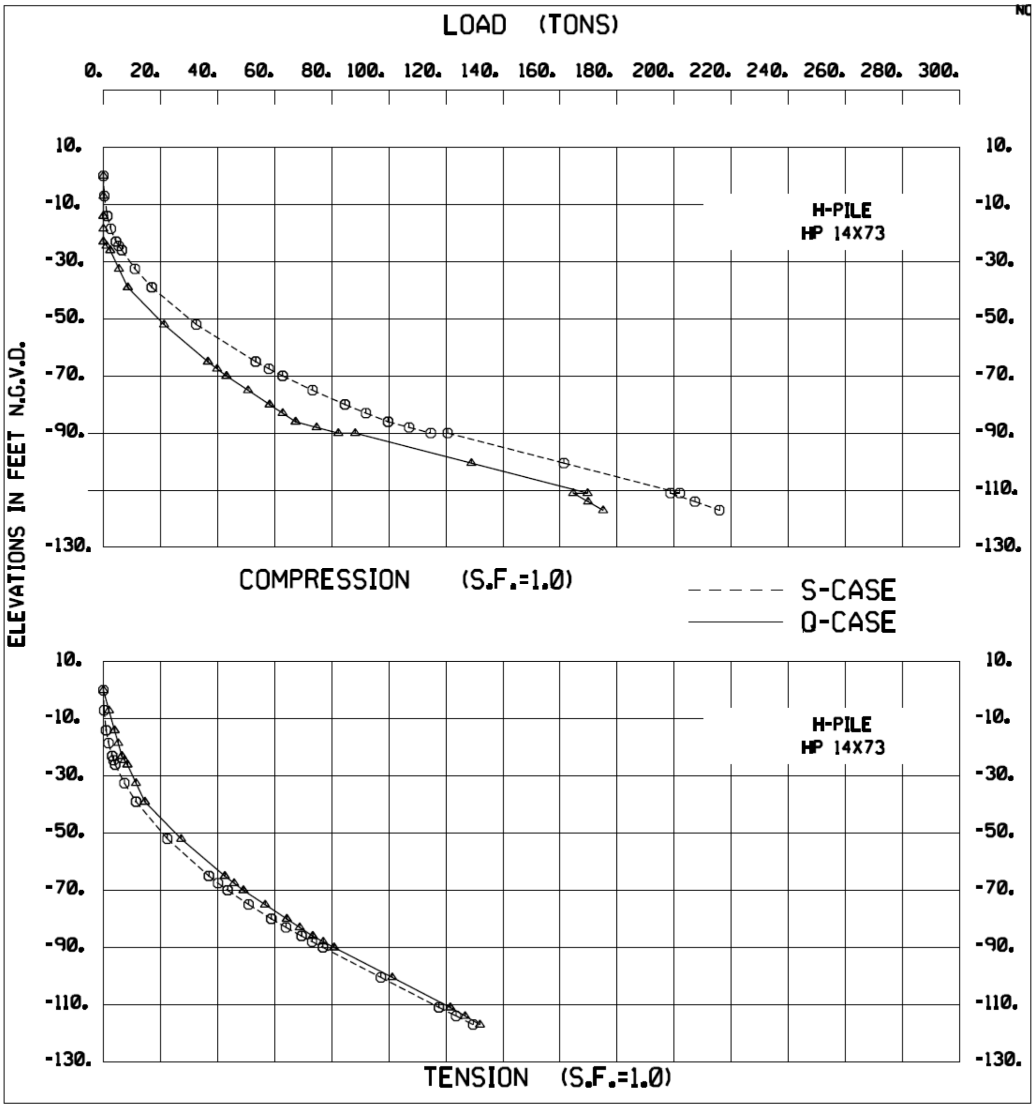


Figure 6. Ultimate Axial Capacity with Depth, Calculated

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.

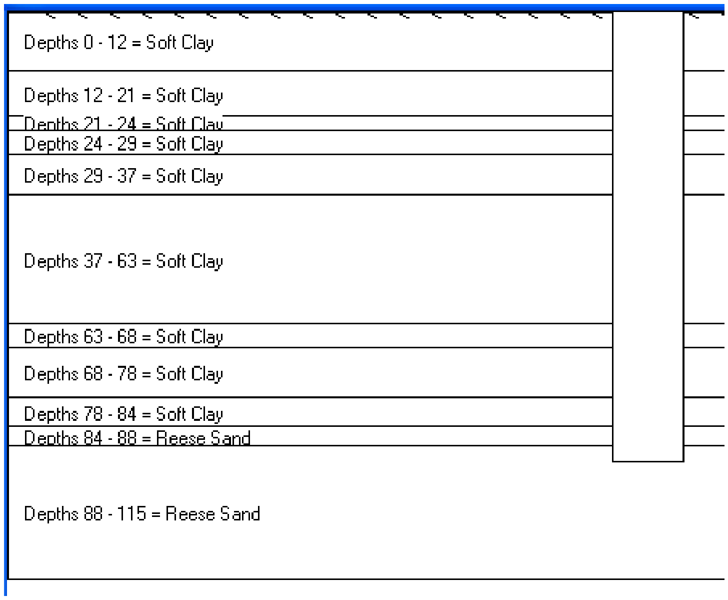


Figure 7. Soil Profiles - Stripped to critical surface of minus 23 for TZPILE and LPILE analysis

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.

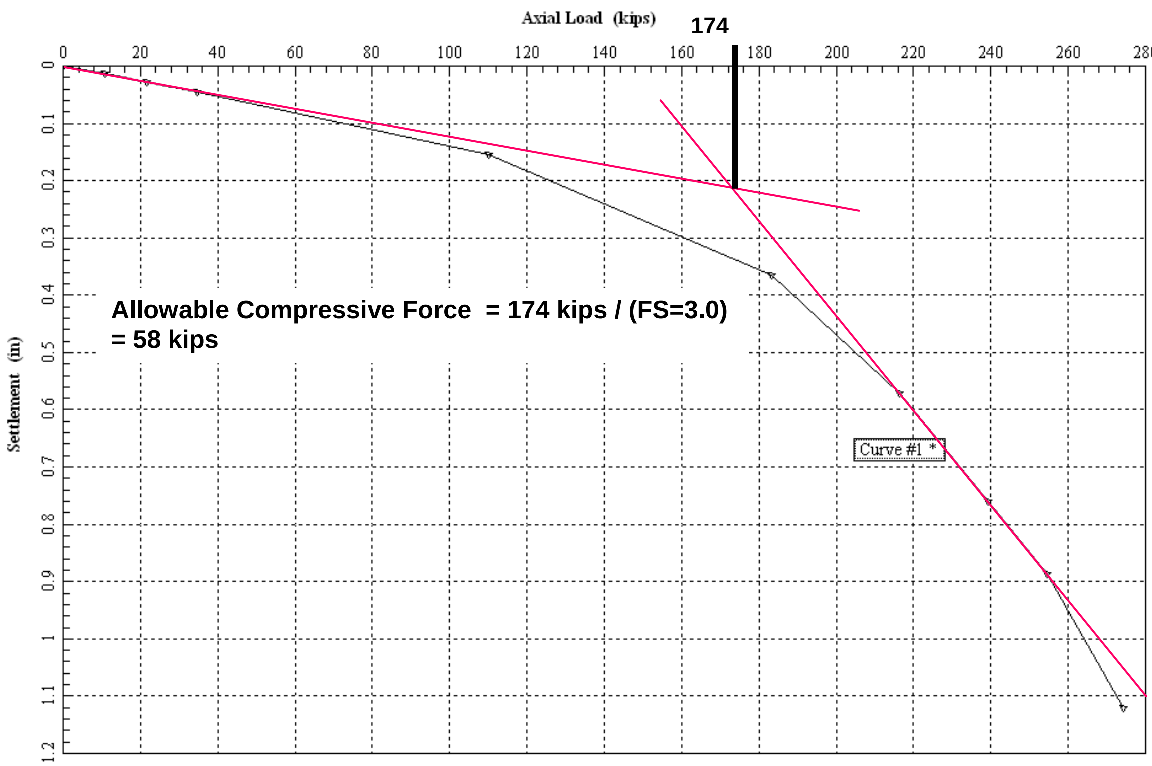


Figure 8. TZPILE Axial Pile Analysis Compression Settlement vs Axial Load Plot for determination of allowable compressive loads in piles by load simulation method.

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.

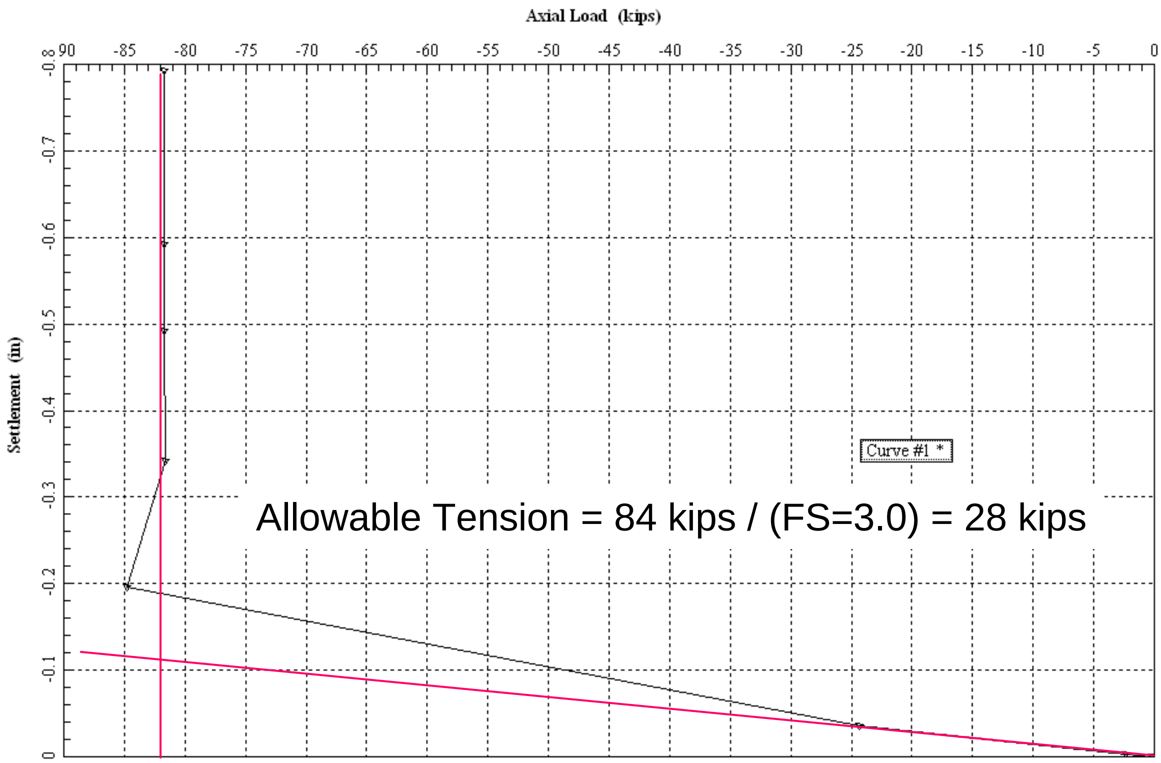


Figure 9. TZPILE Axial Pile Analysis TENSION Settlement vs Load Plot for allowable tensile loads in Piles

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.

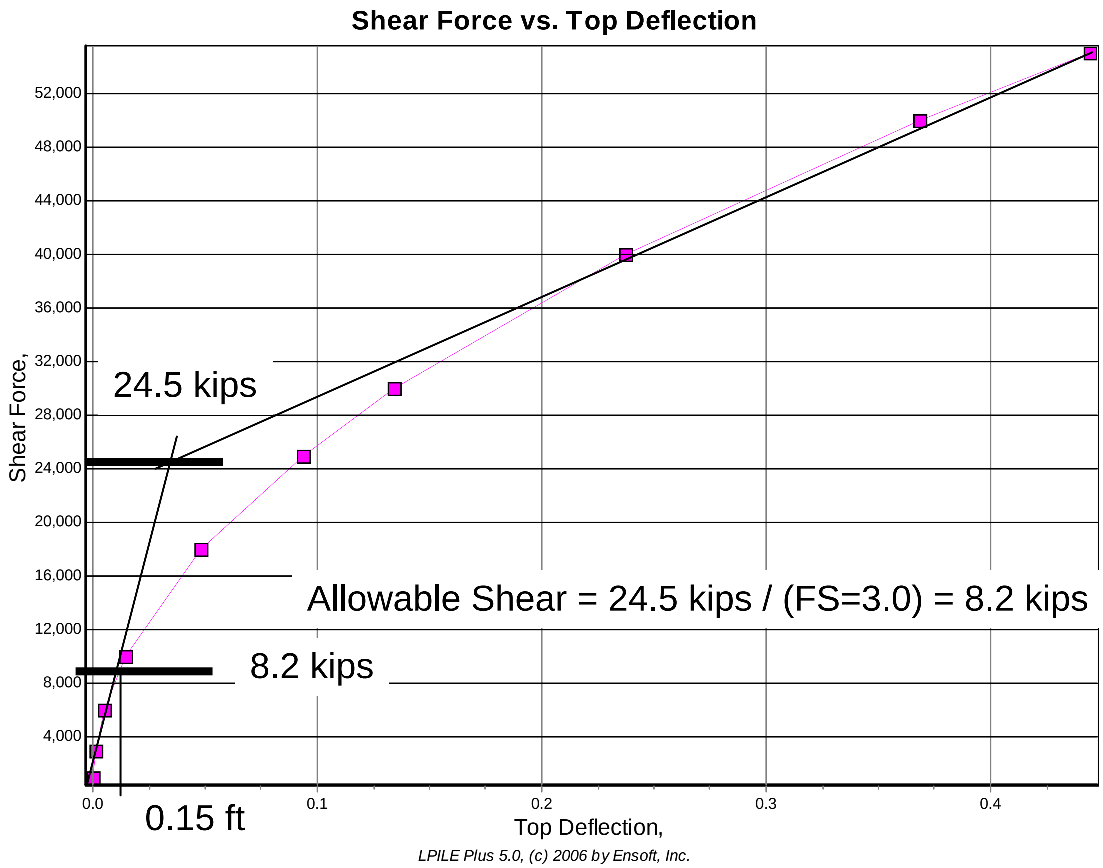


Figure 10. LPILE analysis of Pile head deflection vs shear force at critical surface to determine allowable shear force in piles.

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, 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 121 in = 10.08 feet

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

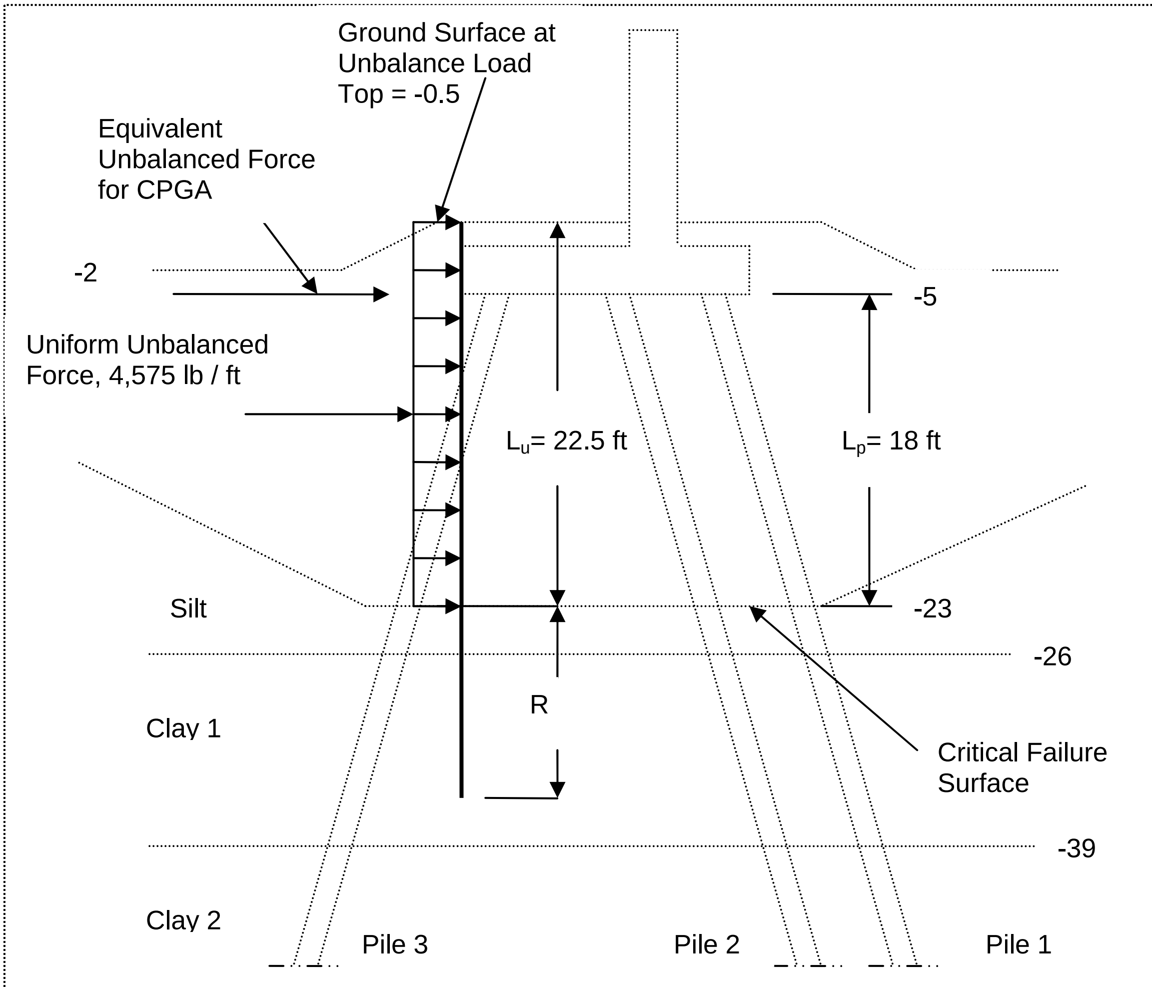


Figure 11. Equivalent Force Computation for Preliminary Design With CPGA

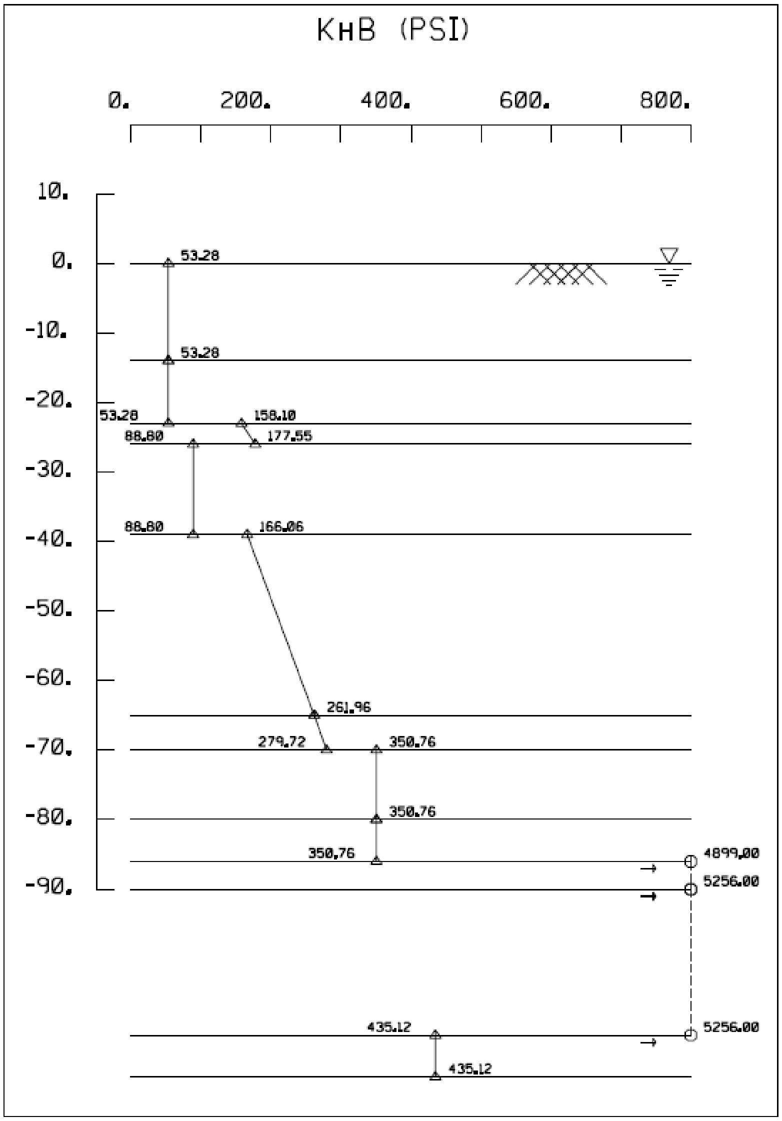


Figure 12. Soil Stiffness with Depth

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:

