Design and Construction of Circular Secant Pile Walls in Soft Clays

Hadi Suroor
Tolunay-Wong Engineers, Inc., Houston, Texas

Mahi Galagoda
Bechtel Corporation, Houston, Texas

Chris McGhee
Remedial Construction Services, Houston, Texas

6th International Conference on
Case Histories in Geotechnical Engineering, Arlington, VA, August 11-16, 2008

DESIGN AND CONSTRUCTION OF CIRCULAR SECANT PILE WALLS IN SOFT CLAYS

Hadi Suroor
Tolunay-Wong Engineers, Inc.
Houston, Texas, USA-77031

Mahi Galagoda
Bechtel Corporation
Houston, Texas, USA-77056

Chris McGhee
Remedial Construction Services
Houston, Texas, USA-77064

ABSTRACT

This paper presents design and construction aspects of two similar circular Liquefied Natural Gas (LNG) impoundment basins in deep soft clays. Each basin has a design spill containment volume of 70,630 cubic feet. The inside diameter of each basin is 60 ft; bottom of the excavation is 32 ft below grade and the excavation retained permanently by concrete secant pile walls. The circular wall is constructed of 3 ft nominal diameter concrete piles overlapping adjacent piles by 6 in; the wall penetrates 60 ft below grade. Excavation stability during construction is the primary concern in soft clays; an inadequate retention system could experience large wall movements and stresses as well as excavation bottom heave often resulting in failure. A finite element analysis (FEA) was performed to evaluate overall stability of the wall and excavation using axis-symmetric model and to design an excavation-wall system which yields a minimum factor of safety of 1.3 during construction. Soil model parameters were established from back-analysis of performance data from a near-by instrumented dike. The conventional stability analyses were performed to verify the results of FEA; it appears that the method proposed by Bjerrum et al (1956) corresponds well with FEA results. The FEA demonstrated that the circular wall is in compression, in agreement with the theoretical analyses, resulting in negligible movements of wall and ground behind the wall.

INTRODUCTION

This paper presents design and construction of a liquefied Natural Gas (LNG) impoundment basin at a Regas terminal in Texas Gulf coast near the Louisiana/Texas border in the USA. The basins are required as containment in case of accidental spill during plant operation. The site is underlain by deep deposit of very soft to stiff clay over a layer of dense sand. The basins are 60 feet in diameter and 32 feet deep below the existing grade. Several options including braced-sheet pile wall, precast tubular caisson type structure and secant pile wall consisting of interconnected drilled shafts were considered for the excavation support. Due to schedule, cost and availability of equipment and material, the seant pile wall system was selected.

SITE CONDITIONS & SOIL PROPERTIES

Surface Condition

The natural ground condition was too weak to support the heavy construction equipment. Drainage was poor resulting in prolonged standing water following rainfall. To improve subgrade supporting capacity, the upper few feet of soft soils were stabilized in-situ with fly ash at the beginning of the construction.

Geology

The project is situated on the outcrop of Holocene age Chenier plain and coastal marsh sediments. The geologic stratigraphy is influenced by the Neches, Sabine, and Mississippi River systems. The Chenier and coastal marsh sediments were deposited in the period between present day to 5,000 years ago. The Holocene consists of recent sediments deposited in present day alluvial valleys, coastlines, marshes, and floodplains of the major area drainage systems. The sedimentary units typically contain cohesionless soils (sands, silts, and their intermixtures) intermixed with cohesive soils (clays, sandy clays, and silty clays). The site is situated on the youngest plain, the Holocene plain that was deposited in the past 5,000 years. The sediments present at about El. –57 to El. –76 ft are late Pleistocene Deweyville Terrace Deposits which were deposited 35,000 to 40,000 years ago. The Beaumont Formation underlies the Deweyville and was deposited about 40,000 to 80,000 years ago. Dredge materials cover the natural terrain over most of the site.

Subsurface Stratigraphy

The subsurface conditions at the LNG impoundment basins were explored by two piezocones, CPT-1 and CPT-2, at the center of each basin. Additional geotechnical data were available from comprehensive geotechnical investigations conducted for the nearby LNG tanks and dikes, process and marine berth areas of the terminal. In general, the subsurface conditions at both basin locations consist of very soft to firm clays to depths about 73 and 78 ft underlain by medium dense to very dense sands followed by stiff to very stiff clays. The cone penetration resistance profiles and generalized subsurface conditions are shown on Fig. 1. Ground water is typically encountered at a depth of about 6 feet below the ground surface.

