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UNIVERSITY OF CALIFORNIA,
IRVINE

Numerical Modeling of an Exploratory Deep Excavation in Los Angeles

THESIS

submitted in partial satisfaction of the requirements
for the degree of

MASTER OF SCIENCE

in Civil and Environmental Engineering

by

Andres Ricardo Lozano

Thesis Committee:
Associate Professor Anne Lemnitzer, Chair
Associate Professor Joel Lanning
Professor Mo Li

2025

DEDICATION

To all my family, friends, colleagues, and coworkers who provided me with an abundance of guidance, understanding, and patience. I thank you all dearly for your support during my academic journey and for nourishing me with the motivation and perseverance necessary to see this journey through to the end. A special thank you to my mother, Patricia Lozano, who has always served as my driving force to pursue my goals and push beyond what I thought I could achieve. To my father, Jose Lozano Sr., for teaching me the meaning of dedication and hard work, and more importantly how these characteristics can be utilized in everyday life to help provide to those around you. To my older brother and little sister, Jose Lozano Jr. and Nayeli Lozano, for always having my back, providing your warming embrace, your constant support, and overall being amazing people who continue to inspire me. Lastly, a thank you to my partner, Gabriela Velasco, for being with me through every step of this journey and supporting my endeavors endlessly. All of your support has helped more than you'll ever know.

“I hear and I forget, I see and I remember, I do and I understand” – Confucius

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ABSTRACT OF THE THESIS

Numerical Modeling of an Exploratory Deep Excavation in Los Angeles

by

Andres Ricard Lozano

Master of Science in Civil and Environmental Engineering

University of California, Irvine, 2025

Professor Anne Lemnitzer, Chair

This thesis investigates the performance of a deep exploratory shaft in Los Angeles, part of Metro's Purple Line Extension project, through extensive numerical modeling. Using the finite element program PLAXIS, both two-dimensional (2D) and three-dimensional (3D) analyses were conducted with four soil constitutive models: Linear Elastic (LE), Mohr-Coulomb (MC), Hardening Soil (HS), and Hardening Soil with small-strain stiffness (HS_{small}). The modeling results were calibrated and compared against field measurements from inclinometers and settlement markers to evaluate accuracy and predictive capability. The work provides important insights into soil-structure interaction and modeling limitations for deep excavations in stiff clayey and tar-infused soils.

INTRODUCTION AND CASE HISTORY

Background and Motivation

Developing underground space for transportation, commercial, or industrial purposes has become increasingly important in densely occupied urban areas where infrastructure is pivotal to daily operations and logistics. It requires the construction of deep excavations and tunnels to create rail, auto, and pedestrian passage. The 2016 approval of Metro's Measure M (<https://www.metro.net/about/measure-m/>) marked a turning point in LA's transportation system; it actively works towards transforming the city's public transit by providing alternative transportation options to relieve congested roadways and to provide significant environmental and economic benefits.

Los Angeles Metro Rail is regarded as the largest and most complex underground transportation network in Southern California. It consists of six lines, including two subway ([heavy rail](#) and [rapid transit](#)) lines (the [B](#) and [D](#) lines) and four [light rail](#) lines (the [A](#), [C](#), [L](#) and [E](#) lines) serving [93 stations](#). It connects with the [Metro Busway](#) bus rapid transit system (the [G](#) and [J](#) lines) and also with the [Metrolink](#) commuter rail system. Several major expansions of the Los Angeles Metro systems are currently underway, i.e., the Crenshaw/LAX Transit, the Regional Connector, and the Purple Line Extension. The Purple Line Extension, formerly known as the Westside Subway Extension, and recently renamed to D-Line, is a new, critically important rail subway corridor in Los Angeles, extending Metro's Purple Line from its current terminus in Koreatown, to LA's westside region (Westwood). Figure 1 depicts the nine-mile, seven-station extension, which is currently under construction and built in three segments. The first section (Section 1; 3.92 miles) between Wilshire/Western and Wilshire/La Cienega was scheduled for completion in 2023.

