REACHING SETTLEMENTS: USING DESIGN MEDIATION TO RESOLVE SUBSURFACE CONFLICT BETWEEN FLEXIBLE AND RIGID FOUNDATIONS WITHIN LEVEE SYSTEM TRANSITION ZONES

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ABSTRACT

The Joint Venture design team recognized during the USACE Independent Technical Review (ITR) process that the transition from the proposed the T-wall section of the levee system to a proposed 16 feet high earthen levee required a detailed understanding of settlement response within this transition zone. The design review was performed on two segments of the New Orleans West Bank and Vicinity (WBV) Hurricane Storm Damage Risk Reduction System (HSDRRS) where the estimated settlements made it clear that careful design consideration was imperative given the anticipated settlements of the pile supported T-wall monoliths were less than one inch, and settlements of the abutting earthen levee embankment were estimated in excess of several feet.

Ground improvement through preconsolidation of thick soft to very soft compressible silt and clay soils was selected for the transition zone where T-wall construction supported by deep pile foundation ends and earthen embankment placed on existing subgrade begins. Preconsolidation was achieved by the use of vertical and lateral drains (i.e., geosynthetic composite drains), a sand blanket drain at the surface and a designed pre-load soil mass. The preconsolidation of the transition zone addressed two significant design concerns: 1) Overstressing of steeply battered piles as a result of free field soil mass movement across the piles during consolidation; and 2) Large settlement beneath the T-wall bases, leading to separation and development of voids as the subgrade settles away from the pile supported base.

A geotechnical instrumentation plan was developed to monitor settlement and porewater pressure response within the foundation soils in response to preloading and evaluate when sufficient consolidation had been reached. This paper presents the data collected from the instrumentation which include vibrating wire settlement gauges and piezometers as well as standpipe settlement platforms and inclinometers.

Using the observed settlements, this paper presents an evaluation of battered pile bending stresses that were mitigated through pre-consolidation. Bending moments are estimated using the method developed by researchers at Virginia Tech and the USACE who were commissioned by the USACE to develop a practical, straightforward approach for engineering design of T-wall pile foundations using the computer program LPILEby Ensoft, Inc.

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BACKGROUND

Since the inundation of New Orleans by Hurricane Katrina, the USACE has implemented design guidelines for the Hurricane and Storm Damage Risk Reduction System (HSDRRS) throughout the greater New Orleans area. This system includes the West Bank and Vicinity (WBV) WBV-09b project located on the eastern extremity of the hurricane protection system (HPS) and WBV-74 that is on the western extremity of the HPS (Figure 1). Both projects are located on the west bank of the Mississippi River. The WBV-09b project crosses Hero Canal, and WBV-74 project crosses Sellars Canal.

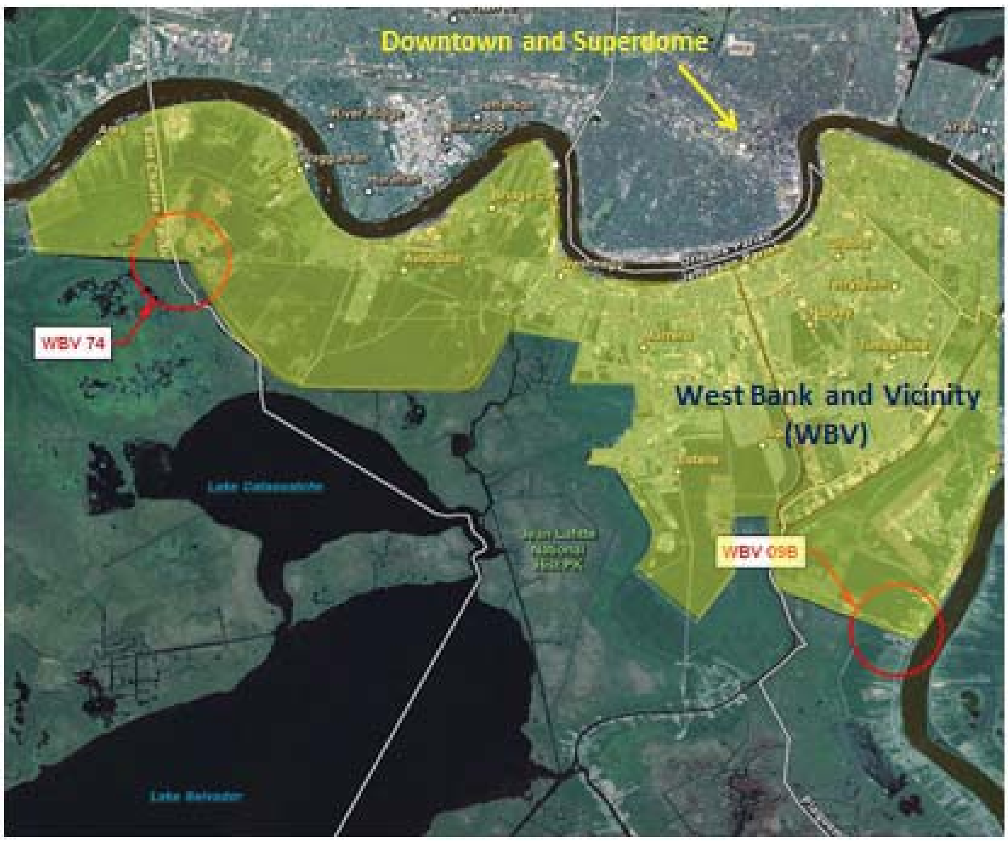

Figure 1. Site Vicinity Map of WBV-74 (Sellars Canal) and WBV-09b (Hero Canal)

The work associated with the HSDRRS includes the design and construction of structural concrete floodwalls (inverted T-walls supported by battered piles) and earthen embankment levees to withstand 100-year storm events (design storm). These two barrier systems present design challenges where these flood protection structures are constructed over soft and very soft soil foundation conditions. The transition from structural T-wall elements to earthen embankment required the design team and the USACE to carefully assess the potential impacts of large differential settlements across the transition zones between the rigid floodwall and flexible levee system components.