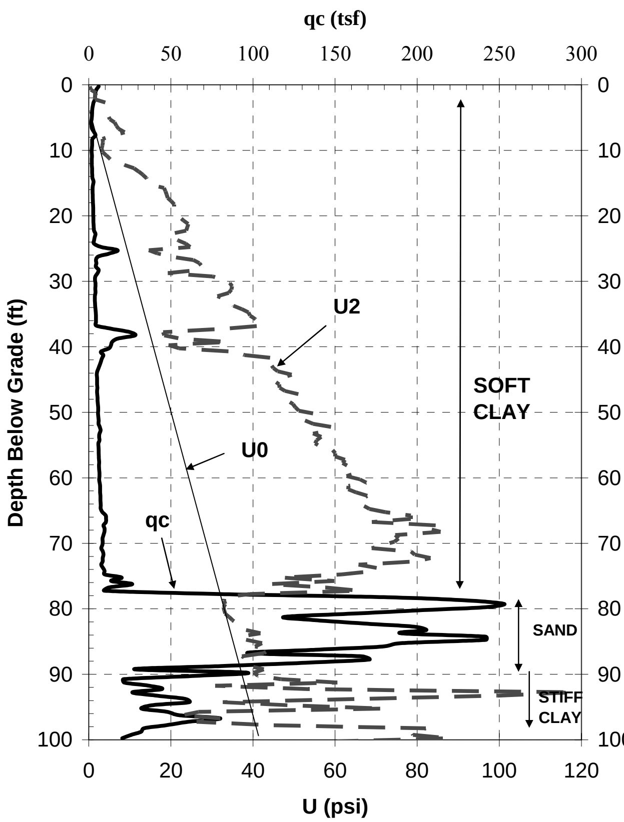


Fig. 1. Measured cone resistance, q_c, pore pressure, u2 and interpreted soil stratigraphy (CPT-2).

Soil Properties

The soil properties were estimated from the interpreted CPT data as well as supplemental soil data from the extensive geotechnical investigation studies conducted for other terminal facilities which included in-situ vane shear tests, laboratory triaxial, consolidation and index tests. Professor Stephen Wright, as a consultant to the owner, conducted an independent slope stability analysis for the marine berth excavation project. Based on the extensive evaluation of the available geotechnical data, he recommended undrained shear strength profile for the soft clays near the berth / jetty area; this profile, termed here as 'Wright's Su Profile' was adopted with some modification in our analysis. The undrained shear strength from CPT data was estimated using a N_k value of 16 (where $Su= q_c/N_k$). Based on the interpreted Su from the CPT data, it appears that the clays to a depth of about 60 ft are relatively weak at CPT-1 (process basin area inland) as compared to CPT-2 (near the jetty area). This is not surprising because CPT-2 was very close to the Jetty area for which Wright's Su profile was developed. In order to incorporate the weaker soils at the shallow depth at CPT-1, the Su at shallow depths was reduced to 50 to 150 psf. The interpreted Su profiles from the CPT data along with Wright's Profile are shown on Fig. 2.

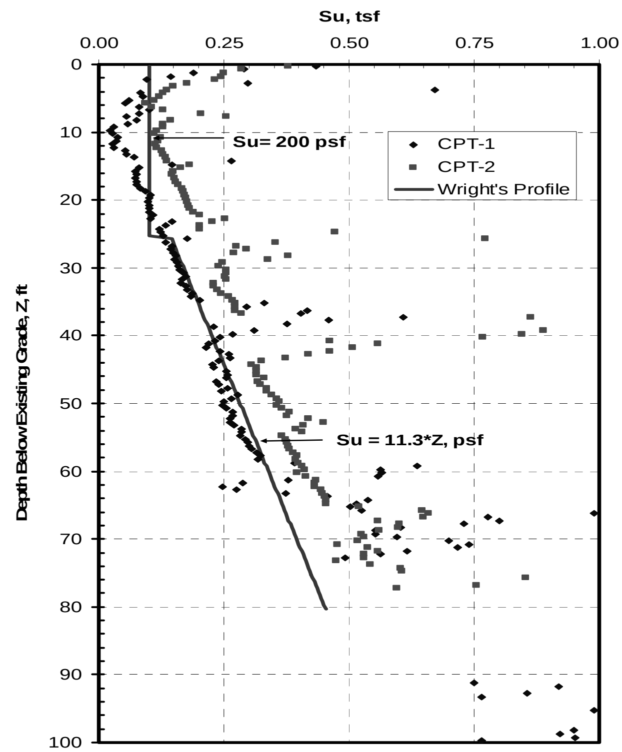


Fig. 2. CPT interpreted and Wright's Su profiles.

The effective strength parameters of 30 degrees and zero cohesion were selected for the clay deposit based on several CIU triaxial tests. A number of consolidation tests were performed as a part of the geotechnical studies for the LNG tanks and dikes; these test results and typical index properties of the soft clays are summarized in Table 1.

Table 1. Typical Index and Average Consolidation Properties

w (%)	LL (%)	PI (%)	OCR	Cc	Cr	e_0	Cv (NC) (ft^2/day)
84	107	79	1.0	0.92	0.15	2.3	0.02

DESIGN CONSIDERATIONS

The basin design was required to satisfy the following loading and stability conditions.

1. ground water table at the surface due to potential for water accumulation
2. short-term SF =1.3 under undrained condition
3. Long term SF = 1.5 using drained, long-term strength parameters.
4. The mud mat at the bottom of the excavation should have only limited deflection to not impact the construction the structural mat at the bottom

DESIGN ANALYSIS

Methodology

The design of the lateral earth support system for excavating in soft to firm clays is often controlled by stability requirements. If the factor of safety is below an acceptable level against base instability, large soil movement could occur resulting in a catastrophic system failure. There are two methods to perform stability calculations for excavations: (1) limit equilibrium methods; and (2) nonlinear finite-element methods. Over the years, limit equilibrium methods have been widely used in design practice which includes separate calculations of basal stability based on failure mechanism proposed by Terzaghi 1943; Bjerrum and Eide, 1956 or overall slope stability using circular or non-circular mechanisms (Bishop 1955, Morgenstern and Price 1965). However, the limitations of these methods are assumptions in selecting the shape of failure surfaces, search procedures to locate the critical surface and inability to consider deformation effect on stability. Non-linear finite element (FE) methods, such as c-phi reduction method, overcome these limitations in evaluating multiple facets of excavation performance ranging from the design of the wall and support system, to the prediction of ground and support movements, and effects of construction activities such as dewatering, equipment surcharge, staged construction on deformation and overall stability. The FE methods can also incorporate shear strength variation with depth in evaluating stability of the excavation and base heave.