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

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

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

Therefore, capacity against flow through is OK

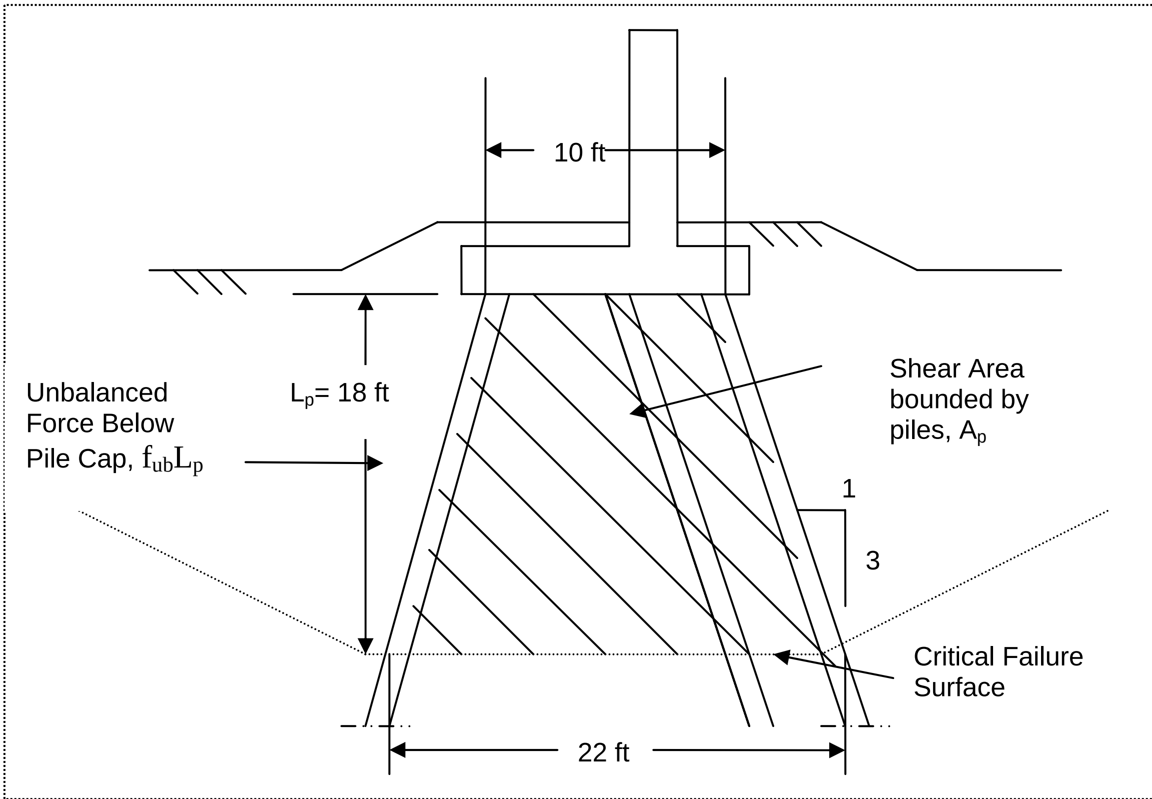


Figure 13. Shear Area for Flow Through Check

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.

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

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.

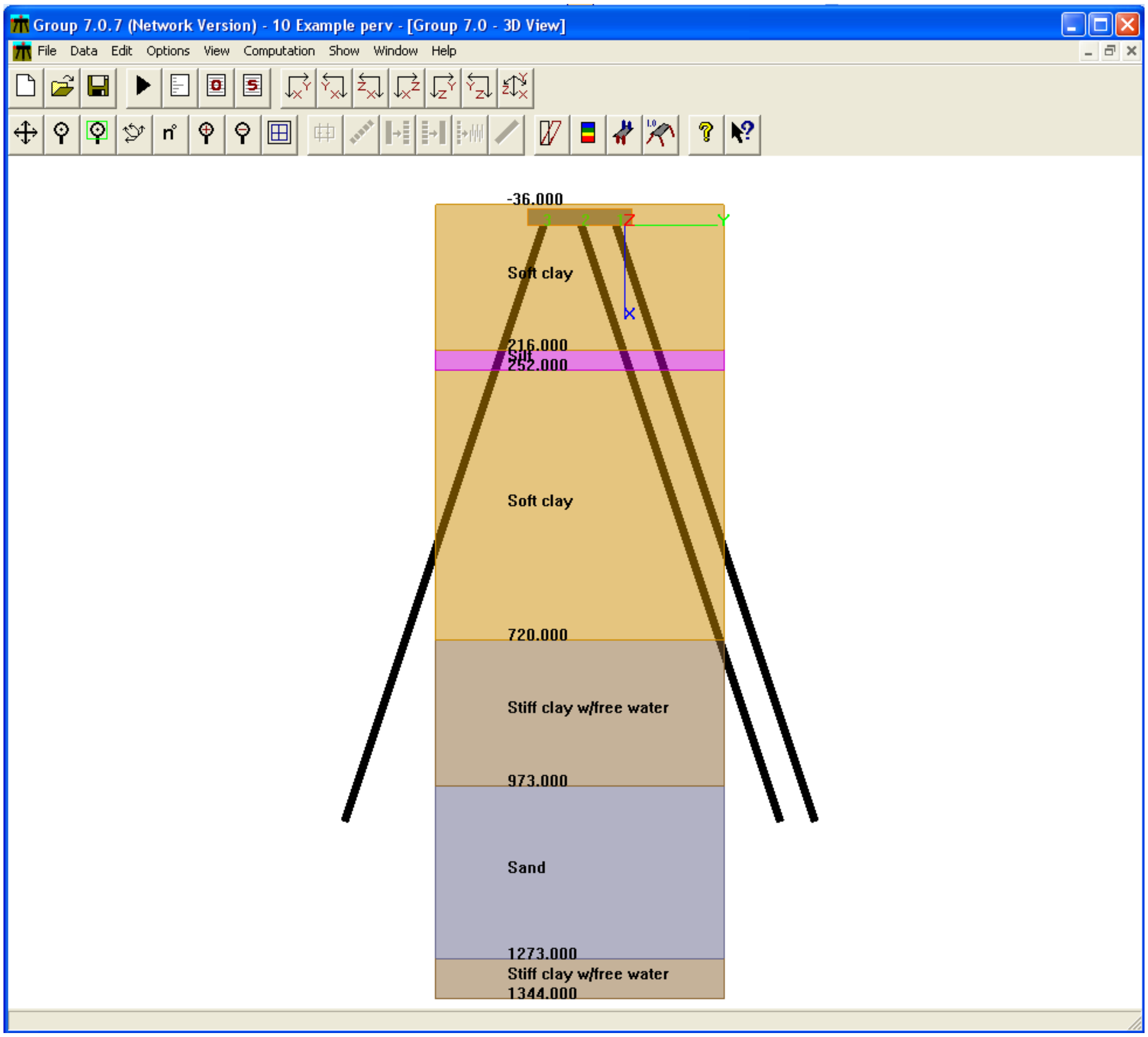


Figure 14. Group 7 Model with Soil Stratigraphy.

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.

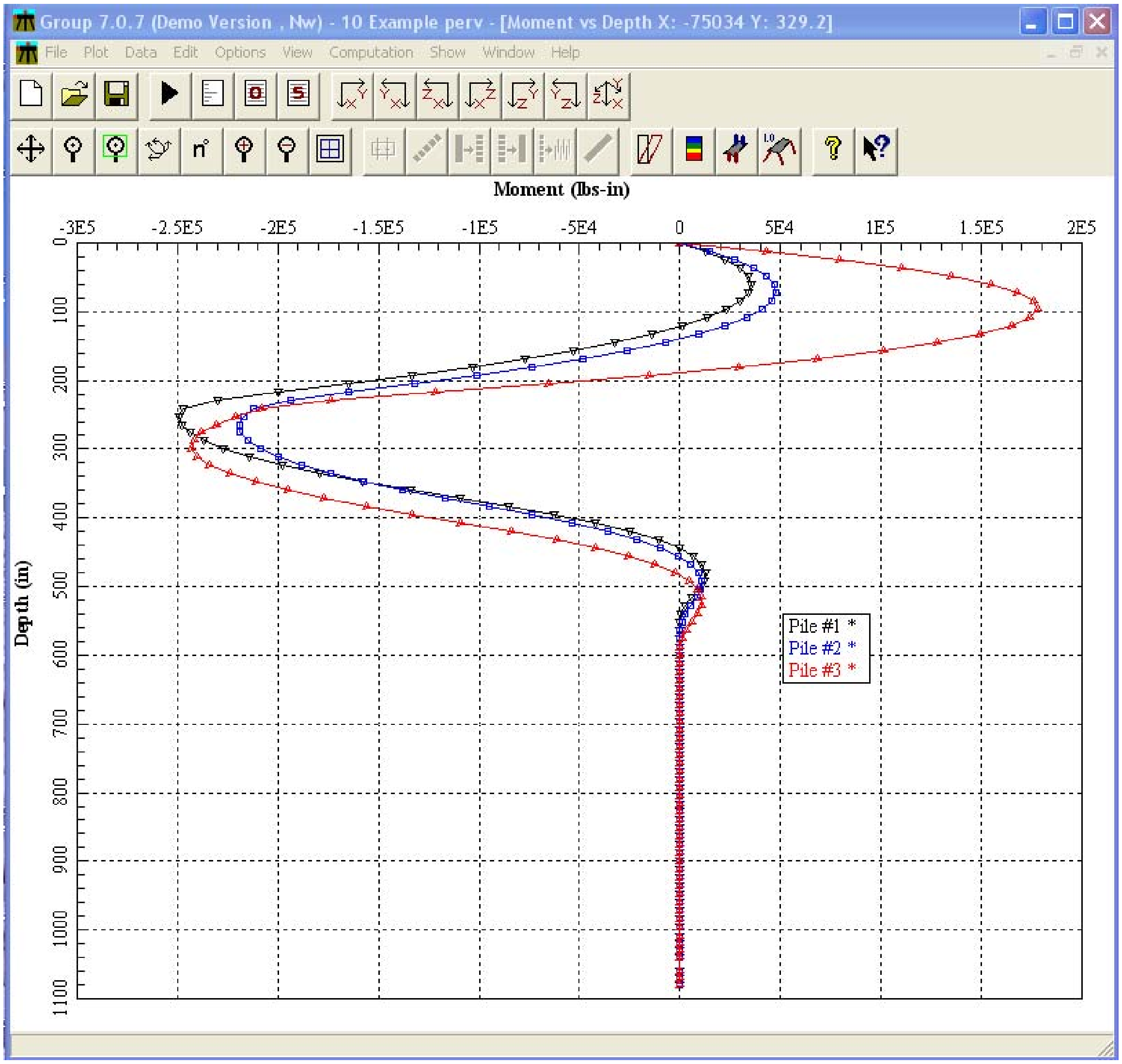


Figure 15. Moment vs depth.

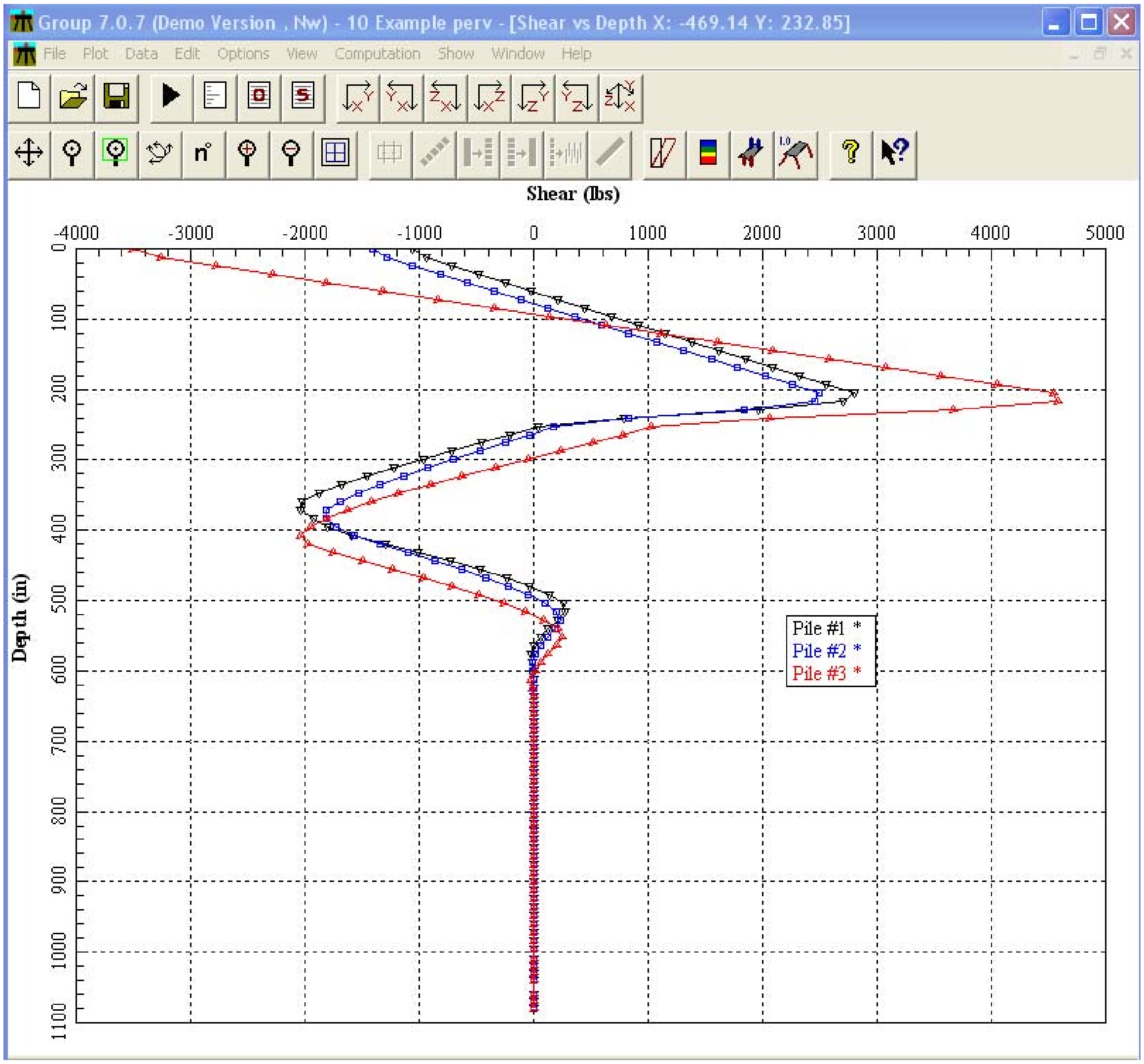


Figure 16. Shear vs depth

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.