The USACE developed and published a five step T-wall design procedure that is contained within the HSDRRS design guidelines dated June 2008. This procedure outlines the typical T-wall design criteria including:

- Performing an initial slope stability analysis to compute the factor of safety of the T-wall section under still water and top of wall level loading using non-circular failure analysis including the Lower Mississippi Valley Division's Method of Planes (LMVD MOP) and Spencer's Method. These factors of safety are compared against the required minimum factors of safety (i.e., target factor of safety).
- Identifying the critical failure plane as the failure surface that produces the greatest unbalanced load (i.e., lateral subsurface load required to achieve the target factor of safety).
- Establishing allowable single pile axial (tension or compression) capacities. This computation ignores skin friction above the critical failure plane. Computing the allowable shear load using an iterative analysis and Ensoft, Inc.'s program LPILE to estimate an ultimate shear capacity. The ultimate shear load is then reduced by the factors of safety used for axial capacity design.
- Computing an initial T-wall and pile design using the computed equivalent unbalanced force that acts on the pile cap and considering soil modulus reductions and soil flow through the pile foundation.
- Performing pile group analysis to yield the response of the piles to the loads applied to the T-wall System.

This procedure provides a T-wall design where the piles supporting the wall also provide the subsurface lateral resistance required to yield a suitable factor of safety for global stability. The design process for the Hero and Sellars T-wall structures resulted in three rows of battered piles with two rows on the protected side and one row on the flood side of the T-wall. To resist unbalanced forces and limit lateral deflection to less than 0.5 inch, the T-wall foundation piles are designed with a batter of 2.5 vertical (V) to 1.0 horizontal (H). Pile tip elevations typically exceeded elevation -95 feet (NAVD88 2004.65) developing nearly all of their capacity from skin friction due to the thick deposits of normally consolidated clays at each project site. Located between the protected side and flood side pile rows is a sheet pile cutoff wall that extends to elevation -35.0 feet for control of underseepage beneath the T-wall monoliths. Figures 2 and 3 below depict the typical section and plan of the T-wall monolith design.

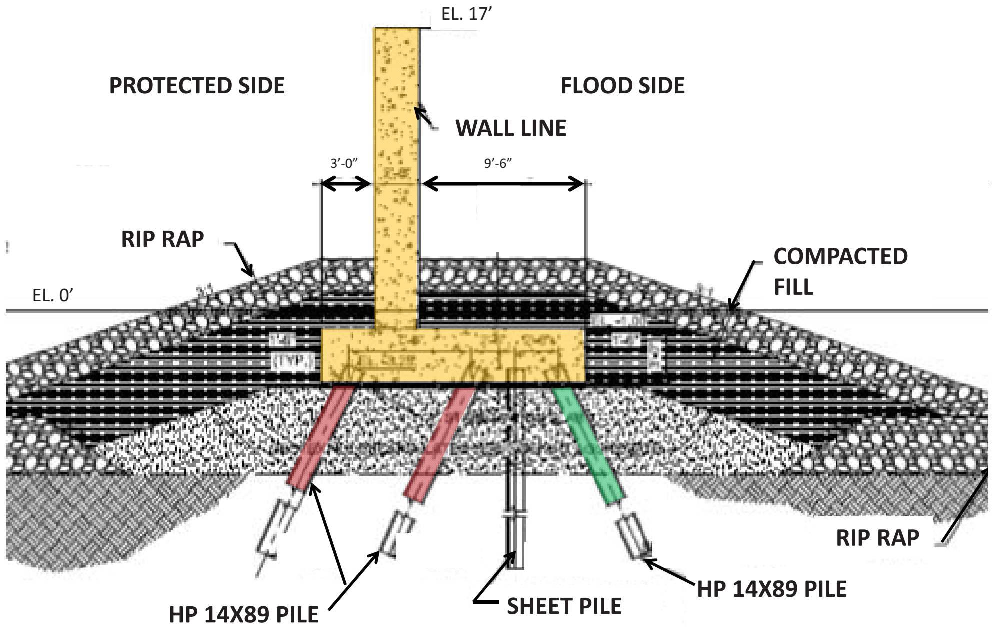

Figure 2. General T-wall Monolith Pile Foundation Layout (Cross-Section View)

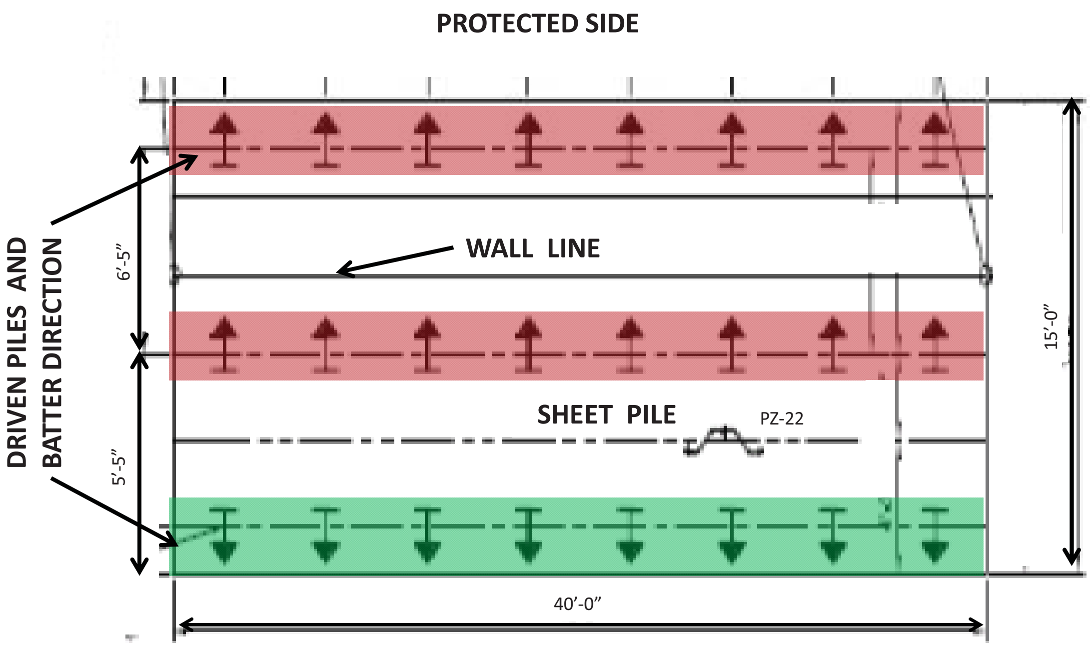

Figure 3. General T-wall Monolith Pile Foundation Layout (Plan View)

At WBV-09b and WBV-74 the T-wall monoliths are the structural components that provide a continuous line of flood protection from gated structures located within the existing, navigable canals to earthen levees constructed over land. At the transition (or wrap-around) where the earthen levee meets the pile supported T-wall monoliths founded on steeply battered pile foundation, the potential exists for overstressed piles as the increased overburden stress from the earthen levee consolidates the foundation soil used to develop pile support. This potential is exacerbated because these levees are being constructed concurrently with the T-wall structures; therefore, the foundation soils at the levee/T-wall tie-in do not have the opportunity to consolidate prior to construction of T-walls. . The USACE was tasked to have the HSDRRS constructed in 2011 as mandated by a Federal Congressional Committee following Hurricane Katrina. In order to achieve this goal, the design and construction schedule for WBV-09b and WBV-74 (among many other projects) was accelerated.