Considering the limited published references on the evaluation of stability of circular retention system using limit equilibrium and advantages of the FE methods, it was decided that FE method will be used for deformation and stability analyses, and the stability results will be verified by the limit equilibrium procedure suggested by Bjerrum and Eide, 1956 (Equation 1) which can be used for circular excavation and incorporate the embedment and rigidity of the retention system as follows.

$$FOS = N_c'{*}S_{ub} / \gamma_t{*}H \qquad (1)$$

Where, FOS= base stability factor of safety, γ_t = total unit weight of soil above excavation base, H= height of excavation, S_{ub}= average shear strength at (H+B/12) below excavation base, Nc' = base bearing capacity factor with wall embedment = $N_c + 2{*}\alpha{*}(D/B)$, N_c= base bearing capacity factor without wall embedment, D=wall embedment depth, B=width/diameter of the excavation, α = adhesion factor between wall and retained soil = 1.0 for rigid wall.

Finite Element Model

The FE code PLAXIS V8 was used in the study. PLAXIS, developed particularly for geotechnical application, incorporates multiple simple and advanced soil models. An axis-symmetric model of half of the retention system was created using 15-node triangular elements. Concrete secant pile wall was represented by plate elements, however due to consideration of half of the basin and axis-symmetry generation of the stiffness matrix, it represents a circular shell element according to the PLAXIS developers. As a result of weaker undrained shear strength profile encountered at the process basin (CPT-1), it was decided to analyze the process basin as more critical of the two identical basins. The subsurface stratigraphy was modeled as 73-ft deep very soft to firm clays followed by a sand layer to 90-ft depth and underlain by very stiff clays. The advanced Hardening Soil (HS) model was selected for soft clays and stiff clays to account for non-linear stress-strain behavior. Mohr-Coulomb (MC) model was selected for sands, base slab, and mud mat. The reinforced concrete secant pile wall was modeled as linear elastic Plate elements.

Per project specifications and the proposed design, the following assumptions were made in developing the FE model.

- Surrounding final grade elevation El 106 ft
- Long-term groundwater elevation El 100 ft
- Groundwater elevation El 106 ft during construction
- Top of wall elevation 106 ft
- Depth of excavation = 32 ft (El 74 ft)
- Top of mud mat elevation 76 ft (2 ft thick)
- Top of base slab elevation 80 ft (4 ft thick)
- Groundwater is at the top of mud mat during base slab construction
- Equipment surcharge of 200 psf is located 3 ft away from the wall top during wall and mud mat construction. No surcharge is assumed at El 106 ft after mud mat construction.
- The upper 5-ft of soft soils are stabilized (S_u= 1500 psf)

A schematic of the excavation is shown in Fig. 3.

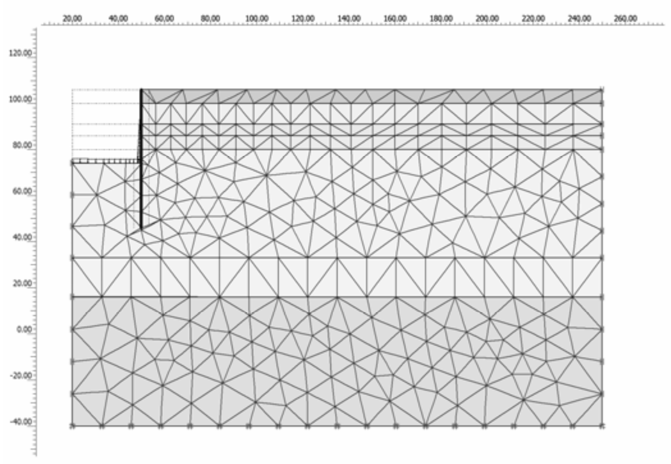


Fig. 3. PLAXIS Model of the excavation.

FE Input Parameters

The input parameters were selected from CPT-1 data, consolidation test and other relevant soil data from previous geotechnical studies performed by TWEI and Dr. Stephen Wright's study. The LNG dikes near the process basin were instrumented with settlement plates, inclinometers and piezometers; the back-calculated soil properties from the field observations were also considered.

Shear Strength. Based on the estimated construction time of about 4 to 6 weeks, it was assumed that any potential failure mechanism would be controlled by undrained strength of the shallow clay. From Fig. 2, it can be seen that the interpreted Su profile at the process basin (CPT-1) appears to be weaker than the jetty basin; the selected design Su profile for the process basin is shown in Table 2.