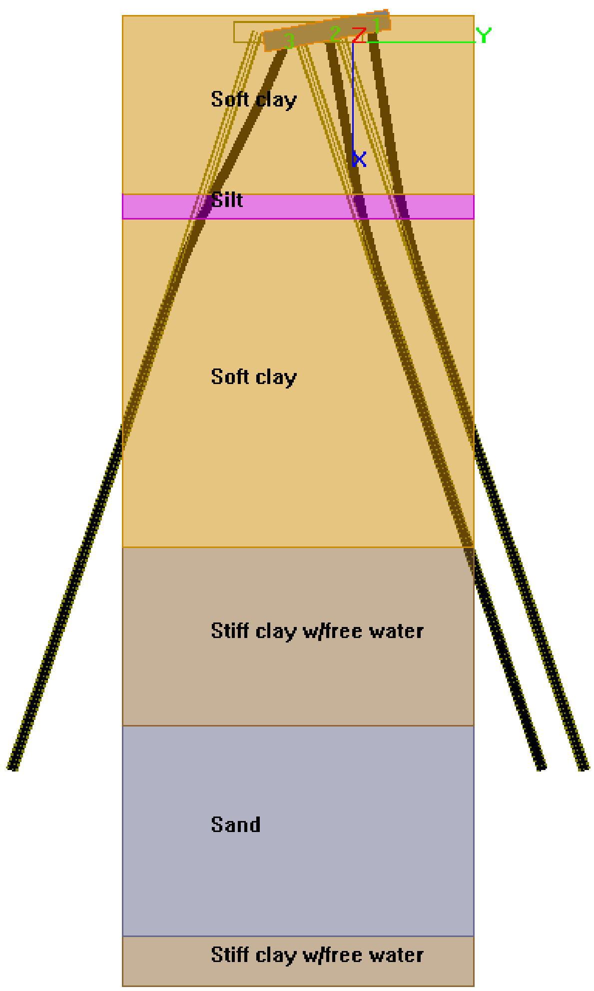


Figure 17 Deformed shape of pile cap

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

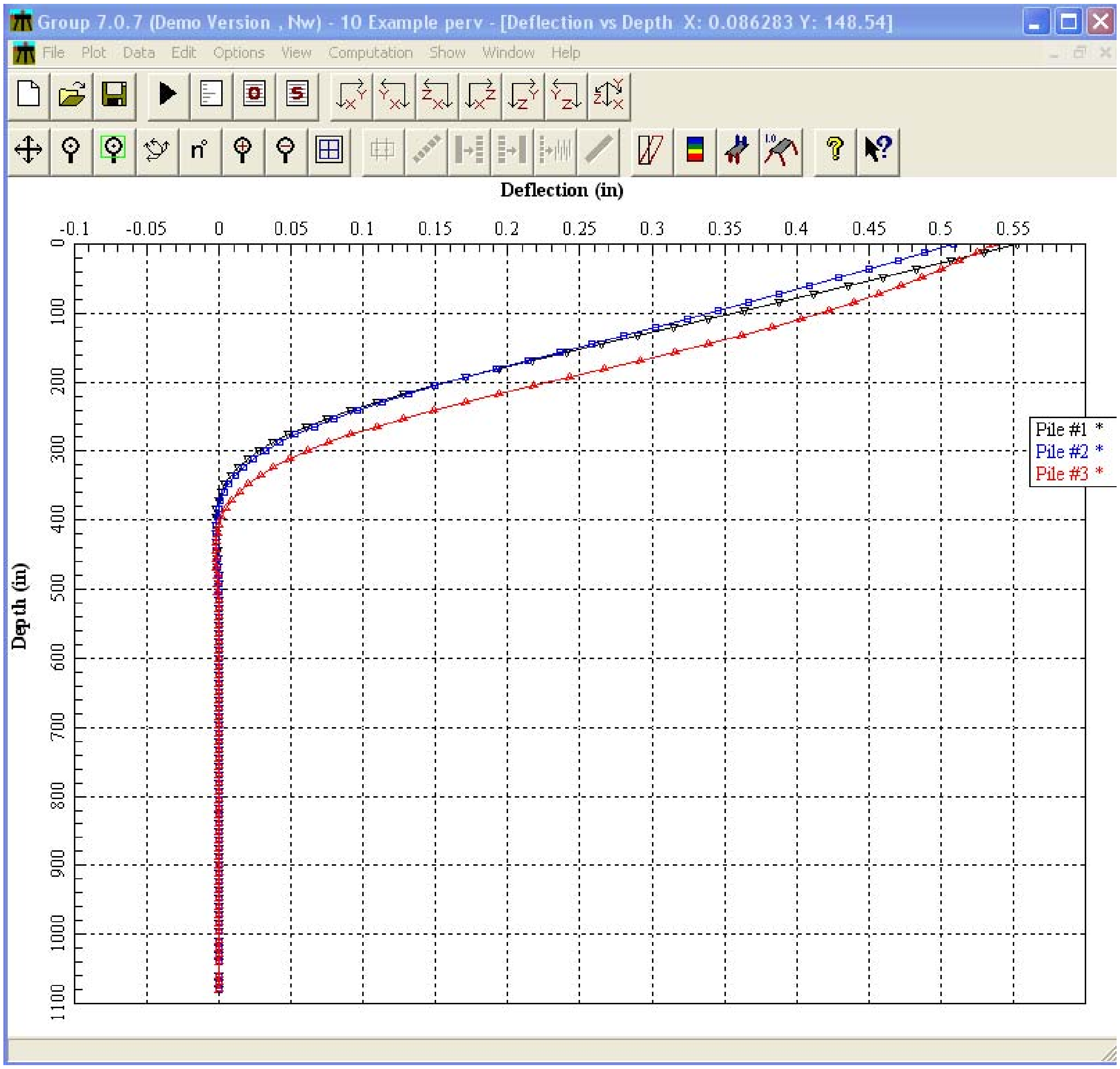


Figure 18 Deflection vs Depth

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.

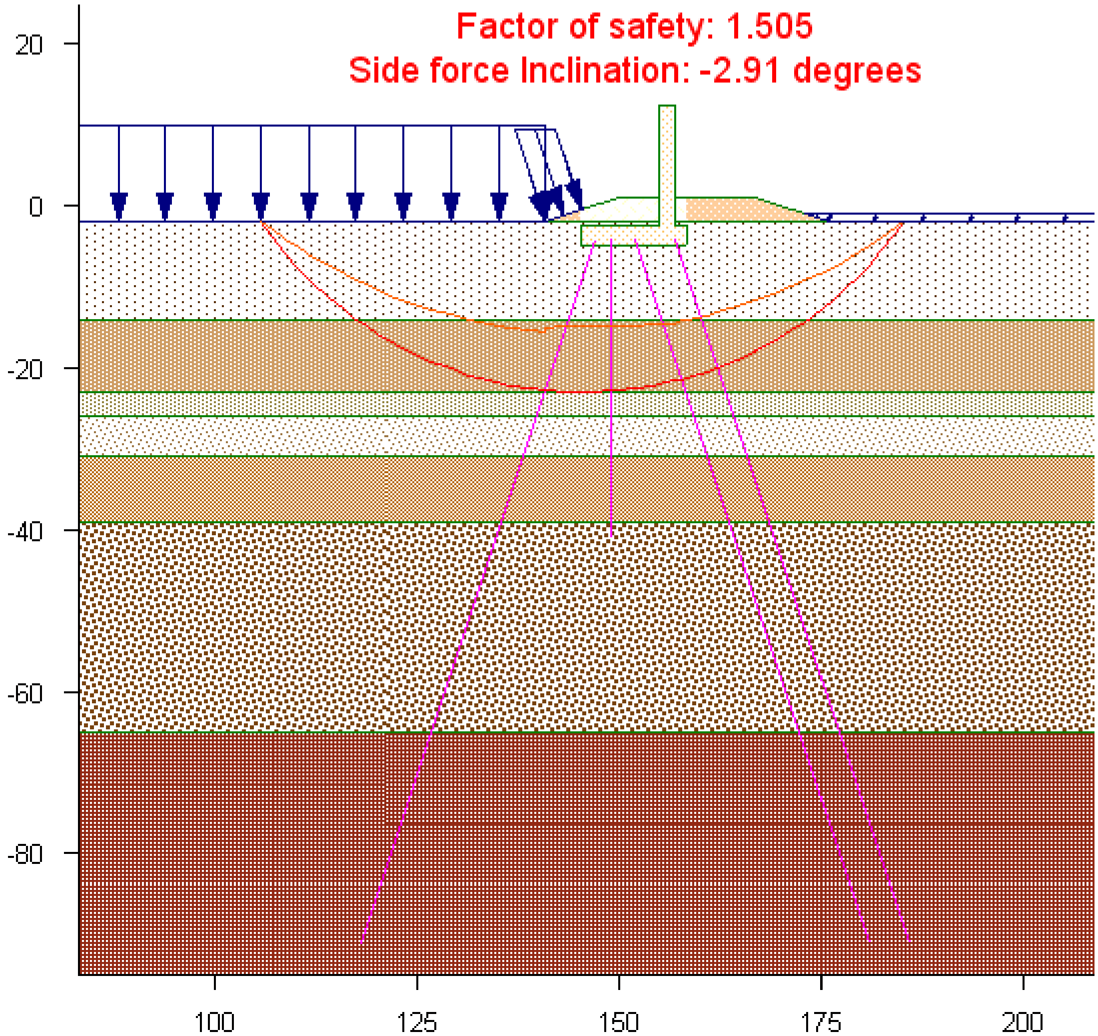


Figure 19. Factor of safety computed using pile forces from Group 7 analysis And critical circle from fixed grid analysis

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.

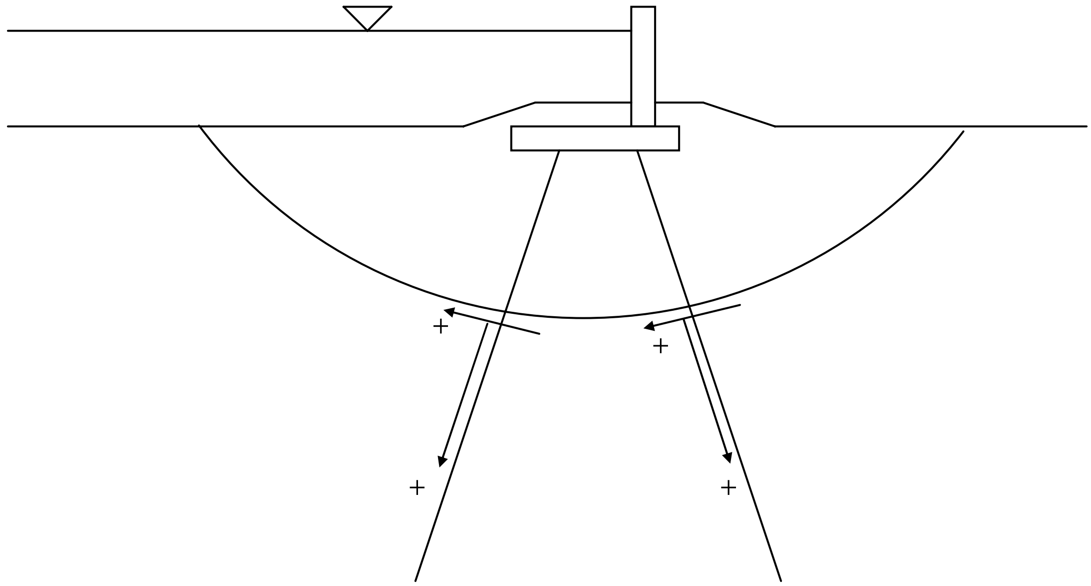


Figure 20. Positive directions for longitudinal and transverse reinforcement loads in pile.

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

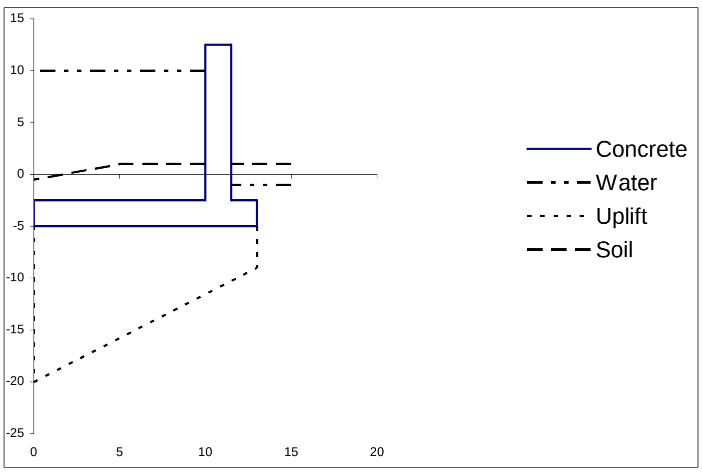
Vertical Forces

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

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	

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', 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

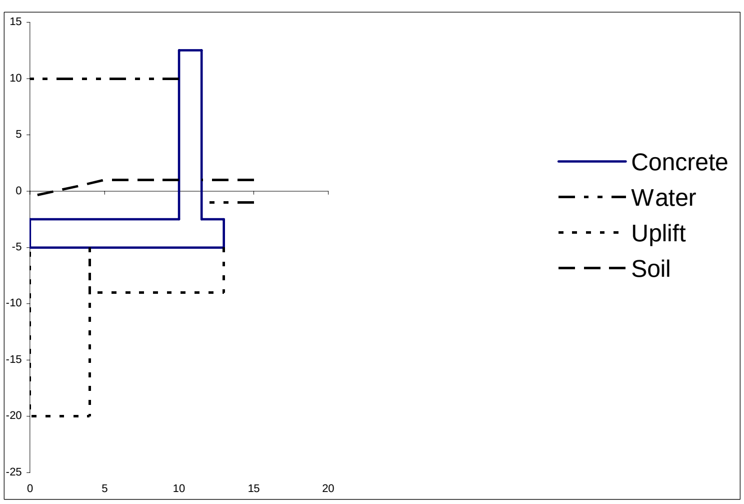
Vertical Forces

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           DZ           R
             IN           IN          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             Z
                                      FT            FT
                                    7.74        -11.20

 LOAD     MOMENT IN
 CASE     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       F2       F3         M1         M2         M3   ALF   CBF
           K        K        K       IN-K       IN-K       IN-K

    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       F2       F3         M1         M2         M3   ALF   CBF
           K        K        K       IN-K       IN-K       IN-K

    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 IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
0.00	0.553E+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
```

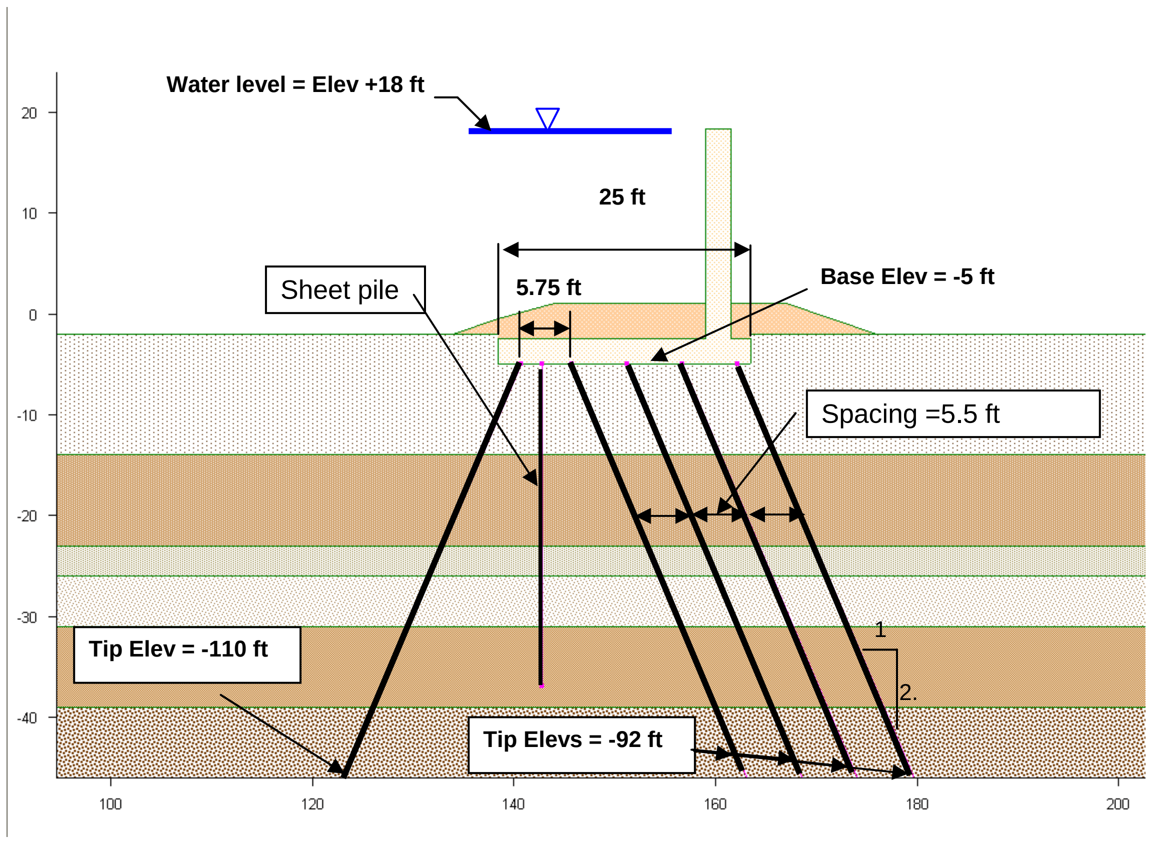
```
          375.00    -31.00

    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.

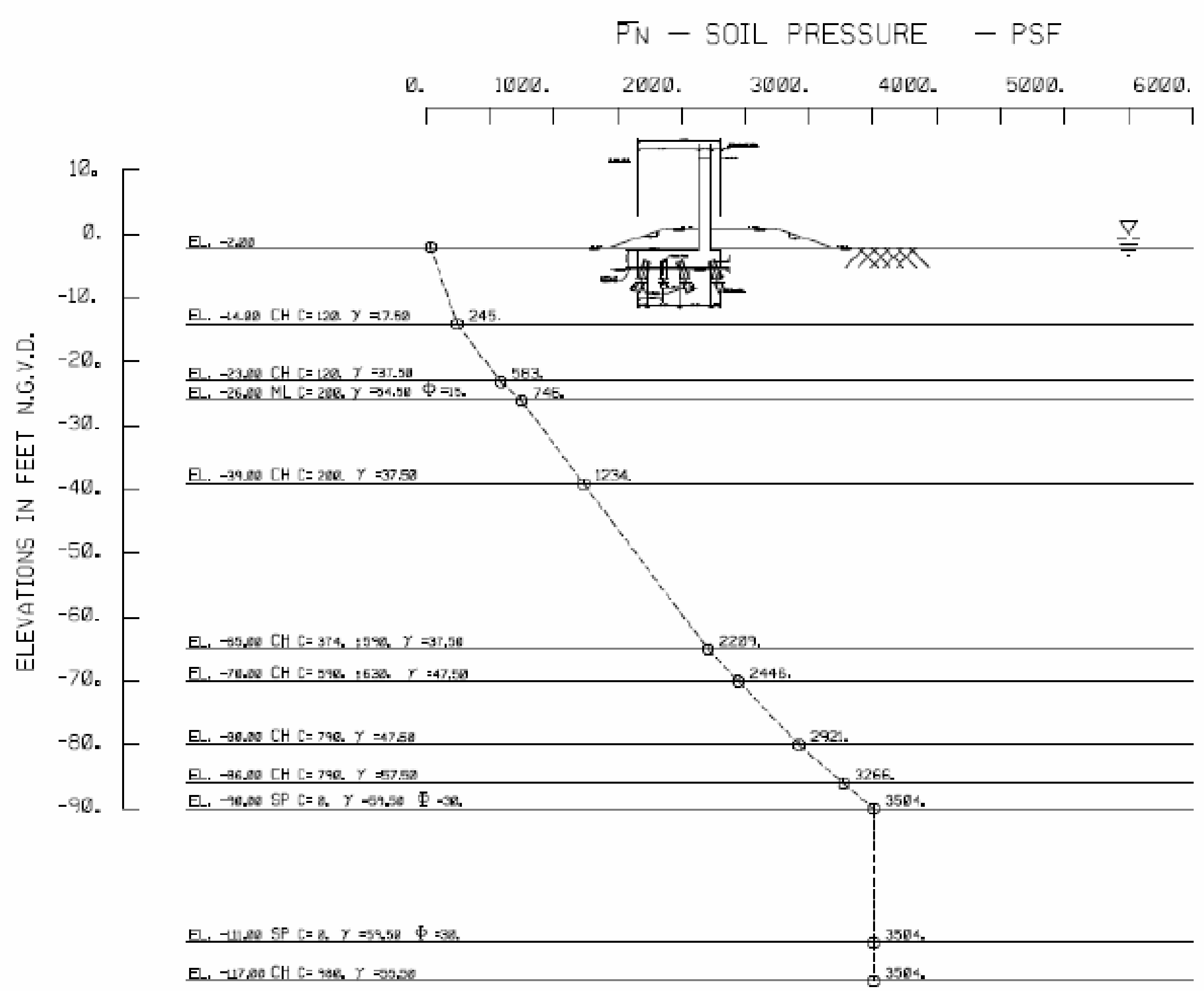


Figure 2. Soil Profile.