Figure 4 below illustrates the typical section through the earthen levees. The levees are approximately 200 feet wide from the flood side toe to the protected side toe. The flanks on both sides are gently sloped to the center of the levee section which has a 15 to 16 feet wide crest with 3H:1V side slopes. The center section of the earthen levee is approximately 60 feet wide with a height of 14 to 16 feet. The center section of the levee overlaps the transition monoliths to complete a continuous crest elevation that has a one-percent chance of being equaled or exceeded in a given year (100-yr design storm).

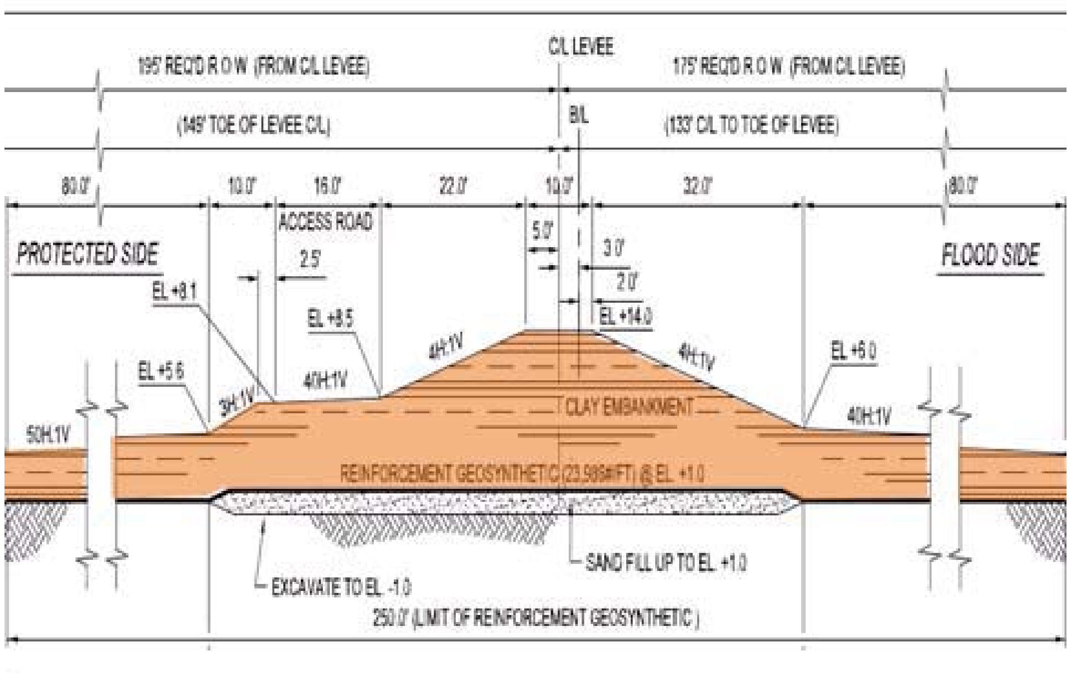

Figure 4. Typical Earth Embankment Section

FOUNDATION SOILS

The soils in the vicinity of each canal that require deep battered pile foundation for the T-wall support are highly compressible under the added weight of the earthen levee depicted in Figure 4. Representative conditions of these compressible soils may be characterized as shallow foundation soils from 0 to 30 feet and deep foundation soils 31 to 70 feet having the soil parameters presented in Table 1. These soils are typically weak and normally consolidated in the upper 70 feet underlain by stiffer soils that have some overconsolidated zones.

Table 1. Foundation Soils

Soil Properties	Shallow High Plasticity Clay (0 to 30 feet)	Deep High Plasticity Clay (31 to 70 feet)
Wet Density (pcf)	118	123
Liquid Limit, LL (percent)	97	83
Plastic Limit, PL (percent)	72	60
Void Ratio	2.12	1.83
Compression Index (C_C)	0.97	0.96
Undrained Shear Strength (psf)	125 to 250	250 to 1,000

The soil conditions represent prodelta silts and clays as depicted by generalized stratigraphic sections of the Mississippi River overbank deposition (Figure 5). The WBV-09b and WBV-74 projects were to be constructed in the low lying areas away from the natural levees (i.e., regions on the left and right sides of Figure 5).

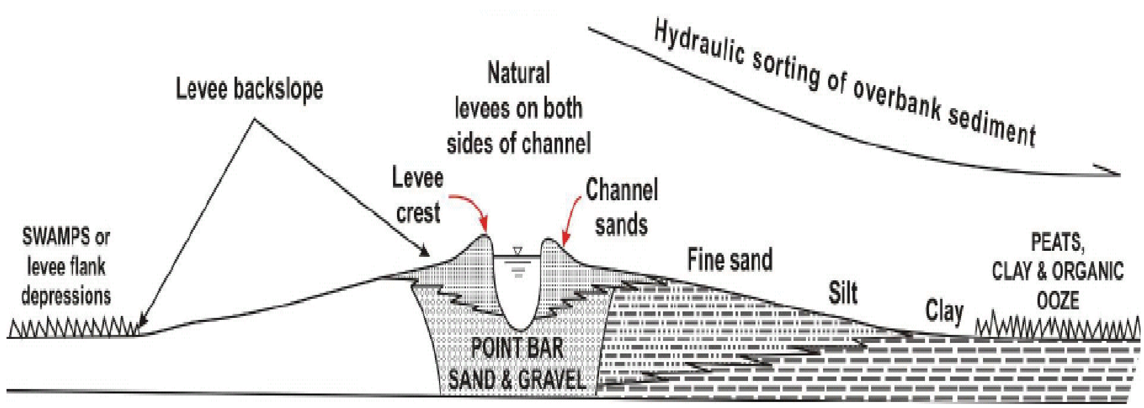

Figure 5. Generalized Stratigraphic Section (Source: Rogers 2007)

The foundation soils in the upper 70 feet were deposited during the Holocene Epoch (termed "Recent" deposits). As previously stated, soils below the upper 70 feet are stiffer, and these soils have an older geologic origin (Pleistocene Epoch). Settlement analysis of the foundation soils found potential ground surface settlement in response to earthen levee construction to range between 2 and 4 feet. The technical reviews conducted by the design team and USACE initiated a formal discussion regarding the impact that settlements of this magnitude would have on battered piles supporting the transition T-wall monoliths. Review of potential analytical approaches that could be used to analyze the stresses of the free field vertical soil movement of the consolidating soil mass on the piles was not well-defined in the engineering community. With this conclusion, the USACE commissioned Virginia Tech to develop a practical, straightforward approach for engineering design of T-wall pile foundations using the computer program LPILE. This "LPILE Method" is documented in two papers listed in the "References" section at the end of this paper.