Table 2. Design Soft Clay Su Profile at Process Basin

Stratum No.	Depth Range (ft)	Elevation (ft)	Undrained Shear Strength, Su (psf)
I	0 - 15	106 - 91	50
II	15 - 20	91 - 86	150
III	20 - 26	86 - 80	200
IV	Below 26	Below 80	11.3*Z

(Z is depth in ft below ground surface)

An effective friction angle, φ' of 35° for the underlying sands and undrained shear strength of 2,000 psf for the bottom stiff clays were selected. The mud mat was modeled as hard clays with undrained shear strength of 4,000 psf. For the long-term deformation and stability analysis, the effective stress shear strength properties of the soft clays were selected as c'= 0, φ' = 30°.

Stiffness Properties. The construction of impoundment basin involves excavation (unloading condition), as such soil behavior and hence the base heave magnitude will primarily be governed by its unloading modulus. The effective (drained) stiffness properties, modulus and Poisson's ratio, were used in undrained and drained analysis. In HS model, three types of soil moduli are required as input while in MC model only one modulus is required; they are discussed below.

The clays were modeled using Hardening-Soil (power law stress-strain) model. The required referenced soil moduli are E_{50}^{ref} (elastic modulus), E_{oed}^{ref} (oedometer loading modulus), and E_{ur}^{ref} (unloading modulus). A parametric study using finite element method was performed to match the observed settlements at the test dike using HS soil model for soft clays and to back-calculate the average E_{50}^{ref} and E_{oed}^{ref} for the soft clays. Plots of measured and estimated settlements at the settlement plate SP-7 from the test dike since the start of construction is shown in Fig. 4, which shows reasonable agreement between the measured settlement of 6.0 ft and the estimated end-of-primary settlement of 5.8 ft. (Suroor, 2007) The average E_{50}^{ref} and E_{oed}^{ref} used in the finite analyses for the upper 73 ft of soft clays were 8,500 psf and 12,500 psf, respectively. The back-calculated E_{oed}^{ref} agrees closely with the estimated E_{oed}^{ref} from the CPT and consolidation test data. A reference stress level, p^{ref} = 2,000 psf was used in PLAXIS.

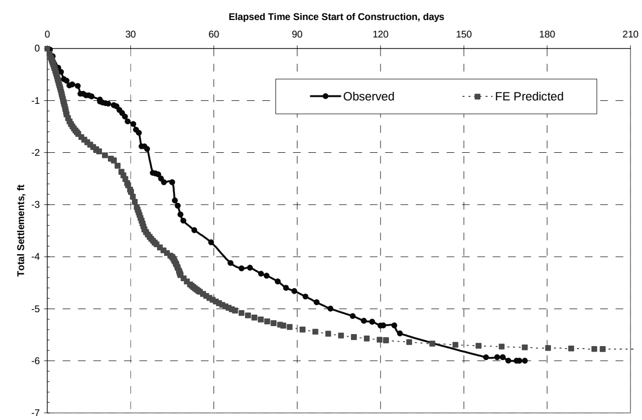


Fig. 4. Observed and FE predicted settlements since start of LNG dike construction.

Published references (Whittle, 2005, Vermeer et al, 2002) suggest that typically clay E_{ur}^{ref} is 5 to 10 times E_{oed}^{ref} ; however, small strain unloading modulus could be much higher. The unloading modulus E_{ur}^{ref} was conservatively selected as 9 times E_{oed}^{ref}.

Typical soil Poisson's ratios for unloading conditions are 0.1-0.2 (Vermeer, et al. 2002). A drained (effective) unloading Poisson's ratio, υ'_{ur} of 0.2 was used in the HS models for soft and stiff clays. As suggested by PLAXIS, the power, m =1.0 was used for the hyperbolic (HS) soft soil model.

The sand, lean concrete mud mat, and concrete base slab was modeled using MC (elasto-plastic) model. The lean concrete mud mat and concrete base slab were modeled conservatively as hard clays to reduce their effect (high stiffness) on stability and deformation analyses; the referenced modulus E_{50}^{ref} was selected corresponding to unloading behavior of hard clays. The Poisson's ratio, υ' of 0.2 was used in MC sand model corresponding to unloading condition.

<u>Initial Earth Pressure Coefficient, Ko</u>. In PLAXIS, the ratio between effective horizontal and vertical earth pressure, K_o is required for the initial (original/equilibrium) stress computation. PLAXIS, by default, computes K_o = 1-sinφ' according to the well known Jaky's formula based on the input effective shear strength parameter, φ'. A K_o=0.5 (φ'= 30º) was used for the normally consolidated soft clays.

<u>Wall Properties</u>. The nominal diameter of the reinforced concrete secant pile is 3-ft; each pile will overlap by 6-in into the adjacent piles resulting in an effective diameter of about 2-ft. The secant pile circular wall is expected to be in compression and experience negligible flexural forces (moment and shear); it was assumed that the concrete will not crack. The secant pile wall was modeled as a circular elasto-plastic plate; the axial stiffness, EA and flexural stiffness, EI stiffness per foot of wall were computed for a plain concrete (fc' = 4,000 psi) section of 2-ft by 1-ft (per foot of wall perimeter).

The required PLAXIS input parameters for soils, mud mat, base slab and circular secant pile wall are summarized in Table 3.

<u>Calculation Stages</u>

The undrained deformation (plastic calculation) analyses were performed at each of the five construction stages shown below.

I. Installation of perimeter reinforced concrete secant piles.
II. Excavation to a depth of about 32 ft below grade.
III. Construction of mud mat.
IV. Construction of reinforced concrete base slab tied to the secant pile wall.