Section 2 (2.59 miles) is currently under construction and scheduled for completion in 2025. Section 3 will add the last 2.56 miles of new rail to the Purple Line and will open for operations in 2027.

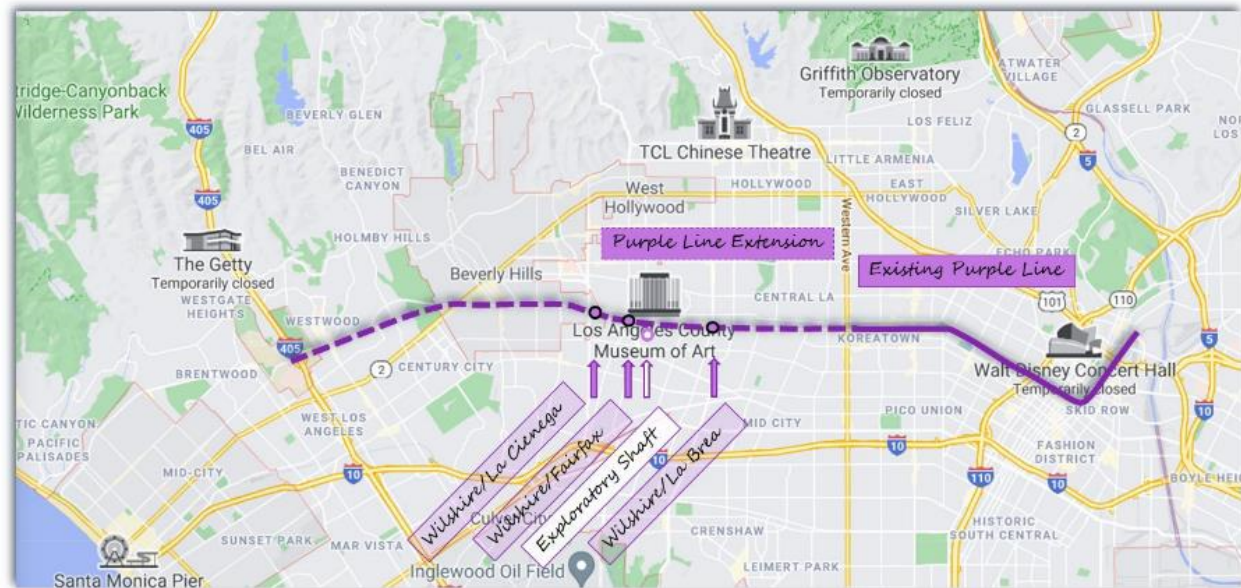


Figure 1: Map of purple line extension section 1 and location of Exploratory Shaft

More interestingly, LA Metro's purple line Section 1 is situated in geologic units which are naturally impacted with bitumen, locally referred to as tar sands. Immediately adjacent to Los Angeles' iconic La Brea tar pits, the Wilshire/Fairfax station as well as the Wilshire/La Brea station tunnels and excavations (see Figure 1) encounter numerous strata of heavily tar-infused fine-grained sands, stiff to hard silts, and weak siltstone and claystone layers. To meet design objectives and better understand excavation performance behavior in tar impacted formations, a small-size Exploratory Shaft was constructed prior to- and in the same manner as- its future large-scale station excavation counterparts. This MS thesis analyzes the response behavior of the exploratory shaft

constructed in Los Angeles near the corner of Wilshire Blvd and South Ogden Drive (see Figure 2).