The objective of the Virginia Tech and USACE study was to evaluate downdrag forces resulting from a consolidating soil mass recognizing that batter piles used to support T-walls are subjected to a component of the downdrag force that acts normal to the pile axis. This lateral soil force (p) has a corresponding lateral soil deflection (y) and is best defined as soil springs acting along the length of the pile. This p-y response produces bending moments in the pile. The bending moments due to the consolidating foundation soils can be significant, and with no clear validated design approach, Virginia Tech and the USACE began the study to provide guidance for estimating the bending moments. To progress the design and issue bid documents in time for construction in 2011, the design team and USACE elected to maintain the current pile size and system design and mitigate the potential for excessive bending moments through ground improvement methods.

DESIGN SOLUTION

Given the earthen levee settlement was predicted to be several feet and settlement of battered H-pile supported T-wall was estimated to be an inch or less the design team considered that a "hard point" would develop at each of the levee system transition zones. The large amount of differential settlement had the potential to induce excessive bending moments in the steel H-piles supporting the T-walls.

The design solution was ground improvement through implementation of an earthen preload program monitored by geotechnical instrumentation understanding that a consensus on an analytical approach to size a pile section to resist the anticipated bending stresses (and moments) could not be achieved during the fast track design phase of the project. To mitigate forces transferred from free field displacement of the consolidating foundation soil mass to batter piles, the design team developed a design for vertical drain installation and preloading of the transition area between the transition T-wall monoliths and the wrap-around section of the earthen levee at both the Hero and Sellars project sites. Figure 6 below is a photograph of the Sellars project site (view looking north). At the time of the photograph a sector gate is being constructed in the canal, and earthen levees are being constructed on the east and west sides. Floodwalls (i.e., inverted T-wall transition monoliths) will extend between the levees and the gated structure in the canal.

Figure 6 - View of WBV-74 (Sellars Canal) during construction (looking north).

For each project site the wick drain and preload design generally consisted of two zones of wick drains both zones with an approximate width of 110 feet. The larger zone (Zone 1) extends 130 feet beneath three transition monoliths. Within Zone 1 the wick drains were spaced 5 feet apart in a triangular grid pattern. The second zone (Zone 2) extends from the limit of Zone 1, 75 feet further beneath the proposed footprint of the earthen levee. Within Zone 2 the wick drains were spaced 10 feet apart in a triangular pattern. The wick drains were installed to a depth of 95 feet below existing grade within both zones using a mandrel to hydraulically penetrate the foundation soils and set each wick drain. The vertical wick drains were connected at the ground surface to a horizontal strip drain (geocomposite) that conveyed porewater to a point of discharge outside of the zone of consolidating foundation soils.

Above the two zones of wick drains the soil preload was placed consisting of compacted embankment soils. The preload was designed to induce stresses and corresponding excess porewater pressures within the foundation soils underlying the preload. The wick drains were designed to shorten the subsurface drainage distance within the thick prodelta clay deposits and allow the excess porewater pressure to efficiently dissipate, thus shortening the consolidation process. The estimated time based on consolidation theory to achieve 85 percent consolidation of the foundation soils was 6 months for the specified wick drain spacing and magnitude of the preload. Without the use of wick drains the time of 85 percent consolidation was estimated to be approximately 30 years. By using wick drains, the actual observed time of 85 percent consolidation was approximately 4.5 months based on results of instrumentation and a monitoring program.

INSTRUMENTATION

To quantify the amount of settlement and to monitor the degree of consolidation over the preconsolidation period, an instrumentation program was implemented. The instrumentation program consisted of vibrating wire (VW) piezometers, VW-settlement gauges, and settlement plates/risers within a preload compacted embankment. A description of each instrument is provided below:

1. VW-piezometers. VW-piezometers consist of pressure head transducers designed to measure fluid pressures such as porewater pressures within foundation soils. The VW-piezometers were installed within boreholes, and fully grouted. The instruments selected provided a range of pressure readings suitable to the depth of installation and the anticipated induced pore pressure levels.

2. VW-settlement gauges. A levee settlement system was selected that uses a pressure transducer attached to a settlement plate. Plates were located on the subgrade surface within the footprint of the levee or T-wall monoliths where the preload was constructed. The transducers include a connection, via two fluid-filled tubes extending laterally, to an instrument reservoir located on firm ground away from the area of anticipated movement. Fluid pressure within the tubes is sensed by the transducer which provides a measurement of the elevation difference between the sensor and the reservoir.

3. Settlement plate/risers. To check the VW-settlement system, settlement plates and survey monuments were constructed within the preload soil mass. These monitoring points were periodically surveyed and the data were compared with the VW instrumentation readings.

The instrumentation layout was developed based on the design of the preload footprint and the alignment of the transition monoliths. For each project site five (5) instrument locations were selected with four (4) of the locations aligned with the “wall-line” and one (1) location offset from the wall line approximately 75 ft. perpendicular of the proposed T-wall monoliths. The instrumentation was installed within piezometers in the case of pressure head transducers and for the settlement gauges at locations adjacent the piezometers at the ground surface. The approximate locations and configuration of the instrumentation layout is shown on Figure 7.