The stability (using c-phi reduction method) analysis was performed after Stage II, deemed critical construction stages. Additionally, the long-term deformation and stability analyses were also performed using effective stress parameters.

Table 3. Pertinent PLAXIS Input Parameters

ID	Name	Model	Type	g_unsat [klb/ft^3]	g_sat [klb/ft^3]	E50ref [klb/ft^2]	Eoedref [klb/ft^2]	Eurref [klb/ft^2]	c_ref [klb/ft^2]	phi [°]	n_ur [-]
1	Clay-1	HS	UnDrained	0.1	0.1	8.5	12.5	75	0.05	0	0.2
2	Clay-2	HS	UnDrained	0.1	0.1	8.5	12.5	75	0.15	0	0.2
3	Clay-3	HS	UnDrained	0.1	0.1	8.5	12.5	75	0.2	0	0.2
4	Clay-4	HS	UnDrained	0.1	0.1	8.5	12.5	75	0.295	0	0.2
6	Clay-5	HS	UnDrained	0.117	0.117	100	160	1600	2	0	0.2
5	Sand	MC	Drained	0.127	0.127	4.0			0.00	35	0.2
7	Mud Mat	MC	Non-porous	0.14	0.14	4.0			4.00	0	0.2
8	Base Mat	MC	Non-porous	0.15	0.15	450.0			10.00	0	0.15

RESULTS

<u>Base Heave & Stability</u>

During excavation into saturated clays, the weight of the blocks of clay behind the retention system tends to displace the underlying clays towards the excavation. Base stability for excavation in soft clays depends on height of excavation, wall embedment depth, undrained shear strength of clays below the base of excavation, and unit weight of soil. The stability of the excavation support system is dependent on the magnitude of heave.

The undrained deformation (plastic) analysis was performed to evaluate base heave and corresponding wall movements during each of the construction stages mentioned above. The results are shown in Fig. 5. Figure 5 shows that a total of about 5-in of base heave is expected after excavating to El 74 ft (H=32-ft) and 3-in base slab construction (H=26-ft). The corresponding horizontal wall movements are negligible as a result of rigidity of the circular retention system.

The basic concept of finite element stability analysis using c-phi reduction method is to reduce the input shear strength parameters by a factor, which is identified as Total Multiplier, ∑Msf in PLAXIS ($\sum Msf = \tan\varphi_{input}/\tan\varphi_{reduced} = c_{input}/c_{reduced}$), which increases gradually until the base of excavation reaches failure. In FE stability analyses, failure is defined by excessively large deformation occurring progressively at a constant ∑Msf, which is the factor of safety (FOS = available strength / strength at failure = value of ∑Msf at failure) against base instability. For practical reasons, it was assumed that a base heave of more than ½-ft is excessive (equal to 25% strain of the 2-ft thick mud mat). The computed base stability FOS after construction Stage II is about 1.35 at a base heave of 0.5-ft (i.e. 25% strain) as shown on Fig. 6; the ultimate FOS at failure is about 2.1.

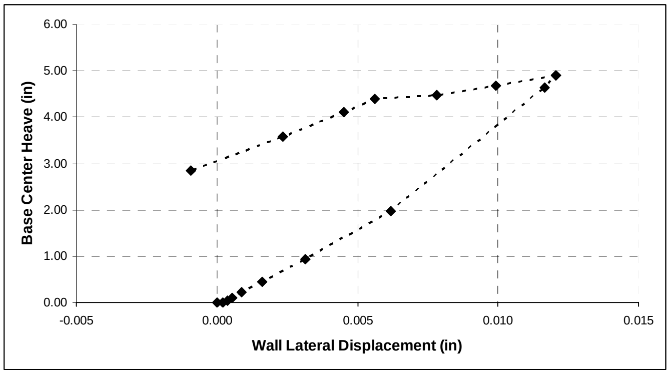


Fig. 5. Predicted base center heave and secant pile wall top lateral movement.

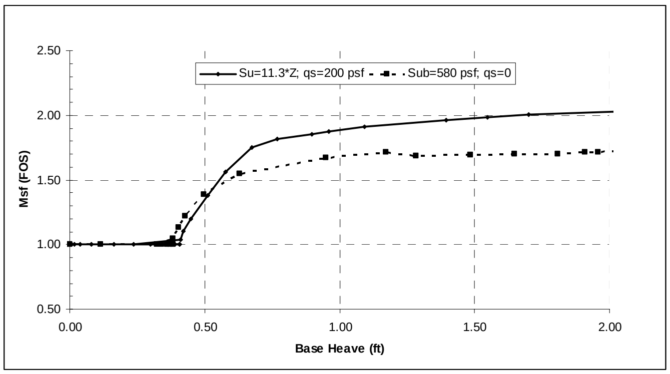


Fig. 6. Predicted base stability FOS for linearly increasing Su and average Su.

Published references on excavation stability do not explicitly account for linearly increasing shear strength with depth. To compare PLAXIS results with FOS computed by published limit equilibrium and FE methods, an average undrained shear of 580 psf (average shear strength between the base of excavation (Z=H and Z=H+B/√2 depth) using S_u= 11.3*Z was selected; surcharge was not considered. The computed FOS by PLAXIS is 1.40 at 0.5-ft of heave (Fig. 6); the ultimate FOS at very large deformation is about 1.9. Using Bjerrum, et al (1956) limit equilibrium approach, the computed factor of safety is 1.45. Cai, et. al (2002) developed finite element based design charts for circular excavation in soft clays having constant undrained shear strength, S_u and supported by elastic concrete diaphragm wall; using these design charts, the computed FOS is about 2.7. It appears that the result of the conventional limit equilibrium method which accounts for support rigidity agrees well with the FE results at reasonable base heave (20% strain).