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.

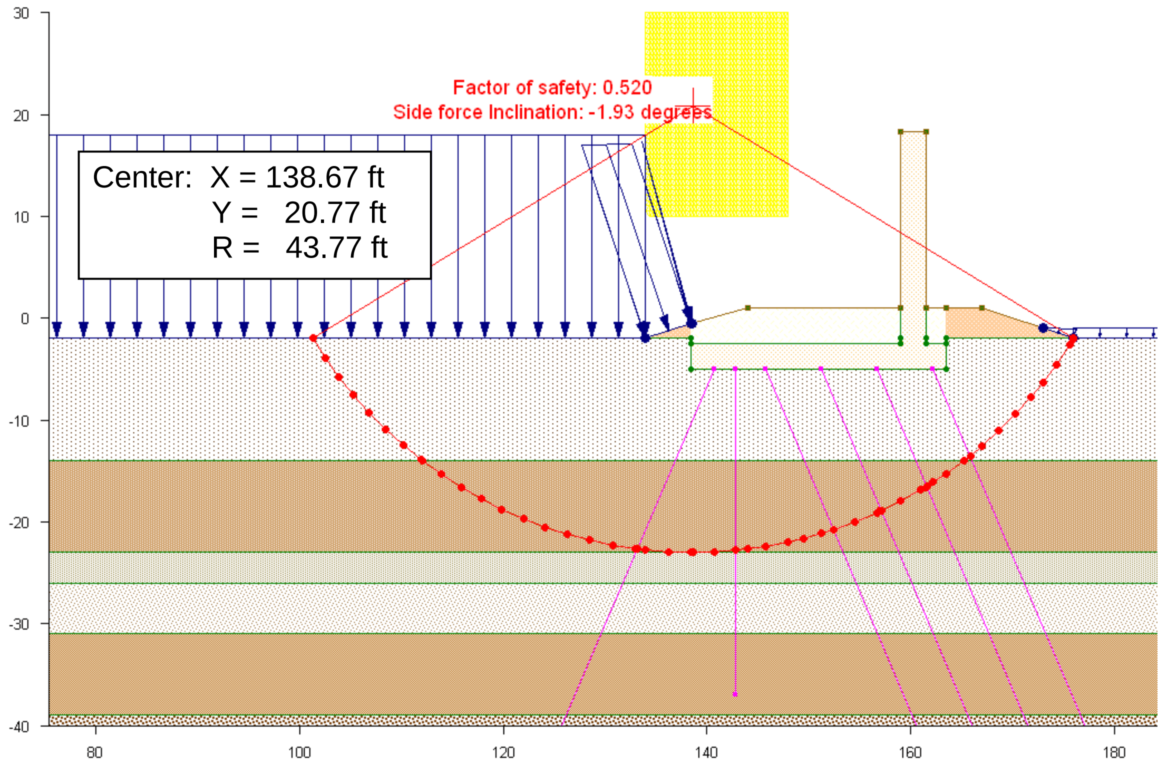


Figure 3. Spencer's analysis of the T-Wall without piles.

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.

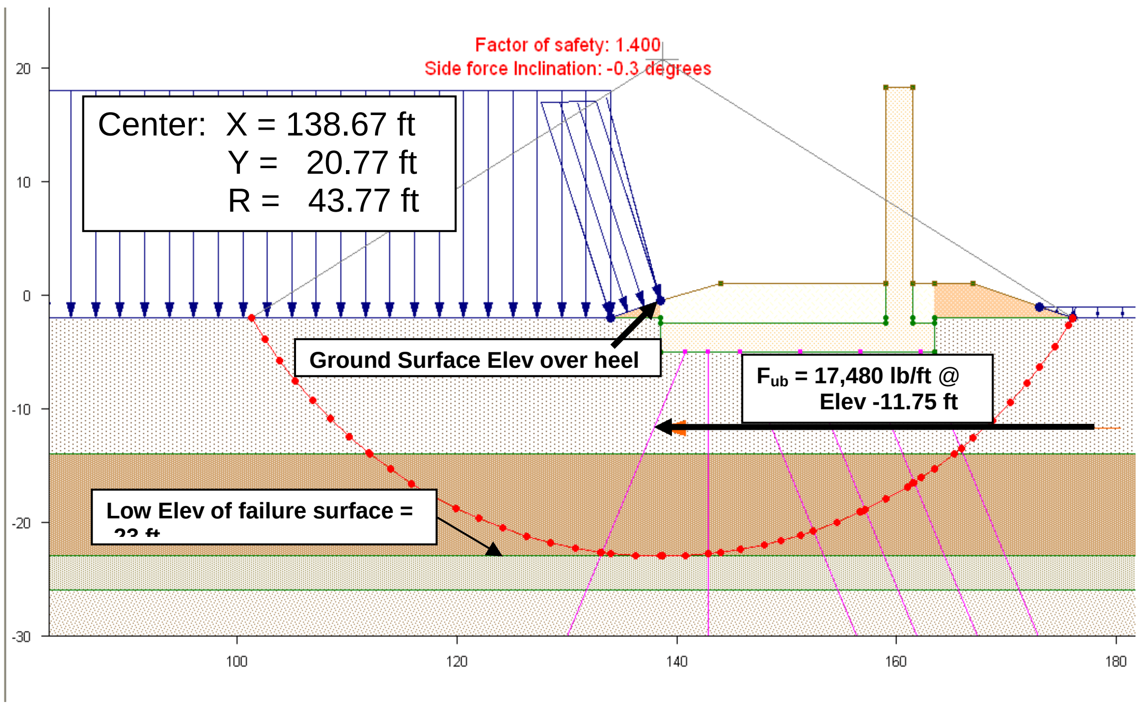


Figure 4. Spencer's analysis of the T-Wall with an unbalanced load to increase global stability.

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.

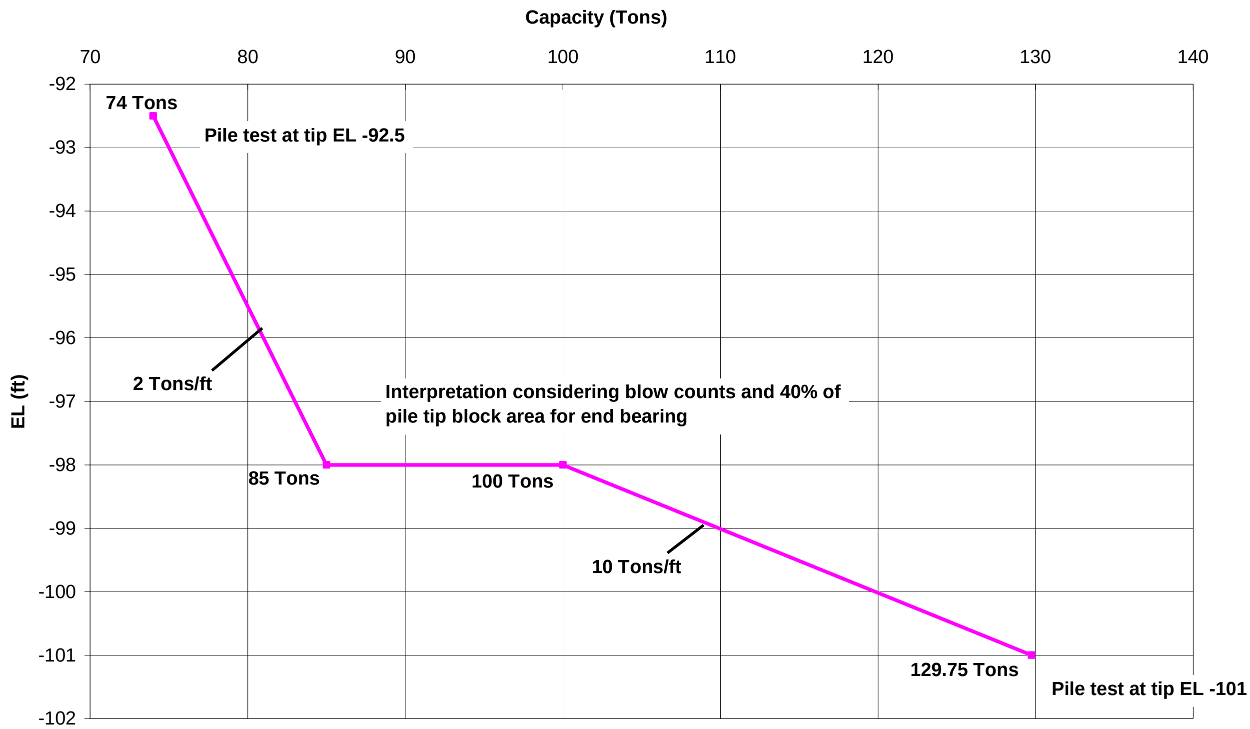


Figure 5. Pile Load Test Data

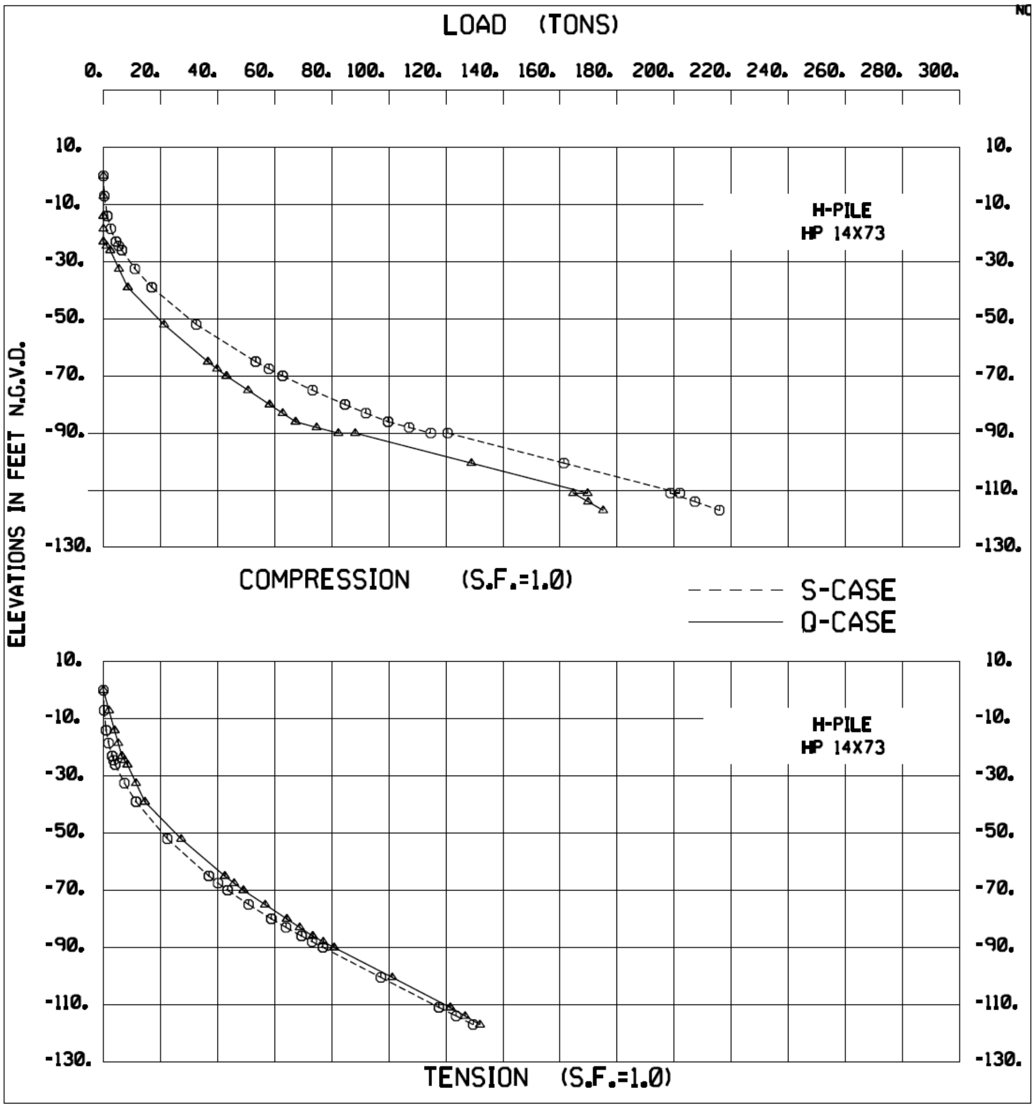


Figure 6. Ultimate Axial Capacity with Depth, Calculated

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

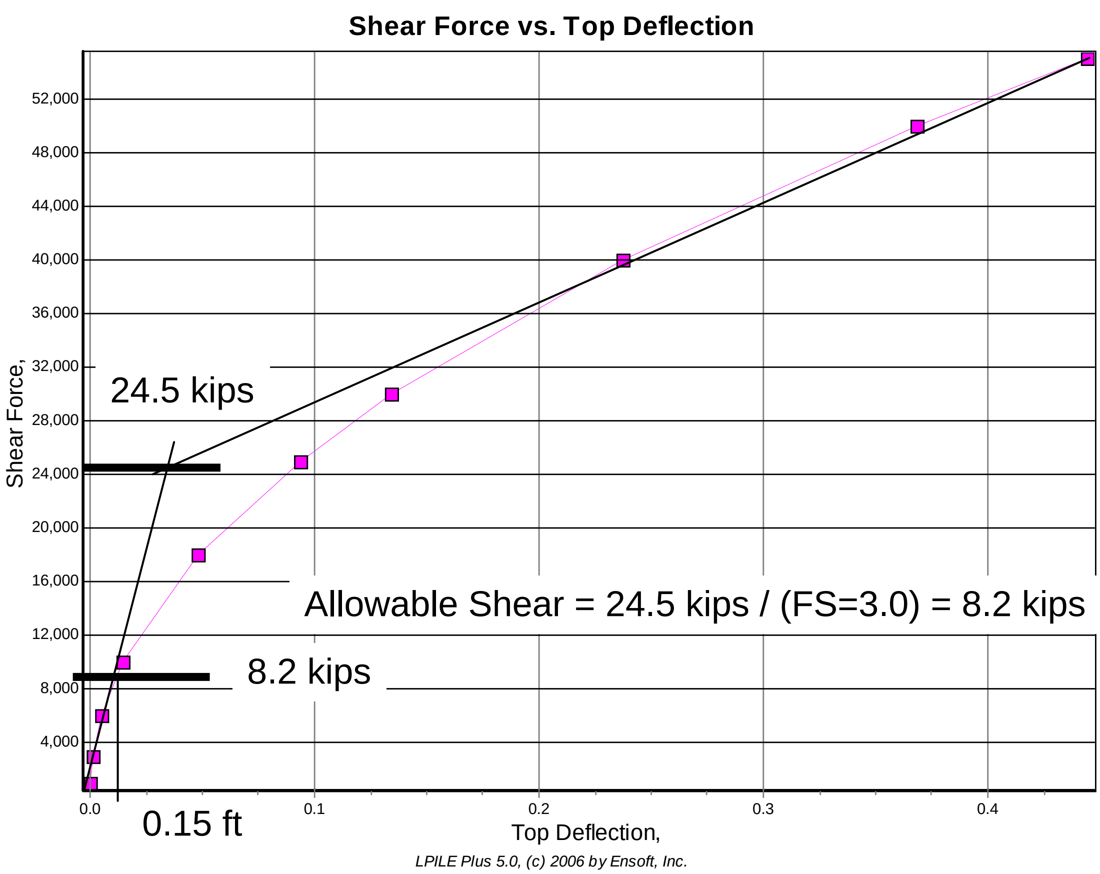


Figure 7. LPILE analysis of Pile head deflection vs shear force at critical surface to determine allowable shear force in piles.