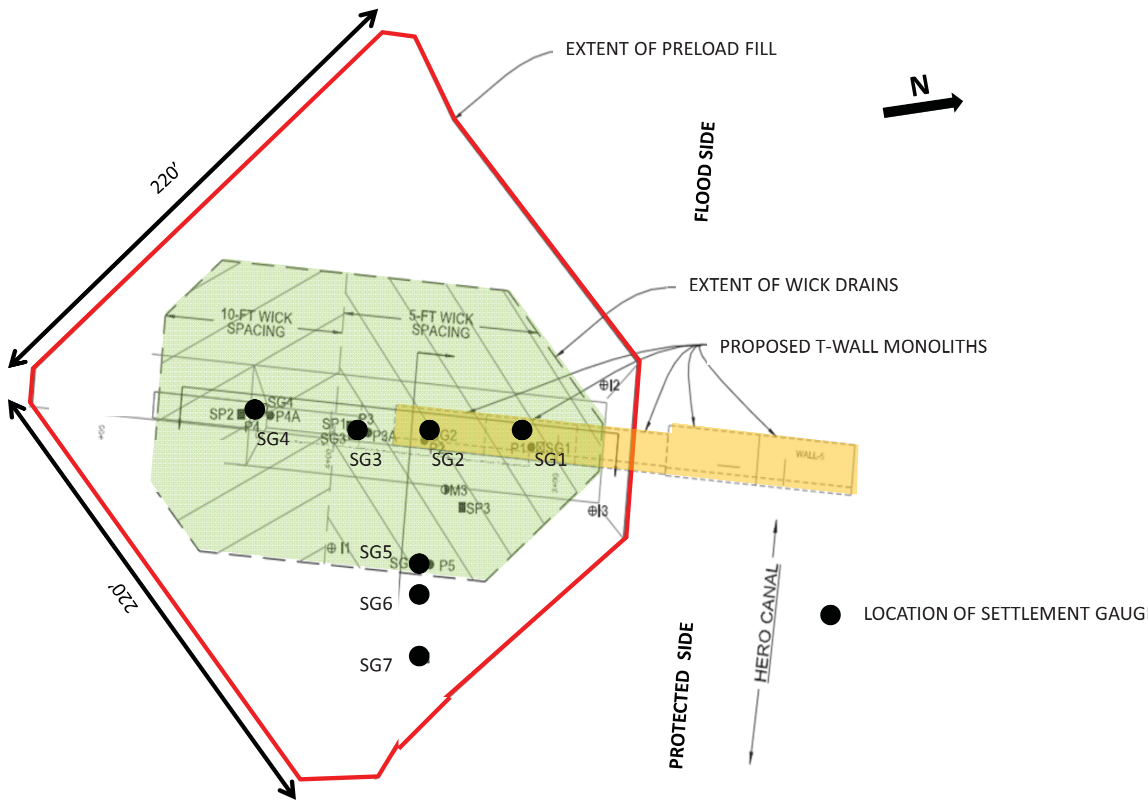

Figure 7. Preload Stack Limits, Vertical Drain and Instrumentation Layout
(Hero Project Site)

Signal and fluid-filled cables from piezometers and settlement gauges were carried from each instrument location within conduit buried within instrument cable trenches. Trenched cable was brought to readout and data logger locations positioned on the protected side of the levee beyond the levee toe of slope. The data loggers were powered by solar panels given power was not available during this phase of construction.

With the instrumentation inplace and wired to data loggers the preload stack construction was initiated and the development of excess porewater pressure monitored to calculate B-bar values to insure that the rate of pre-load stack construction did not trigger foundation soil bearing capacity failure. Each preload stack was constructed over a couple of weeks in October and November2010. Upon completion of preload stack construction, foundation soil consolidation monitoring continued with the tracking of both excess porewater dissipation and total settlement over time. After 22 weeks the rate of consolidation had slowed and the excess porewater pressures had dissipated sufficiently to conclude that sufficient consolidation had been achieved. The preload stacks were removed in March 2011 (Hero Canal) and April 2011 (Sellars Canal). Data from instrumentation was reduced and evaluated to assess the success of the preload program. Figure 8 below is a summary plot of five settlement gauges over the duration of the program at the Hero Canal site. The figure illustrates the magnitude of settlement over time for each settlement gauge. A maximum ground surface settlement of 56 inches was observed at a settlement gauge SG-3. This settlement gage was positioned near the apex of the preload stack which is where the T-wall meets the full levee embankment (i.e., edge of T-wall Monolith No. 9).

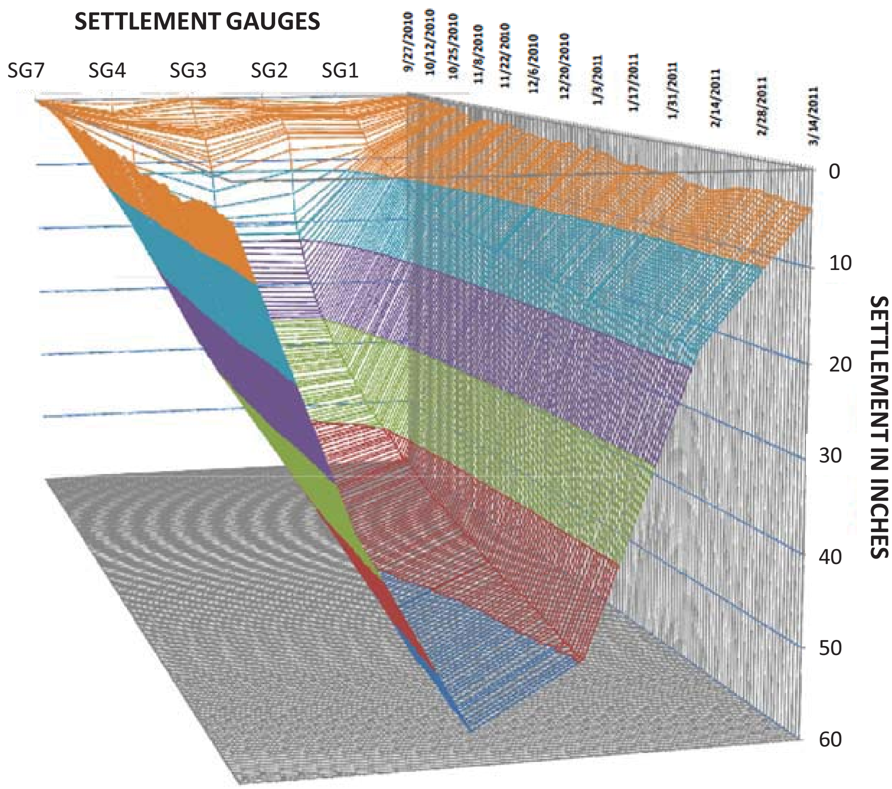

Figure 8. Summary Plot of Measured Settlement Below Preload Stack (Hero Canal Project Site)

LPILE METHOD — VALIDATION OF PRELOAD PROGRAM

Following the preconsolidation of the foundation soils beneath the transition zone monoliths, the design team used the settlement data obtained from on-site instrumentation and applied the LPILE Method for calculating pile bending moment stresses in battered piles as developed by Virginia Tech and the USACE. This method was performed using the Revised LPILE Method to Calculate Bending Moments in Batter Piles for T-Walls Subject to Downdrag, prepared by Michael McGuire and George Fliz of Virginia Tech dated December 2010. This method accounts for a nonlinear settlement profile with depth.