The computed FOS at the end of construction (after Stage IV) is about 4.0. The long-term stability was checked using effective stress shear strength parameters; the computed FOS is on the order of 9. The soil movement at failure (plastic flow) is shown in Fig. 7. The influence zone at failure as shown in Fig. 7 is similar to assumptions made in conventional stability analysis.

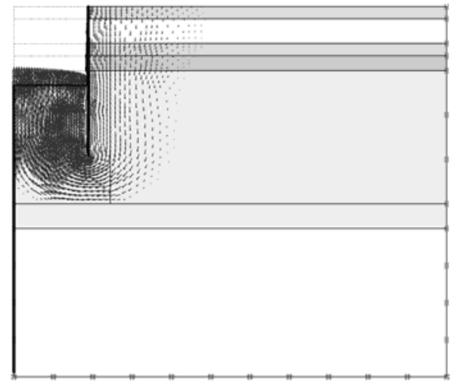


Fig. 7. Plastic soil flow at failure.

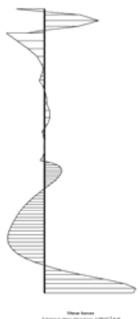
Wall Movements and Forces

A circular wall effectively reduces ground and wall movements as a result of inherent self-braced system. In theory, the perfectly circular wall should not experience any shear and bending moments, and should only experience compressive stresses from hoop and axial forces.

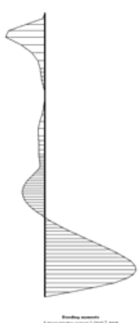
The deformation and forces in the circular wall were evaluated during each construction stage. The most critical stages appear to be after Stage II during construction and in long term. For Stage II construction stage, the vertical and horizontal wall movements as well as axial, shear, bending moments and hoop forces generated per linear foot of wall are shown in Fig. 8; the long-term forces are shown in Figure 10. The results show negligible shear and bending movements in the wall indicating that the circular wall, as expected, is essentially in compression from axial and hoop forces.

The lateral wall movements are also negligible (less than ¼-in) indicating a rigid support system (Fig. 5). As a result of non-yielding support system, the lateral earth pressures on behind the wall and inside the excavation are similar to initial at-rest (K_o condition) earth pressures as shown in Fig. 10. Figure 10 also shows net lateral earth pressure (difference between behind the wall and inside the excavation earth pressures) acting on the wall, which causes hoop forces on the wall. As a result of negligible net earth pressures between about 32 and 50 ft depths, the hoop forces on the wall remain nearly constant, as shown in Fig. 11. The maximum hoop force is generated at the tip of the wall as a result of large lateral stress generated by soil moving towards the excavation.

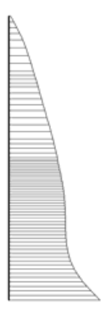
(a) Axial Force



(b) Shear Force



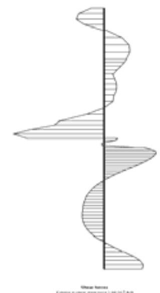
(c) Bending Moment



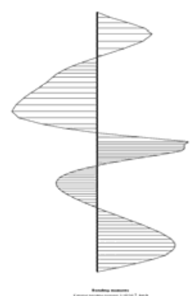
(d) Hoop Force

Fig. 8. Predicted wall forces after construction stage II.

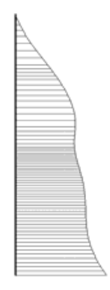
(a) Axial Force



(b) Shear Force



(c) Bending Moment



(d) Hoop Force

Fig. 9. Predicted wall forces in long term condition.

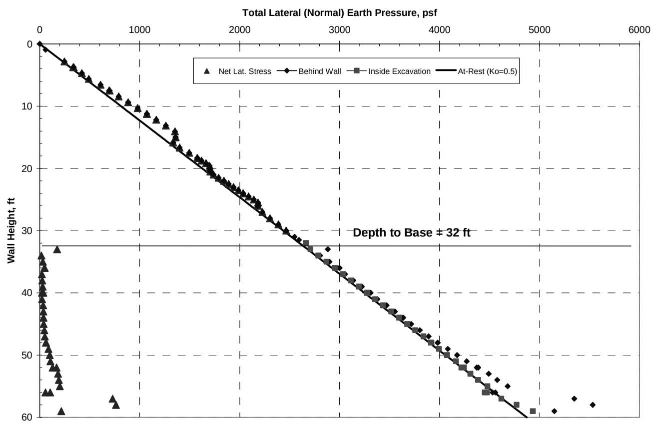


Fig. 10. Predicted lateral earth pressures on the secant pile wall after stage II.