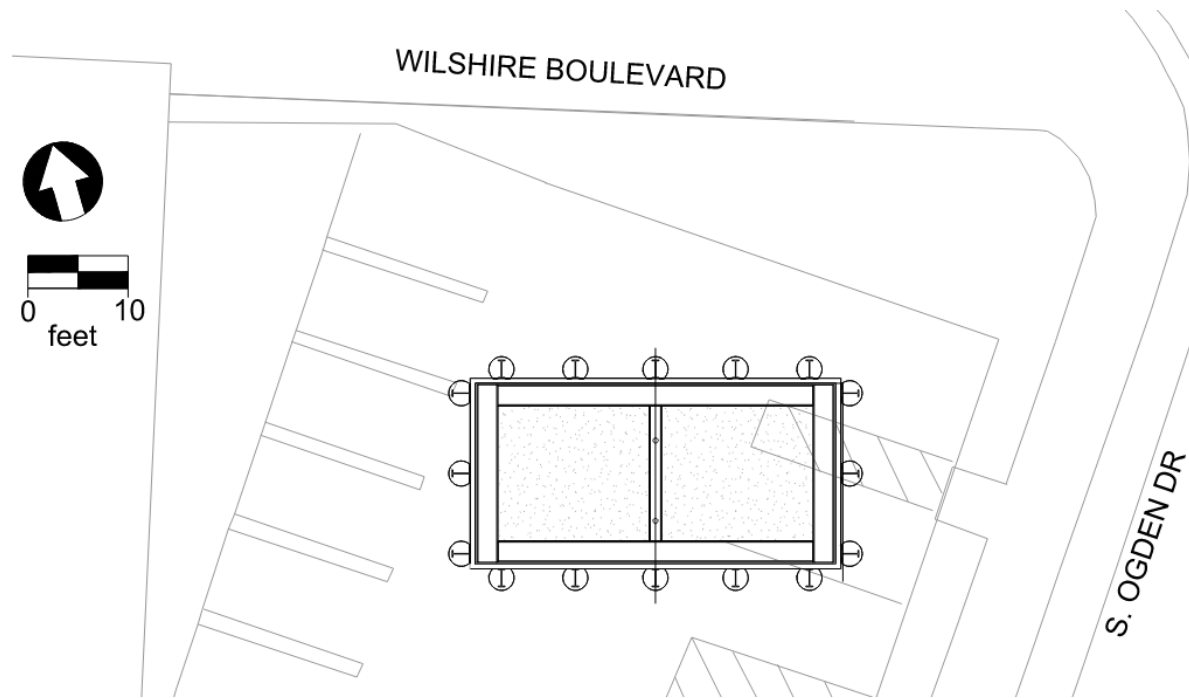


Figure 2: Exploratory shaft near South Ogden Dr and Wilshire Blvd

The excavation and construction of the exploratory shaft (Figure 2) served the following purposes: (1) to investigate the soil stratigraphy and to gather soil data to support the ongoing development of the D-line stations located in similarly challenging tar-infused soil conditions, (2) to develop a small-scale instrumentation and monitoring system that would serve as experimental testbed prior to station construction, and (3) to evaluate the excavation performance behavior in terms of lateral displacements and vertical deformation in the uniquely challenging tar-impacted soils.

This MS thesis will focus on the numerical modeling of the exploratory shaft and its excavation performance using the commercially available software PLAXIS (2D and 3D) by Bentley (insert

reference) and validate/calibrate a numerical model with field measurements recorded during construction. The thesis is organized into two chapters. Chapter one provides the site-specific case history, including background, site conditions, excavation configuration, instrumentation, as well as the analysis of field data. Chapter two presents the numerical modeling, including model description, derivation of model parameters, model results, and comparison with field measurements.

Exploratory Shaft – Structural System and Geometry

The Exploratory Shaft (Figure 3) is a rectangular shaft with excavated dimensions of 19 feet in width, 37 feet in length, and a total depth of 74.5 feet. The structural system consisted of 16 soldier piles, 16 walers, and four struts. The soldier piles consisted of W21x93 steel sections; five soldier piles were placed along the long shaft face and three soldier piles along the short face. All piles were spaced eight feet on-center. The piles extended to a depth of 87 feet below finish grade, i.e., 12.5 feet below the final depth of excavation (74.5 ft). Finish grade is defined as the ground level elevation prior to excavation (Figure 3) and corresponds to the final post-construction elevation, matching the pre-construction elevations of the surrounding area. Walers were placed along the perimeter of the shaft to create a structural frame that was welded to each soldier pile along all four sides. Walers along the short side of the shaft consisted of W24x176 sections at all four levels. The long shaft side utilized W24x192 waler sections in level one and W24x176 sections in the subsequent levels two to four. The struts were located at mid-span of the long side and varied with depth. Level one was supported with a W14x176 strut, while subsequent levels, two to four, employed W14x159 sections. All struts were welded to the walers along the long side (Figure 3). Figure 3 also shows the waler spacing in terms of elevations.