Step 4 <u>Initial T-wall and Pile Design</u>

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

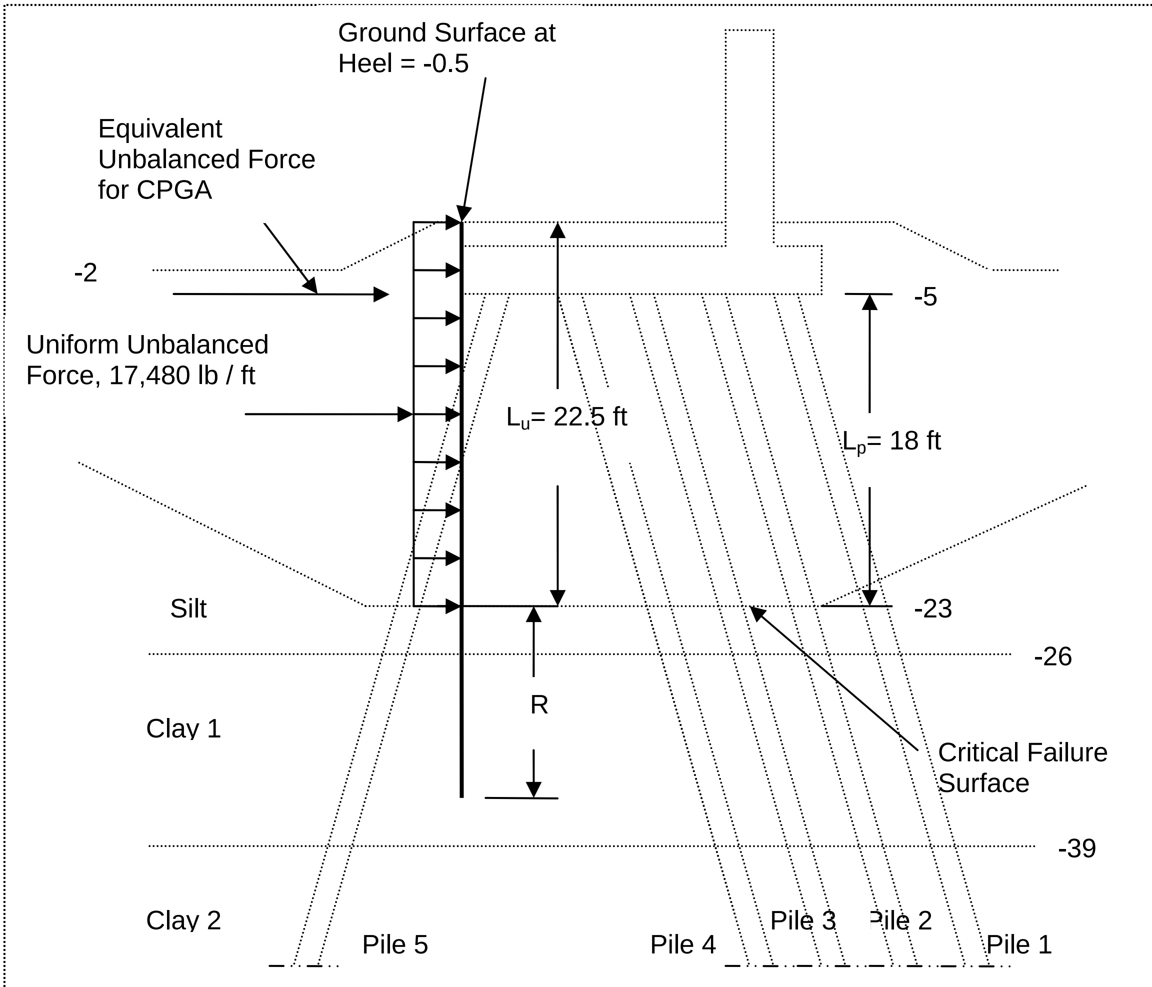


Figure 8. Equivalent Force Computation for Preliminary Design With CPGA

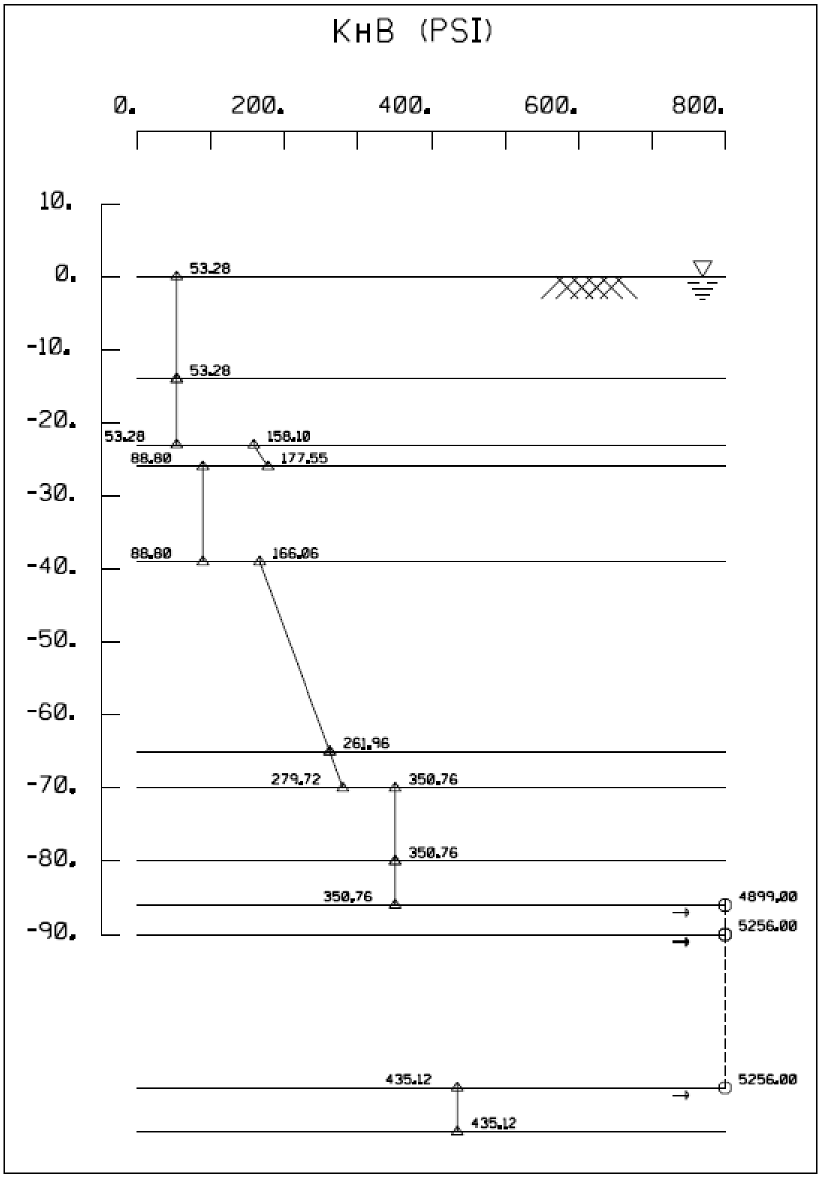


Figure 9. Soil Stiffness with Depth

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.

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

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 = 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

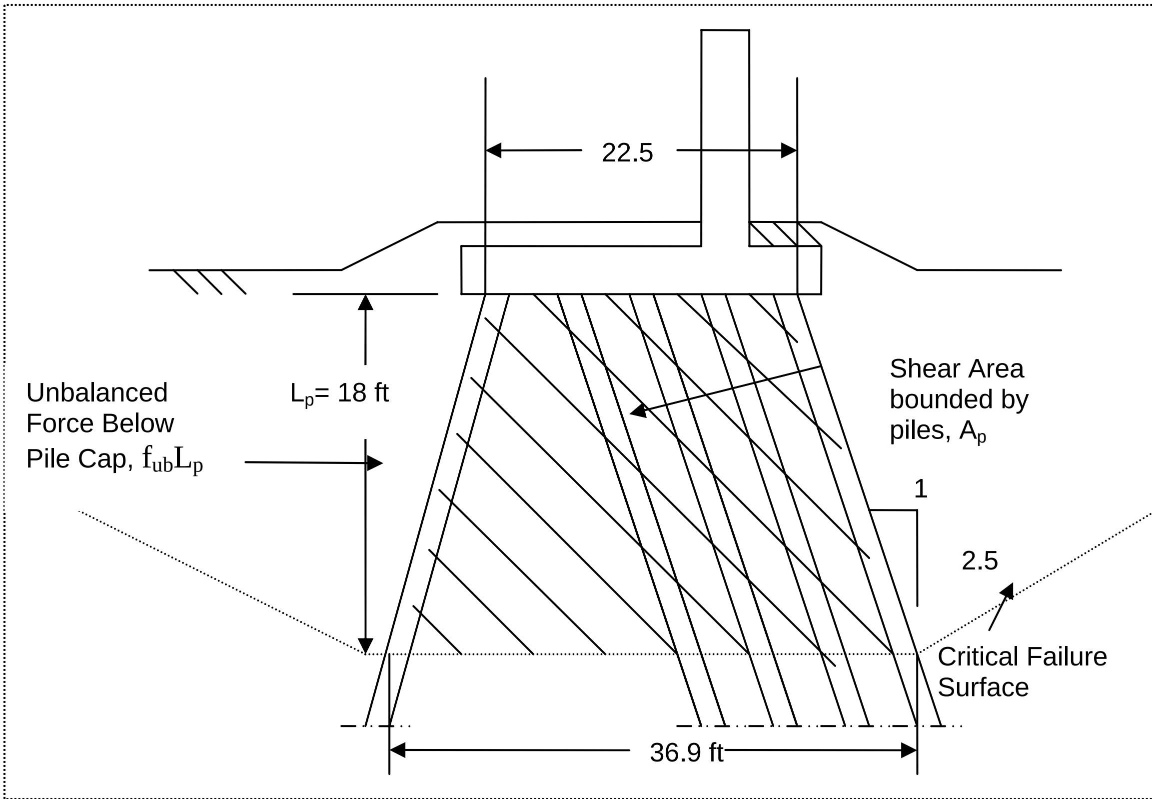


Figure 10. Shear Area for Flow-through Check

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.

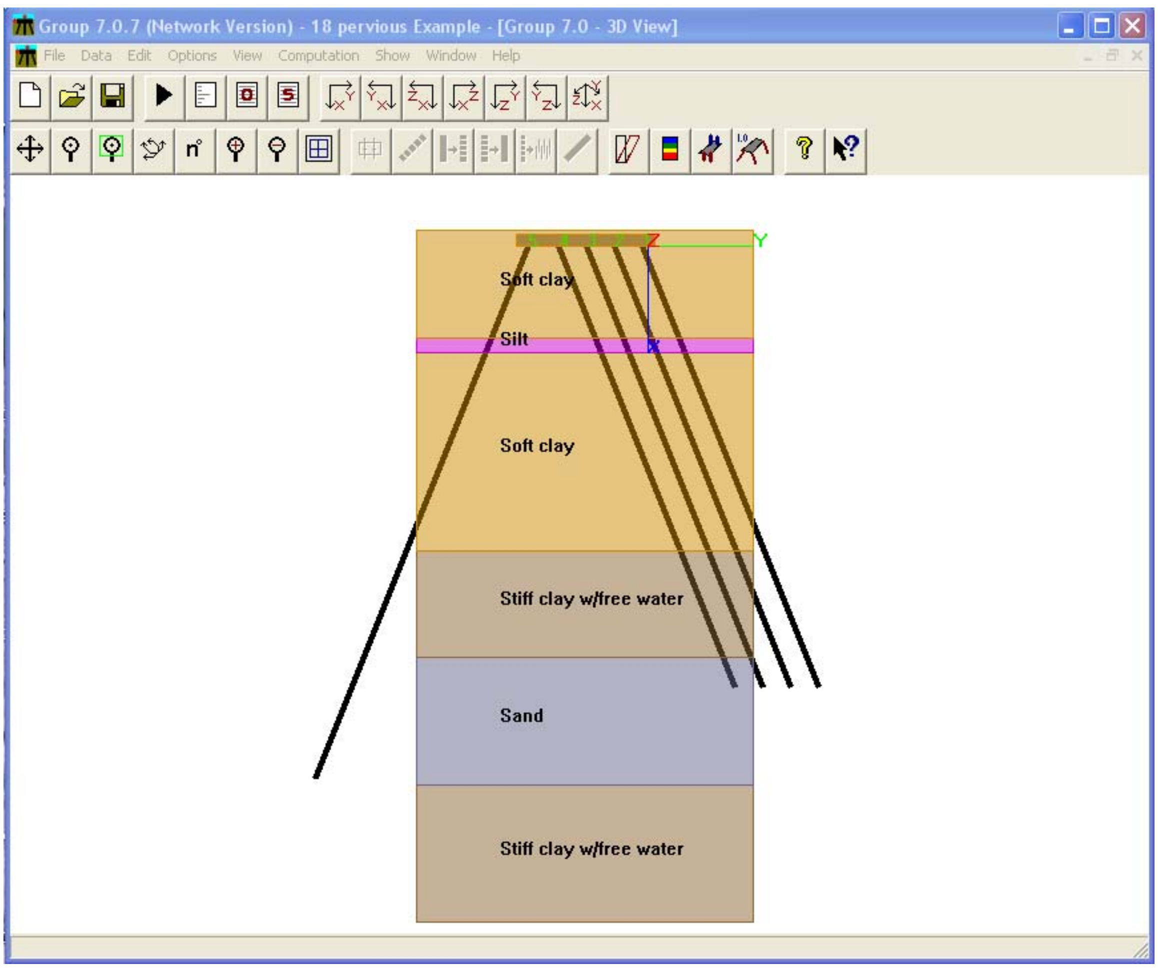


Figure 11. Group 7 Model

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

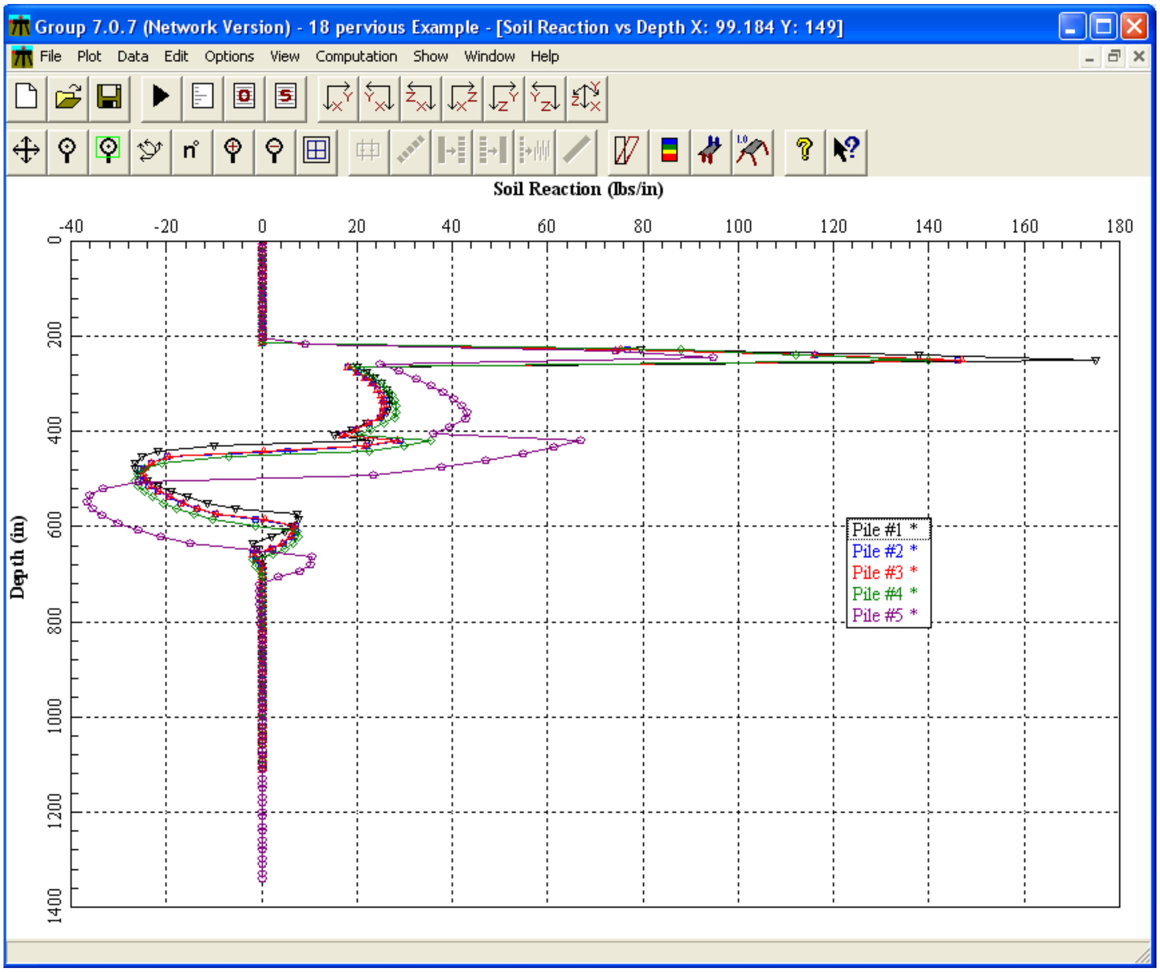


Figure 12 Soil Reaction on Piles with Depth

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.