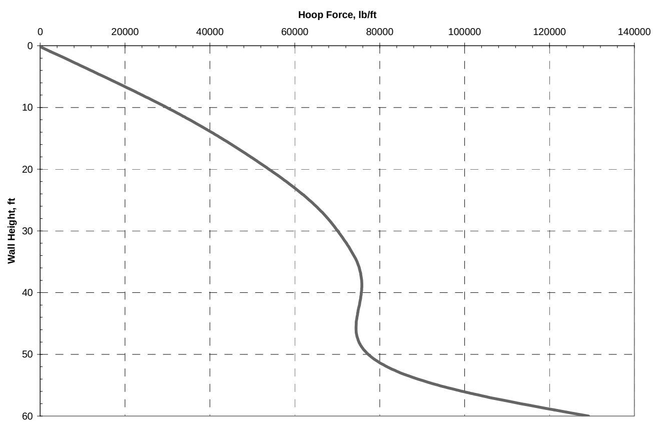


Fig. 11. Predicted hoop forces on the secant pile wall.

Hydrostatic Uplift

As per design specifications, the hydrostatic uplift forces should be computed assuming groundwater level at grade (El 106 ft). Following mud mat construction and slurry removal, the estimated net hydrostatic uplift pressures under the mud mat is about 1,700 psf (32x62.4 – 2x140). It is expected that the mud mat will crack as a result of the unbalanced uplift pressures and base heave following slurry removal allowing groundwater seepage through the mud mat. The base slab will experience permanent hydrostatic uplift forces as a result of the piezometric head of about 30 ft (between El 76 and 106 ft). The estimated net hydrostatic uplift against the base slab is about 1,200 psf (30x62.4 - 4.3x150). The structural design of the base slab considered this net hydrostatic uplift pressure; the base slab was adequately reinforced to transfer this uplift force to the perimeter secant piles, which have sufficient uplift capacity. The computed short and long term factor of safety against hydrostatic uplift forces are 2.8 and 4.1, respectively.

CONSTRUCTION

Site Preparation

A stable work platform was needed to facilitate access and egress of the drilling equipment and the concrete material delivery for the installation of the secant pile wall. The platform footprint chosen was a 120 ft diameter, elongated semicircle, providing a minimum 30 ft working surface around the perimeter of the basin to be constructed. The upper soil mass was stabilized with fly ash to achieve a 20 psi strength soil, to a depth of 8-9 ft below existing grade. Once stabilized, crane mats were placed on top of the platform, covering the entire surface of the inside diameter of the designated secant pile wall area. Laminated mats were placed along the outer perimeter of the wall area during construction to provide a slightly elevated work area which was able to be rinsed daily of mud and concrete debris to help prevent slip and trip hazards as well as provide a clean area to place small tools and equipment.

Construction Equipment & Surcharge

A Delmag RH-32, overhead drive hydraulic drill rig was chosen to install the secant piles. The fully assembled drill rig weighs approximately 250,000 lbs with a torque capacity of 230,000 foot-pounds. The RH-32 is capable of drilling to depths of 180 feet, at diameter widths up to 10 feet. Support equipment included an American 999 crane. The actual configuration of the crane used onsite weighed 170,000 lbs with a lifting capacity of 120 tons. Other support equipment used in the construction of the wall included an excavator, dozer, forklift, and man lift.

Secant Pile Wall Construction

Guide Wall. Prior to drilled shaft construction, 2-foot thick concrete guide walls were constructed using circular metal forms. These walls ensured accurate positioning of each individual unit of casing, while helping to keep them plumb and level during installation. The guide was installed with a finish grade matching the top of concrete of the completed secant piles.

Casing Installation. The following steps outline a typical shaft casing installation. After determining the correct shaft location on the guide wall, the casing is connected to the casing "driver". The casing is twisted into the appropriate hole formed in the guide wall. The casing is checked with a 4 ft. level to ensure that it is plumb upon initial penetration, and every five feet as it is advanced vertically.

The casing is advanced with the hydraulic drill by rotating and pushing or crowding the casing into place with the drill rotary table, the casing is rotated and counter rotated during the process to reduce sidewall friction. The casing is advanced until the penetration rate slows, refusal is achieved, or the bottom of the column is reached. At this time, excavation is performed with an auger attached to the Kelly bar of the drill rig. Excavation is stopped at a depth of 2 feet from the bottom of the casing. At this time, additional casing is added in sections as required, while excavating soil from the casing. Once the water table is reached, the casing is filled with water above the table to maintain a greater head pressure. The driving of casing and the retrieval of soil within are repeated until design elevations are reached.

Concrete Placement. The bottom of the casing is cleaned with a muck bucket and inspected by dropping a weighted tape to the bottom of the hole and sounding the bottom. A 10-inch diameter tremie pipe is placed into the casing, with the bottom pipe within two feet of the bottom of the excavation. A pig is placed in the tremie pipe to separate the concrete to be poured from the water used to equalize head pressure within the casing. A tremie pipe is utilized to ensure the 4000 psi pea gravel concrete mix is uniformly placed at the base of the shaft and continually poured through concrete forcing the soil and water within the casing to remain on top of the pour. The soil and water within the column are eventually forced to the surface leaving the shaft free of contaminates. Retarders and water reducers were added to slow the concrete set time and to maintain an 8-inch slump during the pour. These additives were needed to keep the concrete in a flowing state as well as to retard the set time to allow reinforcement to be placed after the pour. Sections of the tremie pipe are removed as the concrete placement rate reduces. As each section of casing is over poured, the upper casing sections are removed one at a time. This cycle is continually performed until the shaft is full of concrete and the last section of casing is removed.