Two (2) settlement cases were analyzed to understand the impact excessive ground movement has on battered pile foundations supporting T-Wall monoliths and to arrive at a tolerable settlement where free field vertical soil movement would not overstress the pile supported T-wall foundations. To understand the impact excessive ground movement has on battered pile foundations, the LPILE Method was applied at T-Wall Monolith No. 9. As previously indicated, this is the last transition monolith along the alignment before the protection becomes a full earthen levee embankment. The details for the two settlement cases analyzed using the LPILE Method include:

Case No. 1 - Estimate Bending Moments assuming foundation soil preloading does not occur: For this analysis the LPILE Method estimated the bending moments in the batter piles due to ground settlement assuming a preconsolidation program was never executed and the construction of the earthen embankment took place over the highly compressible soil mass.

Case No. 2 – Given an Allowable Bending Moment, what is the Tolerable Settlement?: For this analysis the strength properties for the specified steel H-pile were used to establish a maximum allowable bending moment. This value was then used in the LPILE Method to estimate the tolerable amount of settlement from levee embankment loading of T-Wall Monolith No. 9. The analysis used the results from 3-dimensional settlement modeling as obtained from the program Settle^{3D} developed by RocScience, Inc. The settlement modeling was compared to the settlement data obtain from monitoring instrumentation during the 2010 and 2011 preloading program discussed previously. The comparison was found that predicted ground movement was in general agreement with the measured readings in the field, on this basis the predicted ground movement information from Settle^{3D} was used as input for LPILE analyses.

The LPILE Method analysis for calculating bending moments on the battered piles under the two settlement cases described above was performed using the assumptions listed on Table 2 below.

Table 2. LPILE Method Assumptions

1	The T-Wall monolith foundation was constructed in native soils without the implementation of a preload program and then followed by the construction of a 14-ft earthen levee embankment adjacent to the T-Wall monolith.
2	The foundation soils consist primarily of compressible clay extending 140-ft below the ground surface at El 0, with batter piles extending from the bottom of the T-Wall from El 2.75 to El -120.0 (NAVD 88).
3	The piles are HP 14x89 grade 50 ksi steel and are battered at 2.5V:1H, corresponding to a batter angle (β) of 21.8^{o} from vertical.
4	A symmetric embankment loading exists on the flood side and the protected side of T-Wall monolith No. 9 consisting of compacted clay fill overlain by a rip rap.
5	The native clay is slightly overconsolidated near the ground surface but is otherwise normally consolidated at depth.
6	Pile head connections to the base of the T-Wall foundation slab are modeled as a pin with negligible moment resistance.
7	Ignore axial loading in each pile as it does not significantly impact downdrag-induced bending stress on the pile.
8	Downward movement of the soil normal to the pile axis is responsible for the bending stresses (and moments) on the pile.

9	Vertical soil movement input for LPILE was referenced from preliminary estimates computed using Settle^{3D}. The estimated settlements were close to those recorded in the field during the preload program and were therefore used as part of the LPILE analysis.

Based on these assumptions, the transition T-Wall Monolith No. 9 has a 15 feet wide base (flood side to protected side dimension) and supports the overlying fill soils. The foundation system is comprised of two (2) HP14x89 steel H-piles battered at a 2.5V:1H configuration towards the protected side and one (1) HP 14x89 steel H-pile battered at the same configuration towards the flood side. Table 3 below presents the relevant pile properties used in LPILE to analyze the bending moment stresses on a HP14x89 steel foundation pile.

Table 3: Relevant H-pile (HP14 x 89) details – Input for LPILE Method

Moment of Inertia, in^4	904
Depth and Width of Section, in	13.8 x 14.7
Cross-Section Area, in^2	26.1
Young's Modulus, lb/in^2	29,000,000

To select the location of settlement analysis (in plan view), the LPILE Method recommends the analysis be based on a vertical soil profile (S_{FS}) located at the plan view position at a distance equal to $0.25(L_c)\sin(\beta)$ from the location where the outer flood-side battered pile intersect the T-Wall base as shown on Figure 9. L_c defines the length of the battered pile located in a compressible soil layer and β defines the pile batter angle. Therefore, a soil movement profile at that distance (12.2 ft) from the outer most flood side battered pile was taken from the Settle^{3D} settlement analysis as input for the LPILE Method.

The vertical movement estimated with Settle^{3D} acts at a vertical distance below the existing ground surface (z_V). For LPILE input to be extracted from Settle^{3D}, the distance along the plane of the pile (z_P) was computed based on the batter angle as illustrated in Figure 9.

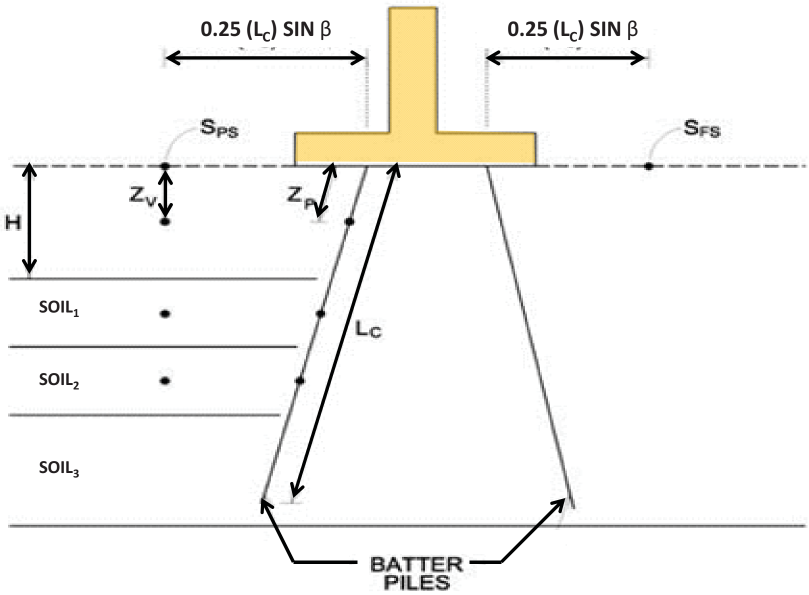

Figure 9. LPILE Method Vertical Soil Profile

The Settle^{3D} analysis was performed to predict free field vertical soil movement resulting from earthen levee embankment construction, and as noted above, the predictive analysis was in general agreement with the measured settlement. Figure 10 below shows the soil movement profile plotted against depth for Cases Nos. 1 and 2. This curve was referenced for input into LPILE.