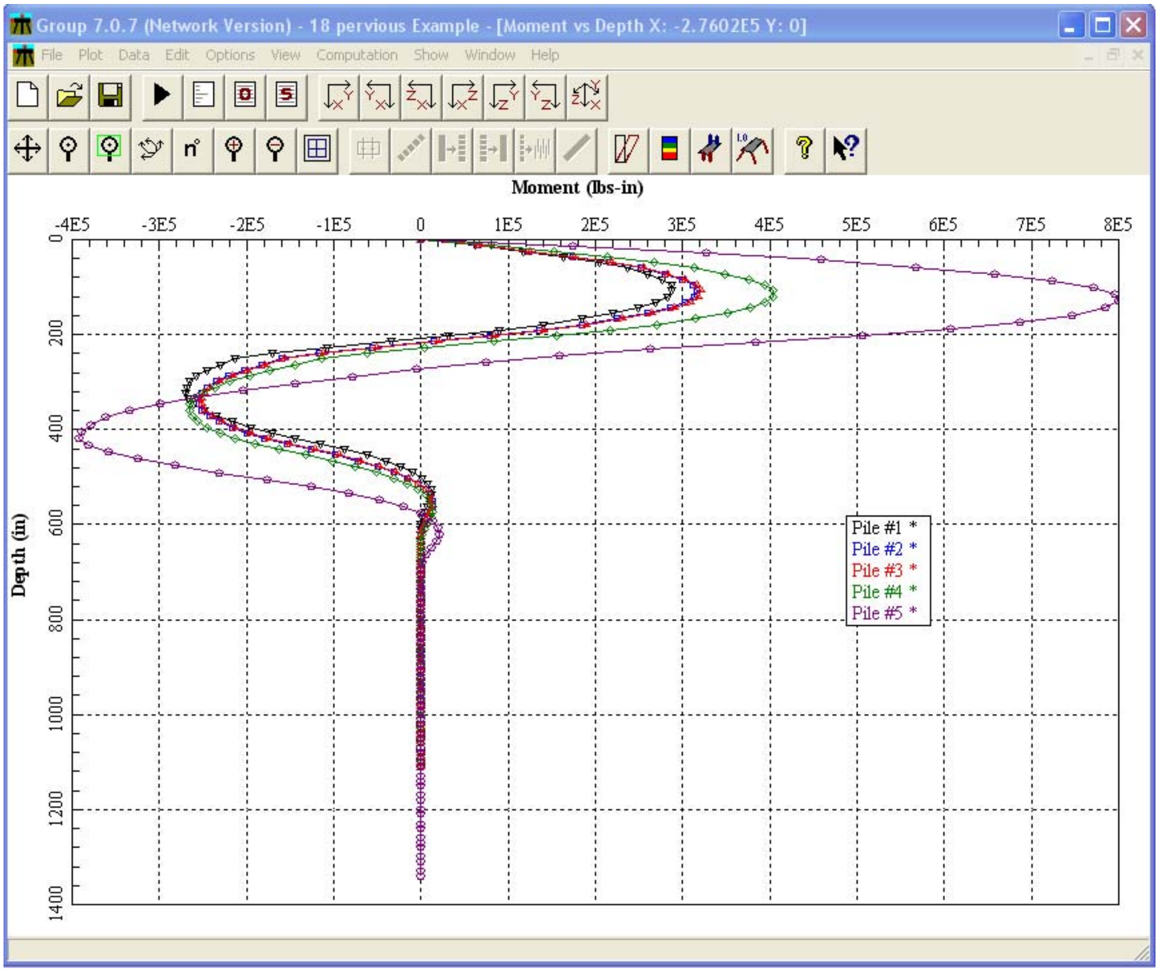


Figure 13 Moment in Piles With Depth

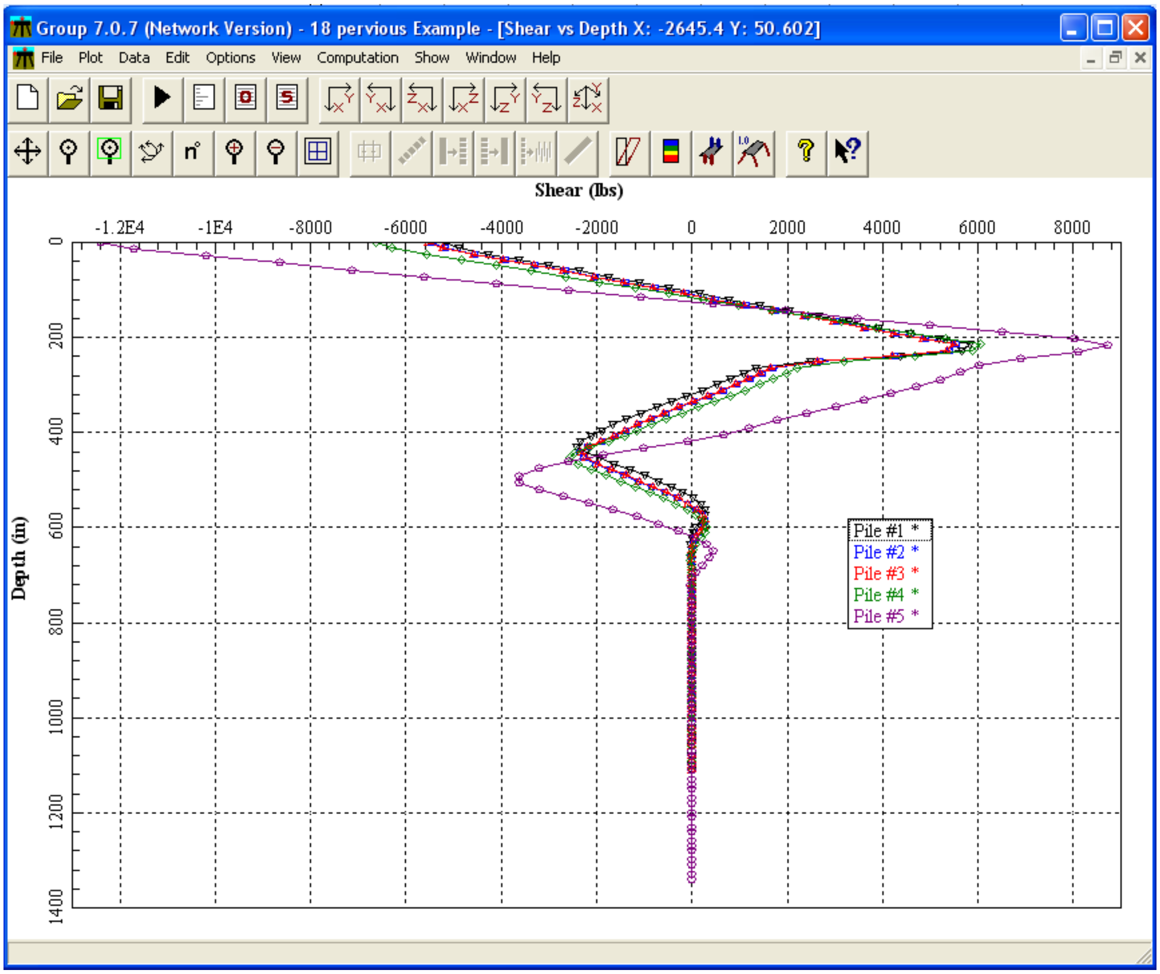


Figure 14. Shear diagrams for each of the four piles.

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.

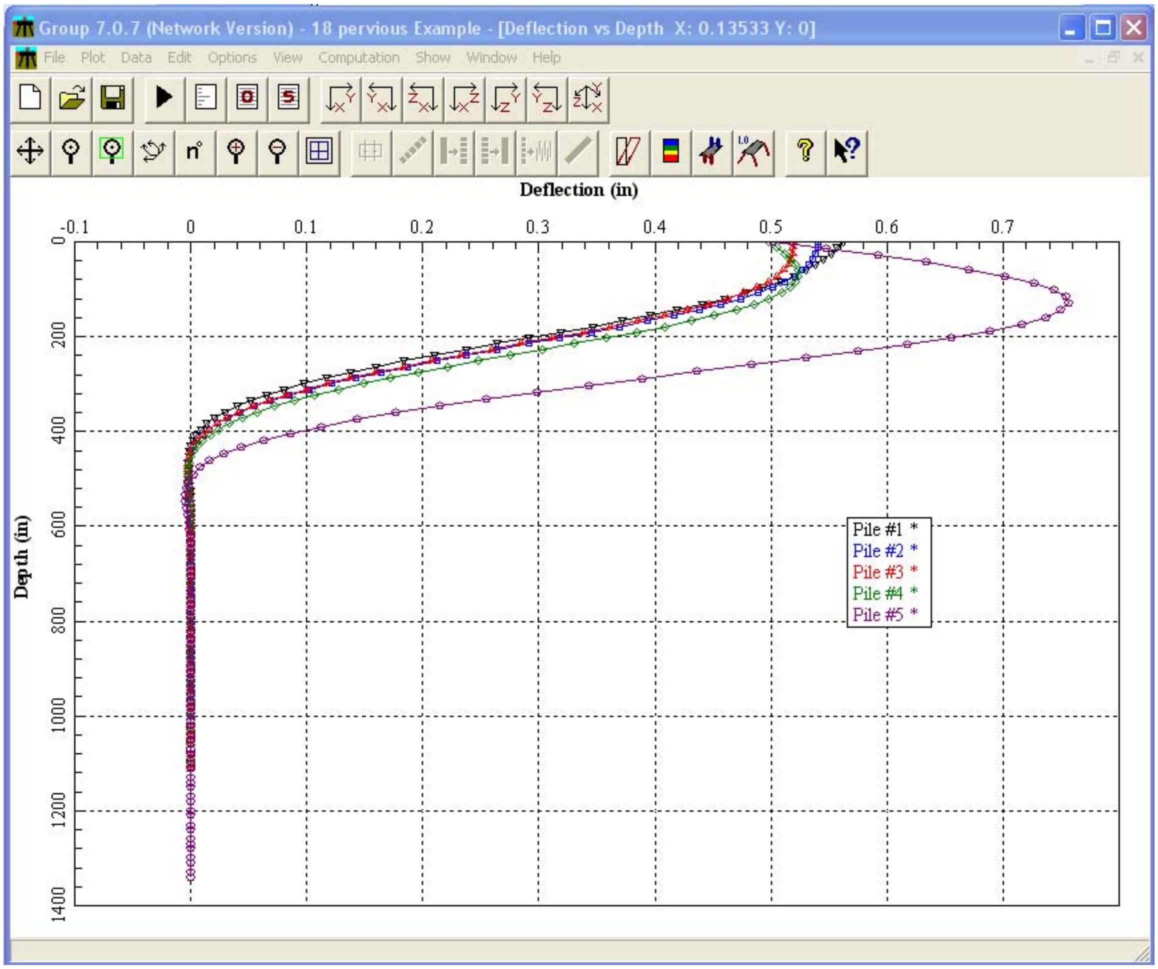


Figure 15. Deflection with Depth for the Pervious Foundation Condition.

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

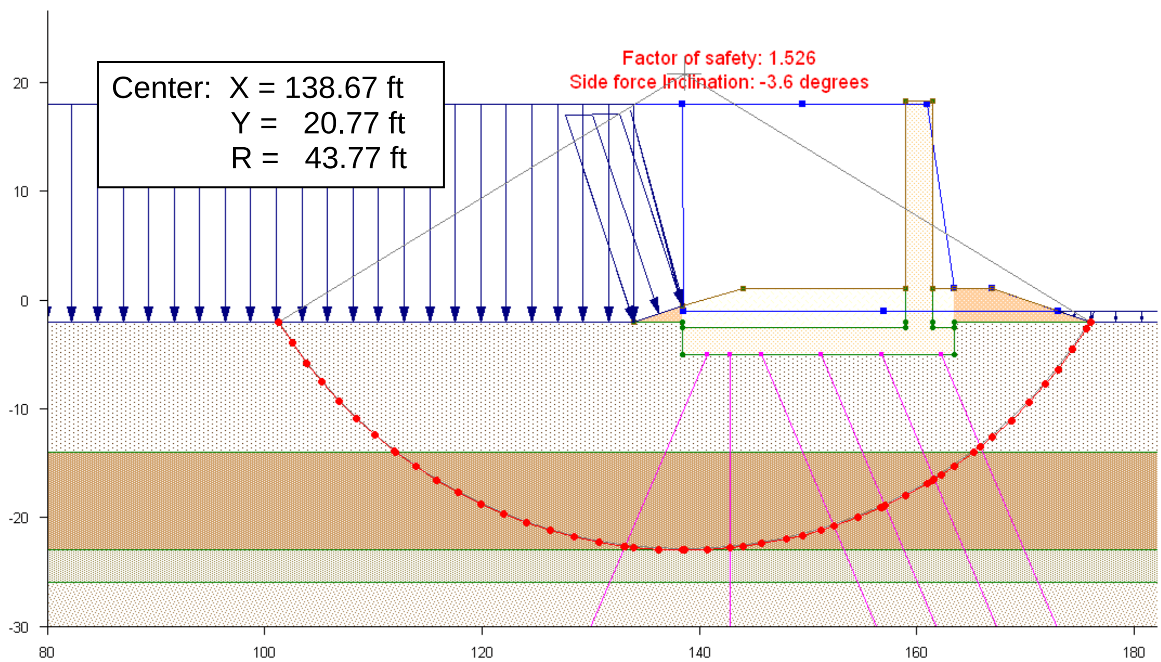


Figure 16. Factor of safety computed using pile forces from Group 7 analysis And critical circle from fixed grid analysis

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
```

Attachment 3 Structural Loads for CPGA and Group Analyses

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

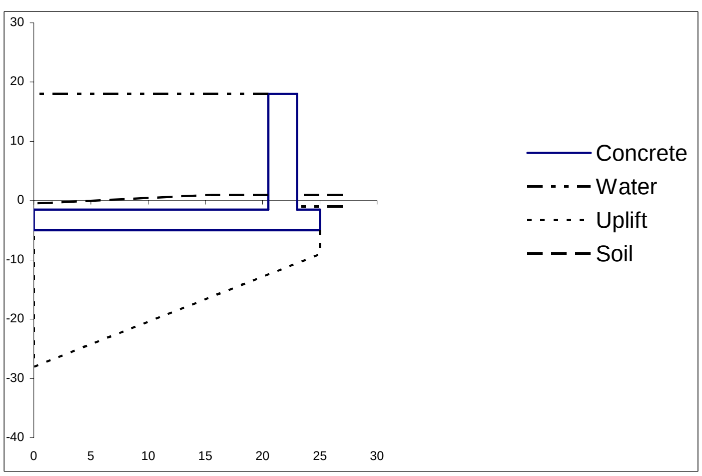
Input for CPGA pile analysis **Pervious Foundation Assumption**

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

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

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

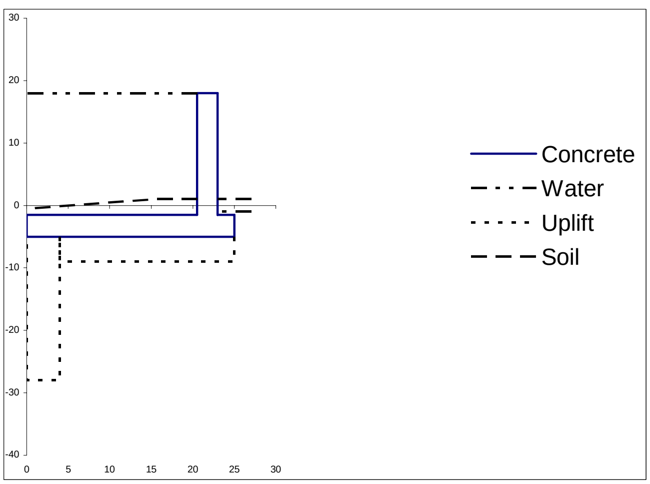
Vertical Forces

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

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 IN	DEFLECTION IN	MOMENT LBS-IN	SHEAR LBS	SOIL REACTION LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
0.00	0.562E+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

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     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
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    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  DEFLECTION    MOMENT     SHEAR      SOIL       TOTAL    FLEXURAL
                                             REACTION     STRESS   RIGIDITY
       IN      IN        LBS-IN      LBS      LBS/IN    LBS/IN**2  LBS-IN**2
     ***** ********** ********** ********** ********** ********** **********
      0.00  0.519E+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
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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
```

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.

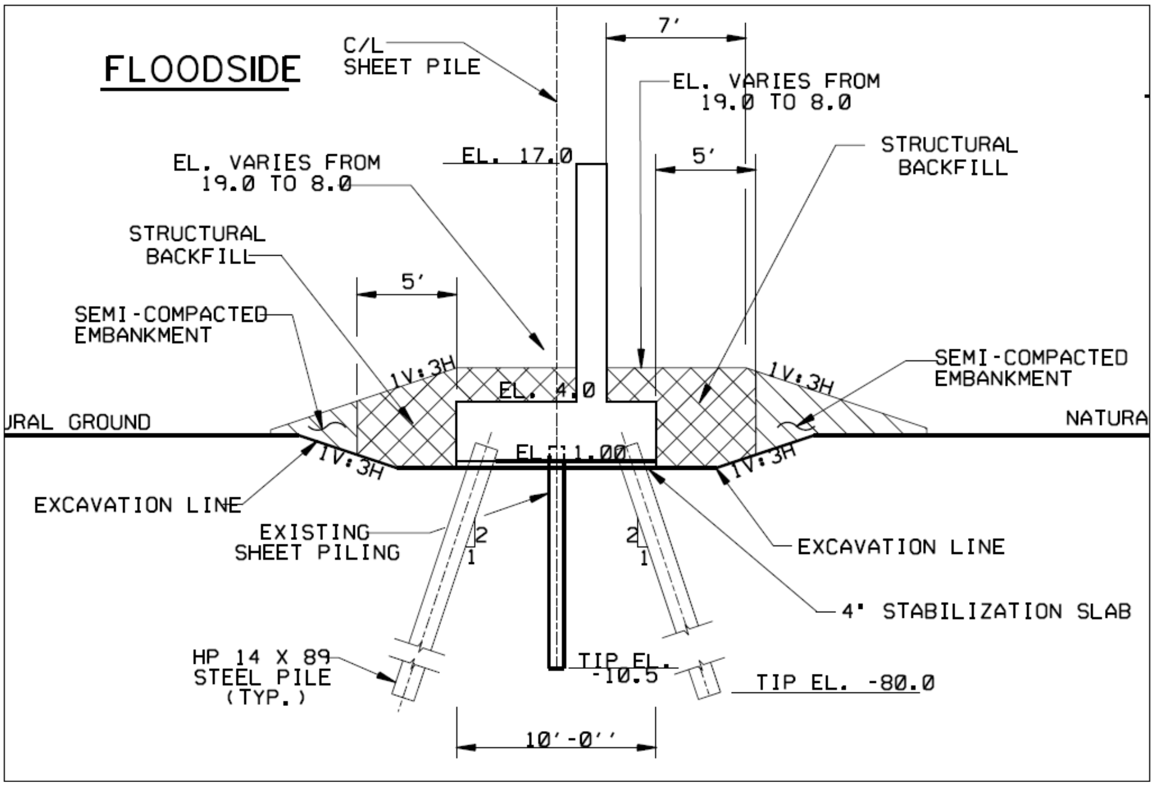


Figure 1. Wall Geometry.

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.

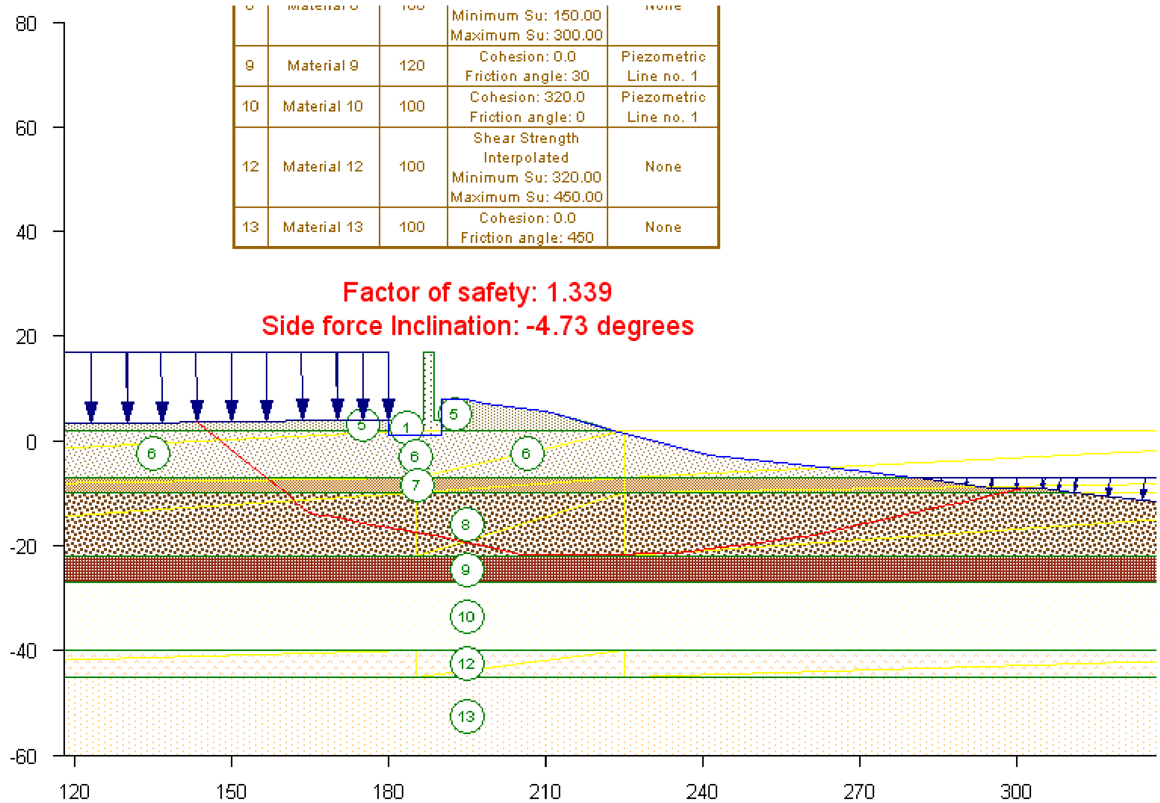


Figure 2. Spencer's analysis of the T-Wall without piles.