The concrete is over poured to insure that sediment is displaced and that good concrete is present at the surface. After the concrete is poured to grade, a temporary metal template is placed on the corresponding guide wall “circle” to facilitate accurate surface positioning and guidance of the wide flange beam reinforcement during installation. A W12X58 wide flange beam was part of the structural design of the secant pile wall. The beams were installed with the service crane by suspending it over the center of the template at the surface. Once the beam is over the template, it is lowered into the wet concrete (under its own weight) until refusal. The beam is then driven to the final tip elevation with a small vibratory hammer. The beam is checked for plumbness with a level, initially and every five feet during placement.

Excavation

Before the basin floor could be installed, a mud mat had to be placed in order to safely allow access of personnel. The depth of excavation was 32 ft below ground surface. Approximately the top twenty feet of the excavation was excavated in the dry. The interior of the basin was then flooded to prevent heave of the floor soil as the remaining twelve feet was excavated. Inclinometers and settlement plates were installed around the outside perimeter of the wall before excavation and measurements were recorded to establish a baseline. These instruments were monitored continuously until the basin floor was installed. Two long reach excavators, weighing 60,000 lbs each, were positioned on mats at opposite sides of the basin each performing 6-inch cuts along the surface in their designated areas. The excavation was performed at an intentionally slow pace. 2025 cubic yards were removed the first day, representing removal of 12 feet. Final depth was reached the next day by excavating the last 1400 cubic yards giving a final depth elevation of 32 feet below ground surface. Once the base was leveled and made ready for the mud mat, a 6 inch pipe was installed near the sump side of the basin. This would act as a conduit for ground water to relieve the buildup of hydrostatic pressure on the mud mat.

Mud Mat Construction

A 3000 psi lean concrete mix was chosen as the mud mat to resist the hydrostatic uplift and to provide a working surface for construction of the base mat. It was placed with a concrete pump through the water onto and around the base of excavation achieving a final thickness of a minimum of 2 feet. After concreting, the water was pumped from the basin. In order to maintain a dry working condition through the remainder of construction, ground water seeping through the relief pipe, was controlled by a small pump.

Base Mat Construction

The structural design dictated a 4 foot-6 inch thick concrete floor was adequate to resist the uplift pressure in an empty state. This involved over 35 tons of steel reinforcement covered with over 500 cubic yards of 4000 psi concrete. A hydraulic hammer was used to create a 9 inch keyway into the secant piles around the inside perimeter, under the finish grade of the basin floor. Reinforcement included two mats of closely centered #10 rebar. Concrete was poured by pneumatically pumping into place.

Construction Difficulties

Installing a secant pile wall in a soft soil environment had its challenges. Placing concrete in near fluid state soil prevented the installation of casing in multiple locations before concrete placement. In tighter environments, it is typical to have 4 or more locations ready to pour. That is, to drive casing and excavate out each hole, then place the concrete in all locations afterwards. This process was not appropriate for this project due to the zero strength soil conditions. If two or more strings of casing were installed, without a concrete shaft in between the adjacent open holes, concrete would migrate through the soils to the nearest open casing during the second half of the pour. As a consequence, only one shaft could be excavated and poured at a time, unless there were completed shafts between the newly excavated shafts, which provided enough resistance to cut off any avenues of concrete flow from casing to casing.

SUMMARY & CONCLUSIONS

It was demonstrated with sufficient confidence that the FE method can be used effectively to predict wall stresses, deformation and the base stability by comparing the FE results with the published solutions using limit equilibrium method. Simple check analysis was also used to verify hoop stresses in the wall. Performance of the wall system will be monitored using inclinometers installed next to the wall during the excavation stages. Such data will provide valuable input for practicing geotechnical engineers.

ACKNOWLEDGMENTS

The authors wish to extend their gratitude to the project owner Cheniere Energy for allowing to use the data. The support and encouragement provided by Tolunay-Wong Engineers, Inc., Bechtel Corporation Houston office, and Remedial Construction Services are appreciated.

REFERENCES

Bishop, A. W. [1955]. "*The Use of Slip Circle in the Stability Analysis of Slopes*". Geotechnique 5.

Bjerrum, L. and Eide, O. [1956]. "*Stability of Strutted Excavation in Clay*". Geotechnique 6.

Cai, F, Ugai, Keizo and Hagiwara, T. [2002]. "*Base Stability of Circular Excavation in Soft Clay*". Journal of Geotechnical and Geoenvironmental Engineering, vol. 128, No. 8.

Morgenstern, N.R, and Price V.E. [1965]. "*The Analysis of the Stability of General Slip Surfaces*". Geotechnique 15, No. 1.

PLAXIS 2D-Version 8 [2006]. "*PLAXIS Manual*". PLAXIS bv, Delft, Netherlands.

Suroor, Hadi, [2007], "Performance Evaluation of Instrumented LNG Retention Dikes on Louisiana Soft Clays'. 7th International Symposium on Field Measurements in Geomechanics, Boston, USA.

Terzaghi, K [1943]. "*Theoretical Soil Mechanics*". Wiley, New York.

Vermeer, P.A., Marcher, T., and Ruse, N. [2002]. "*On the Ground Response Curve*". FELSBAU 20, No. 6.

Whittle, A.J. [2005]. "*Computational Geotechnics (Plaxis)*". Short course handout, University of Houston, Texas.