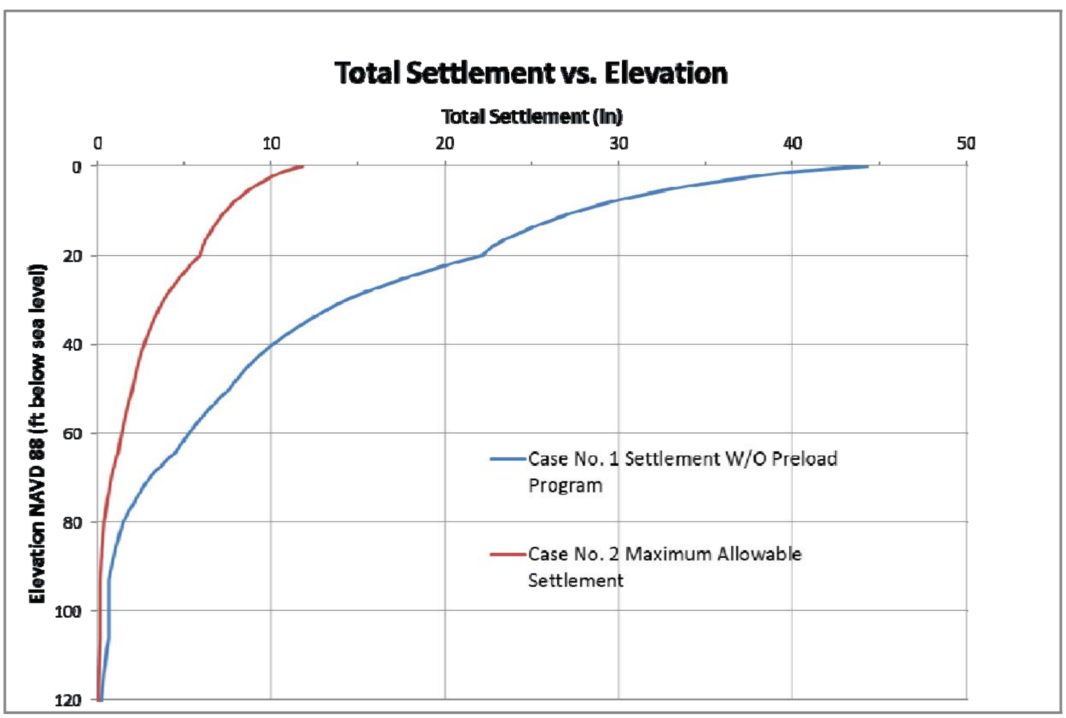

Figure 10. Estimated Soil Movement from Settle^{3D}

Using the estimated settlement from the Settle^{3D} a non-linear analysis was completed to provide input for the Revised LPILE Method preformed on each of the defined settlement cases. For settlement analysis Case No. 1, the far right curve shown on Figure 10 was used as a reference for soil movement estimates from the ground surface to depth. The ground surface movement at the computed location on the protected side of the T-wall was estimated at 44-inches. For settlement analysis Case No. 2, the far right curve (blue curve) shown on Figure 10 was shifted to the left until a settlement value was reached (red curve) that produced a corresponding bending moment that is less than the allowable bending moment for the specified H-pile section.

The soil properties and layer thicknesses were modified prior to being used as input for LPILE following the method shown in Figure 11. This is required because the coordinate system in LPILE is parallel to the pile axis. A factored unit weight for each soil layer considered in LPILE was also transformed by multiplying the unit weight of the material by the cosine of the batter angle.

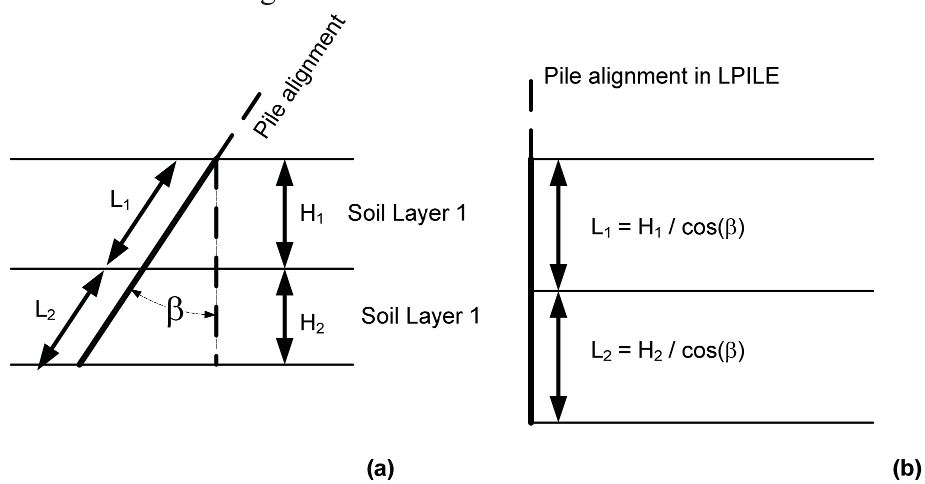

Figure 11. (a) Actual soil stratigraphy, (b) Soil stratigraphy modified for pile

The results of each analysis are outlined below.

Case No. 1 - Estimate Bending Moments assuming foundation soil preloading does not occur: Using the estimated ground surface settlement of approximately 44 inches, as supported by 3-D settlement modeling and settlement gauge data from the preload program, bending moments over the length of the specified steel H-pile were computed. The computation results showed that excessive bending moments developed within batter pile foundation on the flood side of T-Wall in response to the settlement input. The maximum allowable bending capacity of each H-pile following USACE requirements outlined in the HSDRRS design guidelines is approximately 273 kip-ft. Using the Revised LPILE Method, the estimated bending moment in each pile due to free field vertical soil movement was calculated to be approximately 527 kip-ft. This is nearly double the allowable capacity and indicates that piles will be overstressed if a preload program was not used.