Step 2 <u>Unbalanced Force Computations</u>

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.

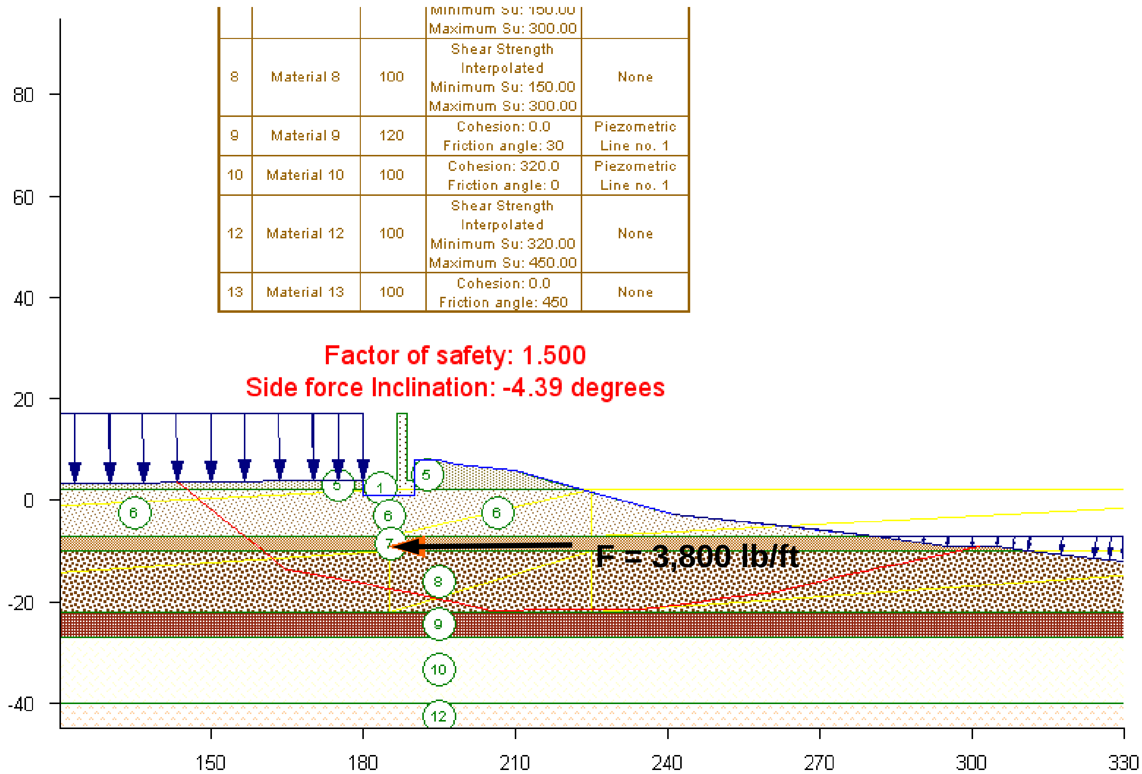


Figure 3. Spencer's analysis of the T-Wall with an unbalanced load to increase global stability (note FS is slightly below target FS=1.5 in this example).

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.

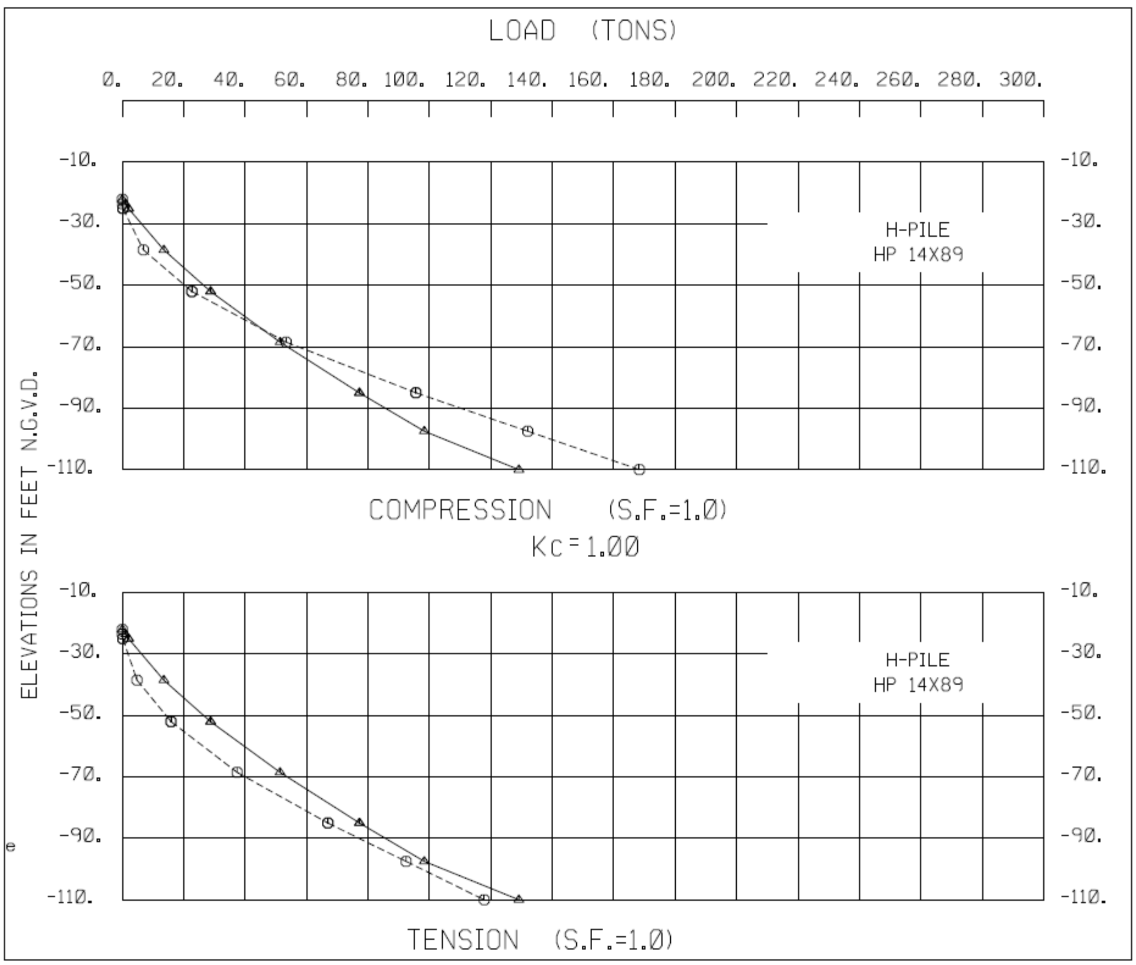


Figure 4. Ultimate Axial Capacity with Depth, Calculated

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 <u>Initial T-wall and Pile Design</u>

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.

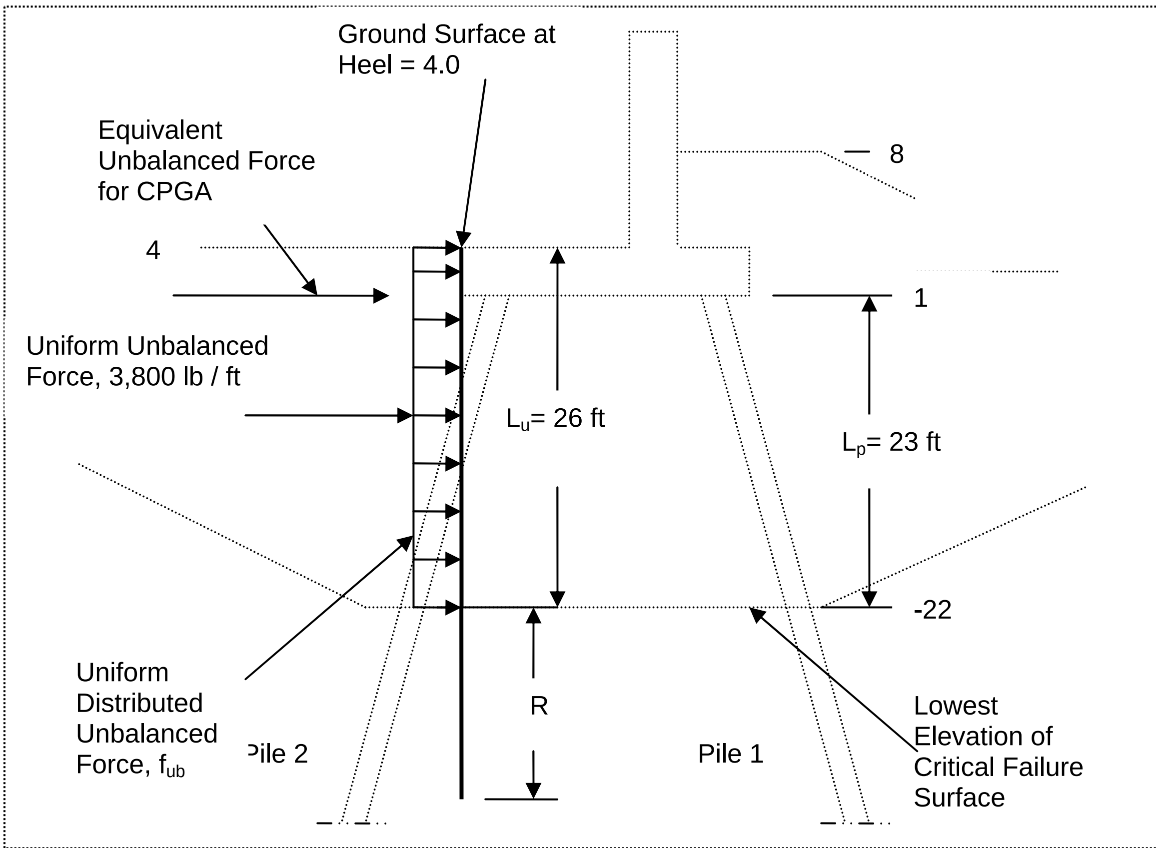


Figure 5. Equivalent Force Computation for Preliminary Design with CPGA

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.

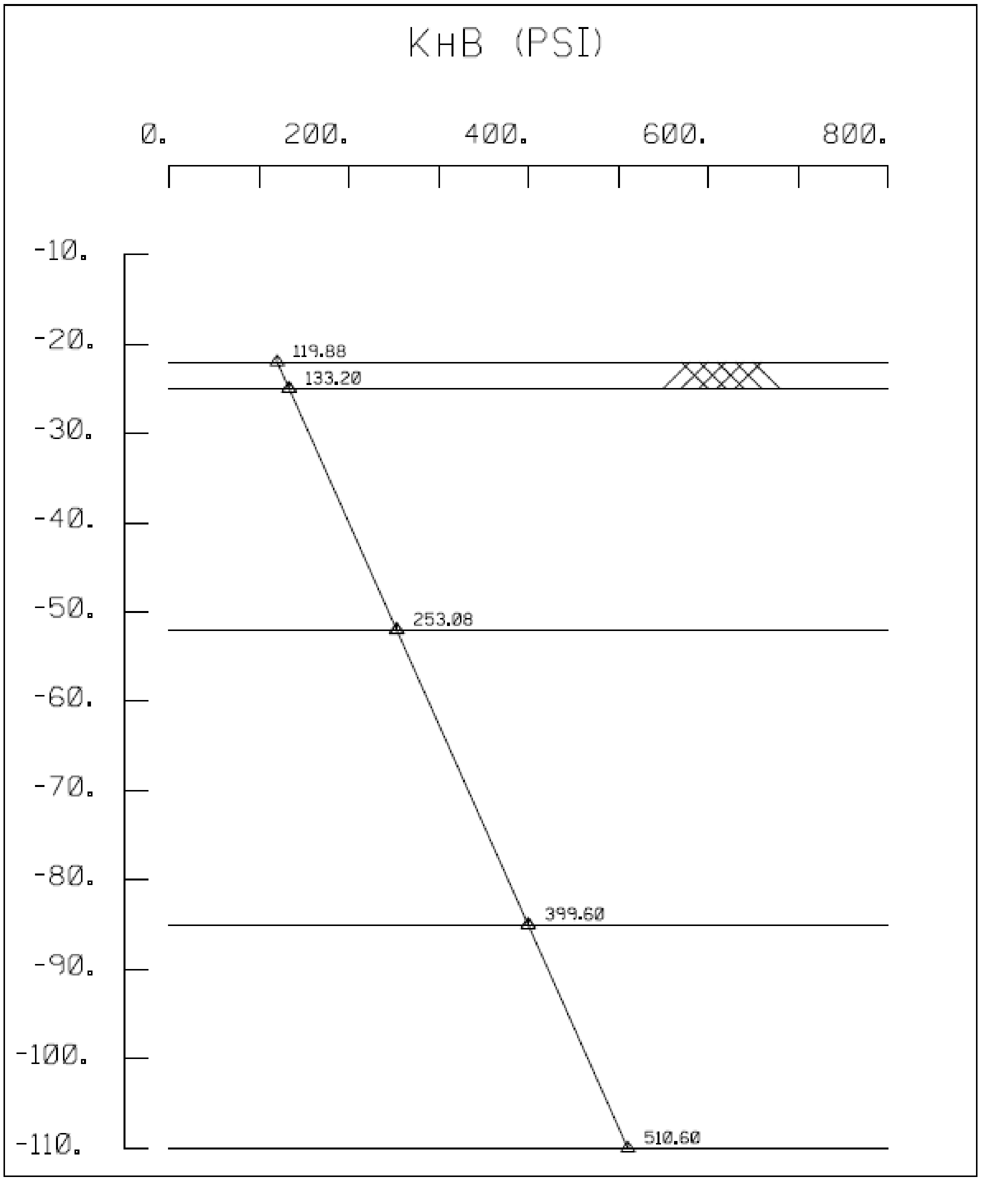


Figure 6. Soil Stiffness with Depth

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 IN	DZ IN	R 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:

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

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 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

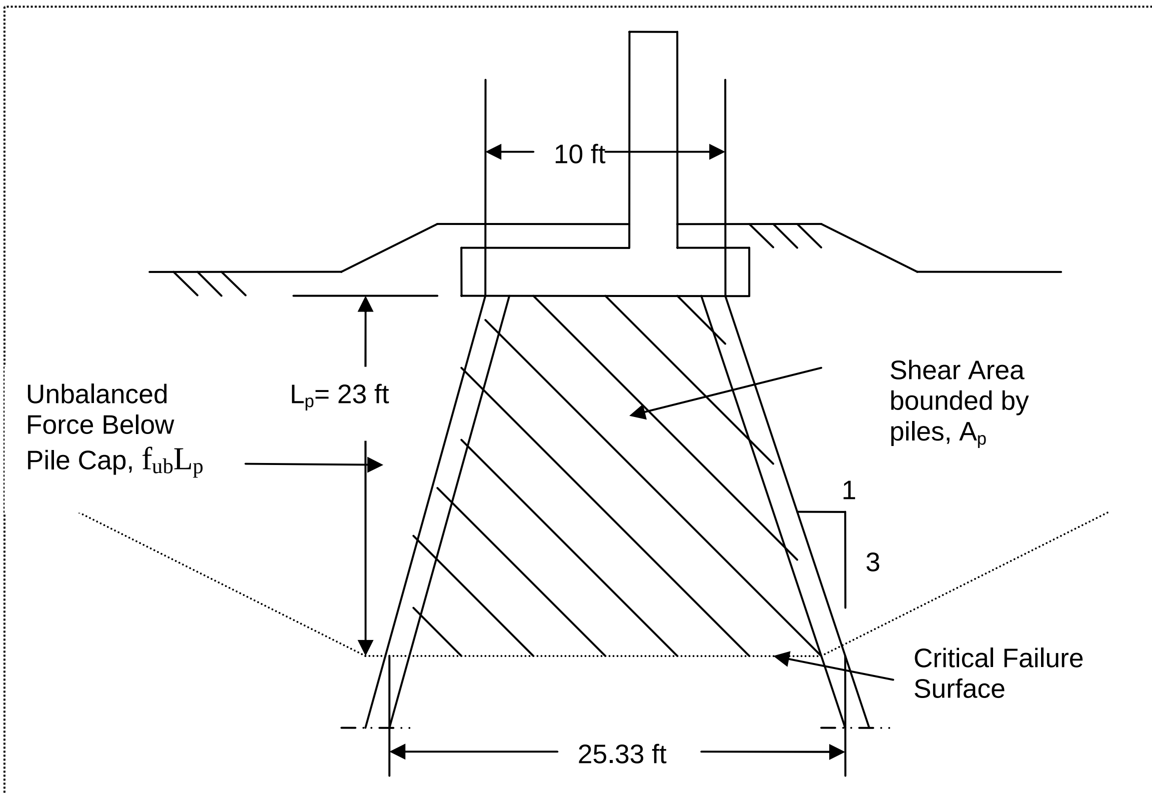


Figure 7. Shear Area for Flow Through Calculation

Step 5 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 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 lb) = 72,450 lb