Case No. 2 – Given an Allowable Bending Moment, what is the Tolerable Settlement?: Consistent with Case No. 1 the maximum allowable bending capacity of each H-Pile is approximately 273 kip-ft. The LPILE program was used to develop bending moment profiles to determine a maximum bending moment for an assumed surface settlement value. Through an iterative process the ground surface settlement values from surface to depth are systematically decreased to find the value for which a corresponding maximum bending moment of 273 kip-ft is reached. This process found that at approximately 12 inches of surface ground surface settlement the maximum bending moment is equal to the allowable bending moment of the foundation piles. Therefore, for the specified steel H-pile the limiting ground surface settlement of the consolidating foundation soil mass is 12 inches in the vicinity of the transition monoliths.

Figure 12 depicts the estimated maximum bending moment in a foundation pile supporting T-Wall Monolith No. 9. The figure shows the estimated maximum bending moment under the Case No. 1 settlement where it is assumed a preload program was not implemented. The figure also presents the bending moment profile for Case No. 2 where the maximum bending moment is not allowed to exceed the allowable pile bending stress of 273 kip-ft. To keep the maximum bending moment below this threshold value the foundation soil mass surface settlement must be 12 inches or less.

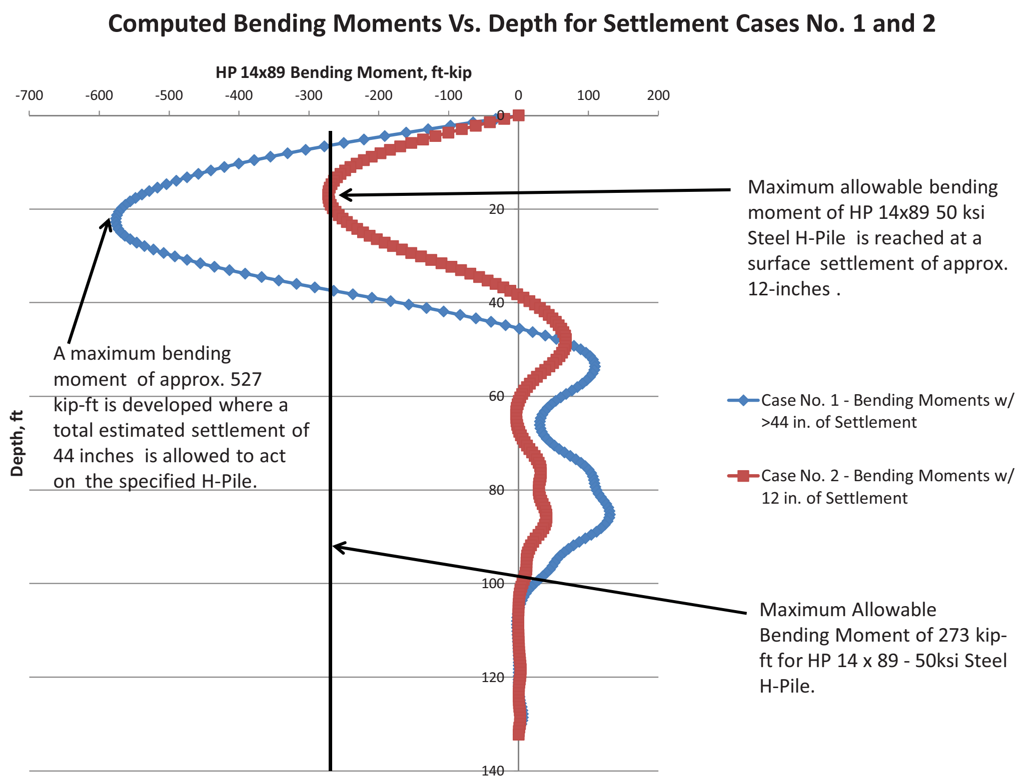

Figure 12. Cases No. 1 and No. 2 Pile Bending Moment Profiles

CONCLUSION

Given an aggressive design and construction schedule as mandated by a Federal Congressional Committee following Hurricane Katrina, the USACE was tasked to have the HSDRRS constructed in 2011. Through independent technical review, a hallmark of the USACE's design process, the design team for the Hero Canal (WBV-09b) and Sellars Canal (WBV-74) projects identified a critical subsurface condition that required innovative design considerations for levee hydraulic barrier construction. The design process identified a transition zone where incompatible foundation types, an embankment on soft foundation soils, and a rigid T-wall monolith on piles overlap inducing the development of a "hardspot". The design solution involved the development of a practical analysis of bending moment within steeply battered piles and the use of an effective ground improvement technique to mitigate the anticipated settlements.

Using ground improvement, the design team developed and designed a preload program to load compressible foundation soils within a designated transition zone, and induce efficient consolidation with the use of vertical and lateral drains beneath the earthen preload. The ground improvement was evaluated by monitoring and recording data from geotechnical instrumentation installed within the preload stack and vertical drainage area footprint. A line of VW-piezometers monitored and recorded the development of excess porewater pressures as the preload soil stack was constructed and then monitored and recorded the dissipation of these pressures as the foundation soils consolidated. At the ground surface vibrating wire settlement gauges arranged in a similar manner as the piezometers monitored and recorded total settlement along and transverse to the T-wall transition monolith centerline. At the conclusion of the consolidation period (approximately 5 months) the maximum total settlement was approximately 56 inches.

Simultaneous to the above activities, the USACE worked with Virginia Tech to develop a practical straightforward approach for engineering design of T-wall pile foundations using LPILE (developed by Ensoft, Inc.). The analytical method developed by the researchers models free field movement of a consolidating soil mass across battered piles and computes the resulting bending moment within a specified pile. This value is then compared to the allowable pile bending stress. Through an iterative process using LPILE, a project specific steel HP 14x89 pile was evaluated to determine at what magnitude of ground surface settlement would the bending moments exceed the allowable bending moment of 273 kip-ft. The results of the evaluation found that settlements in excess of 12 inches across the 2.5V to 1H battered piles would overstress these critical foundation elements. Based on the observed settlements from the preload program exceeded 48 inches beneath the transition zone monoliths, the design team concluded the preload program mitigated the risk of excessive bending moments, and the overstressing of the transition zone monolith battered pile foundations was avoided.

REFERENCES

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