50% F_p = 1,679 lb

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

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

5.4 The Group 7 model is shown in Figure 8.

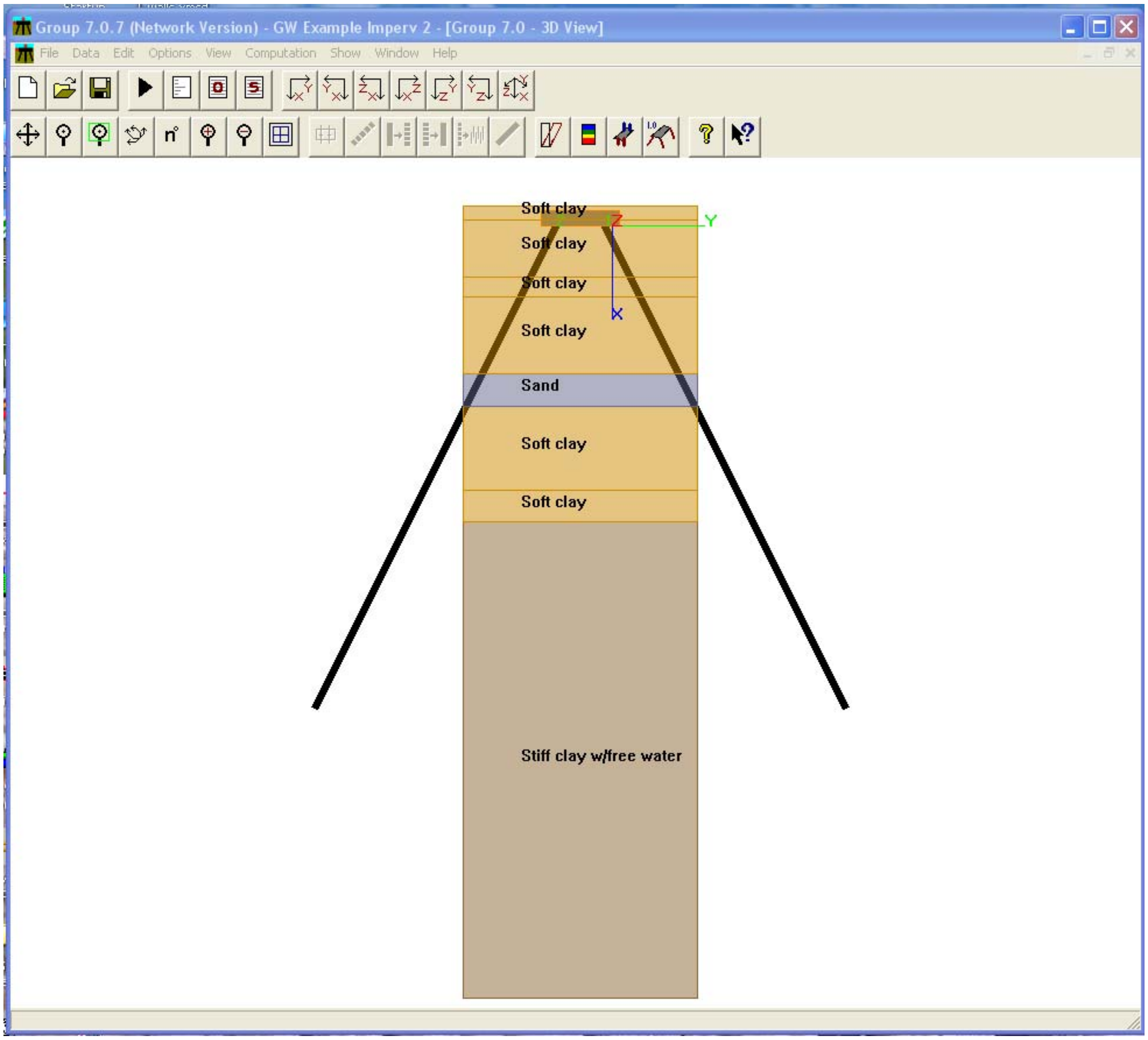


Figure 8. Group 7 Model

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.

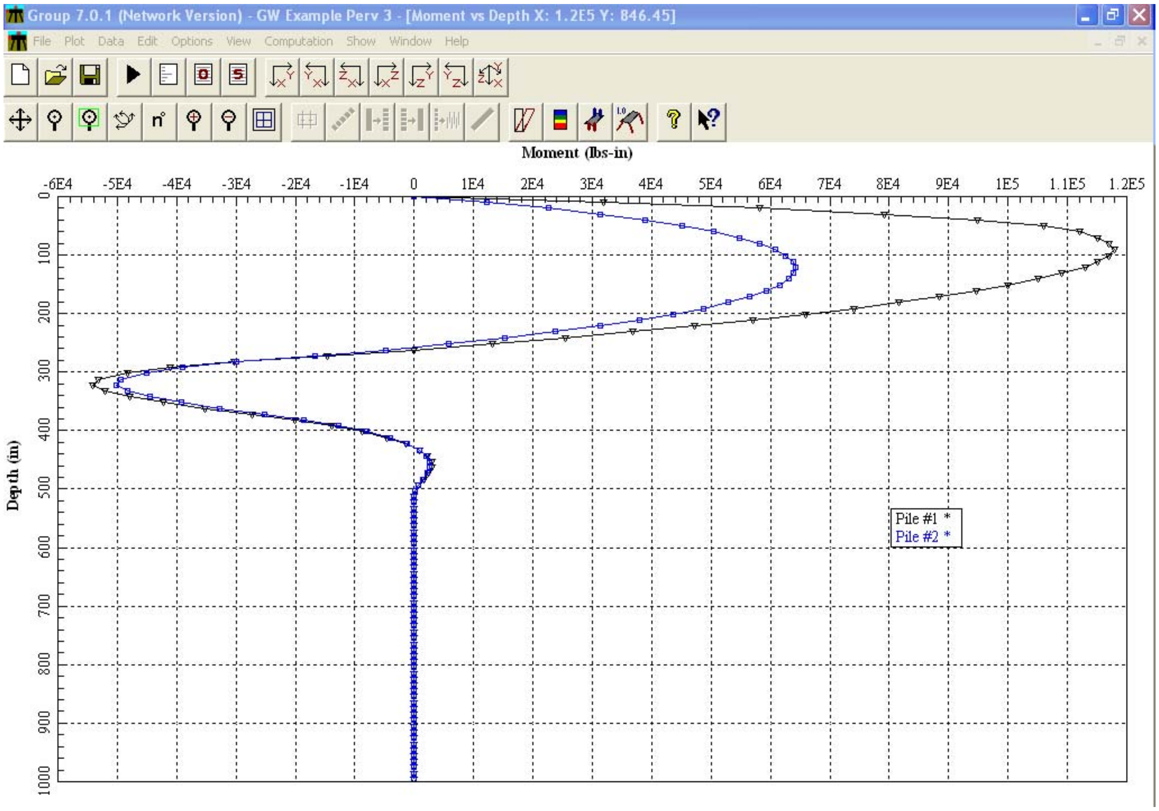


Figure 9. Moment in piles with depth for the pervious case

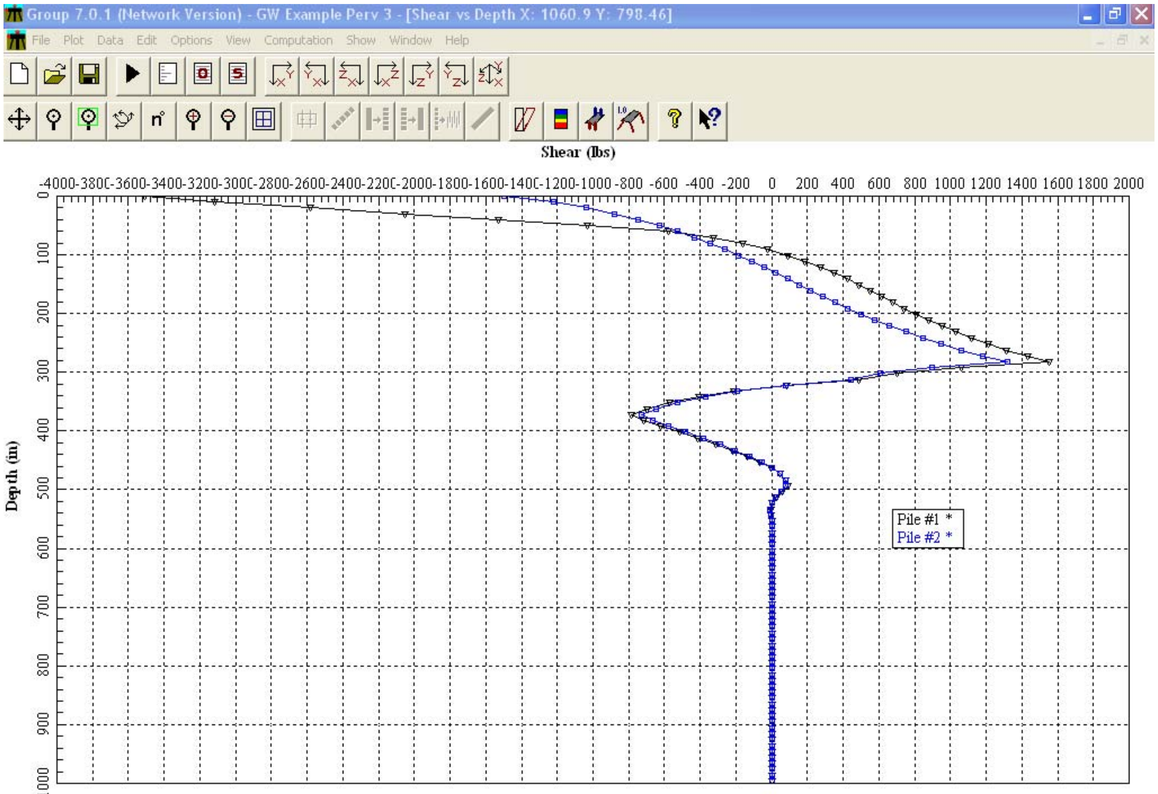


Figure 10. Shear versus depth for the pervious Case.

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.

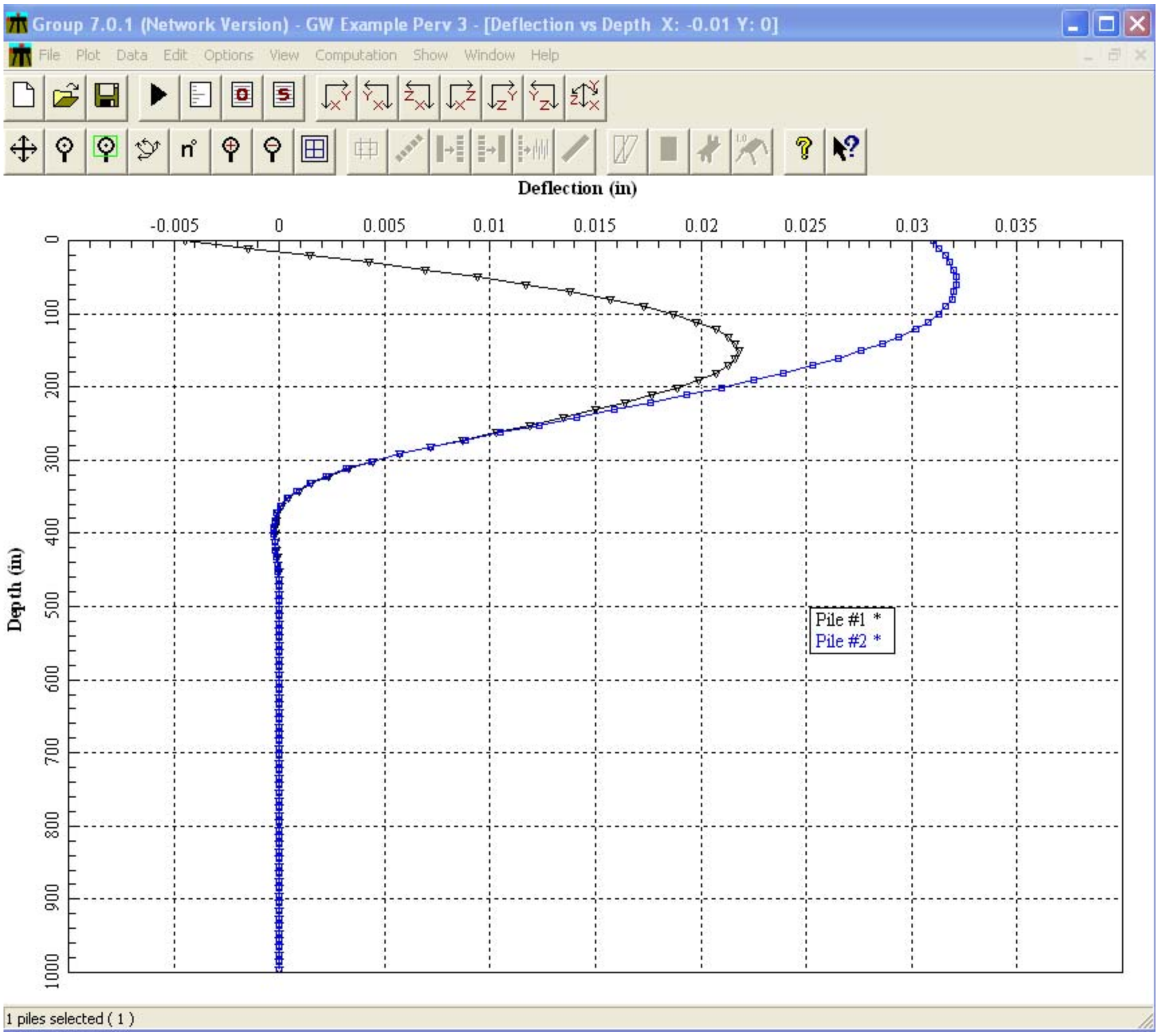


Figure 11. Deflection with Depth for the pervious foundation condition.

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.

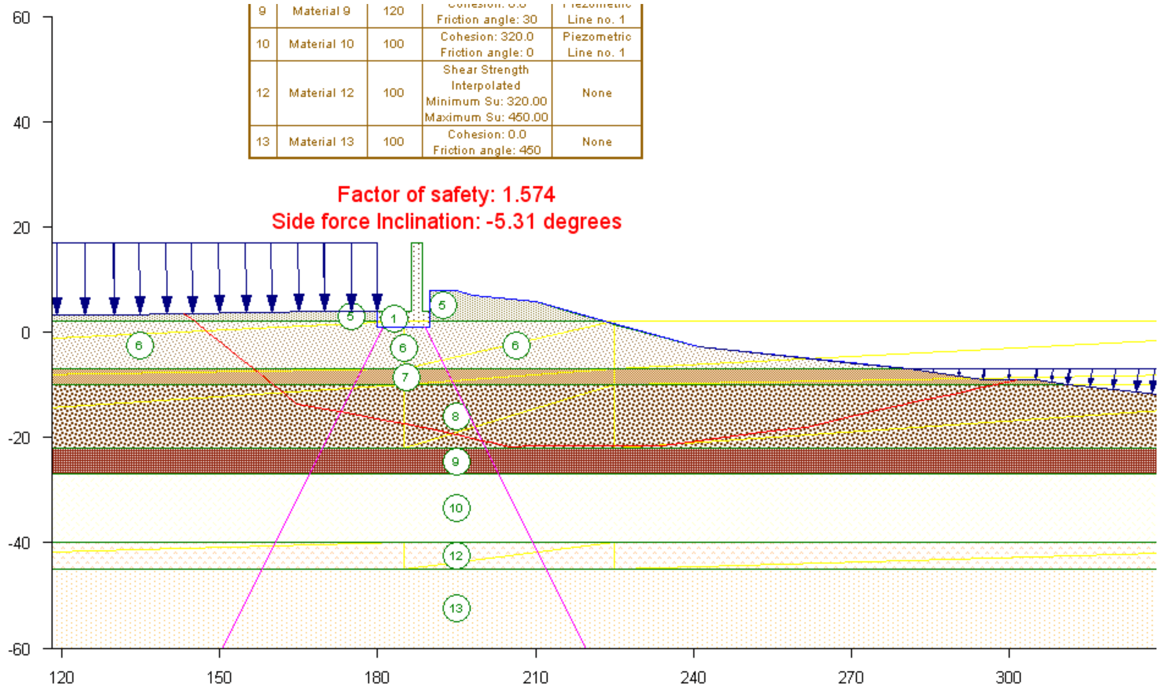


Figure 12. Factor of safety computed using pile forces from Group 7 analysis And critical failure surface from Step 2

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
```

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: 07/03/07	SHEET:
	SUBJECT TITLE: Gainard Woods, Pervious	CHECKED BY:	DATE:	

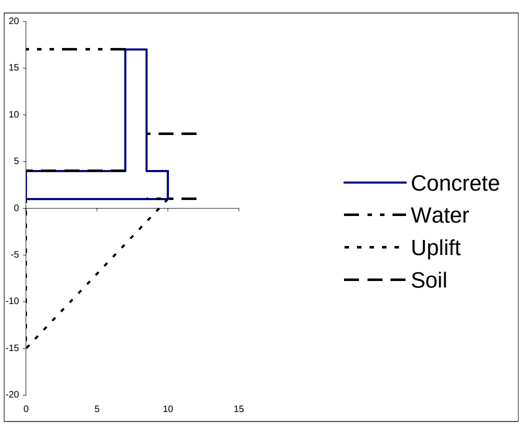
Input for CPGA pile analysis **Pervious Foundation Assumption**

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

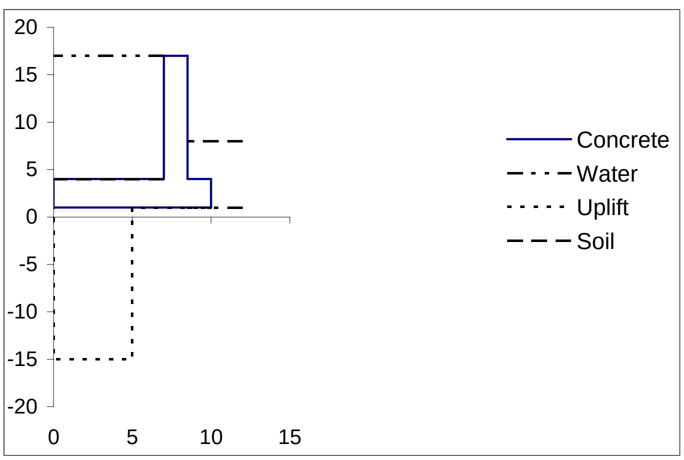
Vertical Forces

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

UPDATED 23 OCT 07

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 K/IN**2	LENGTH	L FT	LU 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 FT	Y FT	Z FT	BATTER	ANGLE	LENGTH FT	FIXITY
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         C        PHI,DEGREES     E50        FMAX       TIPMAX
  IN     LBS/IN**2                           LBS/IN**2    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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