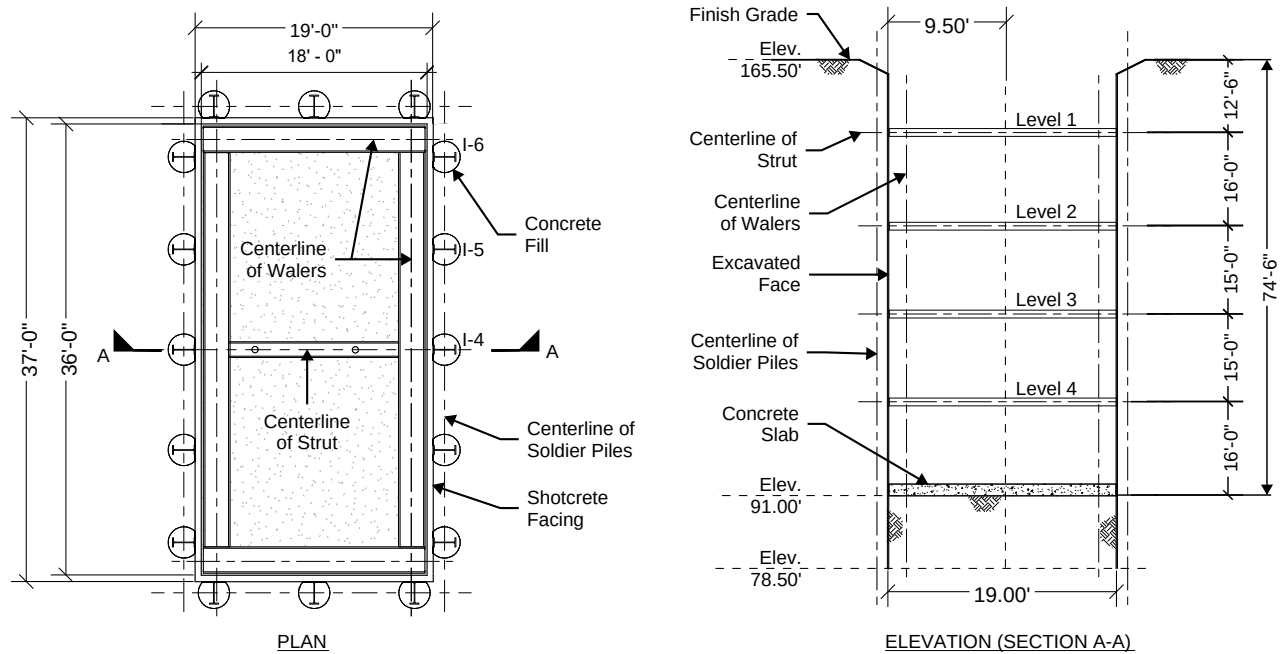


Figure 3: Exploratory shaft plan and elevation views

A geocomposite drainage system, consisting of two 1-foot wide, 0.40-inch thick, drainage mats, was placed between each soldier pile and fastened against the excavated face using 60D penny nails to prevent the shotcrete from getting in between the filter fabric and the soil. A 6-inch-thick shotcrete facing with welded wire reinforcing was cast against the drainage system and fastened to the soldier piles through welded studs. The installation of the drainage system is intended to prevent hydrostatic build-up at the excavated face, reducing overall lateral pressures. Depths of 55-feet to 65-feet required timber lagging to stabilize wall sloughing before the placement of shotcrete facing.

Limited information was provided about the strut preloading. As-built plans suggested struts to be preloaded by a total load of 176 kips at each strut level; however, an observation report by the

contractor notes that only minimal preloading during construction occurred [1]. Furthermore, the construction progress reports indicate preloading of the struts at certain levels; however, no record of preloading magnitude is documented. While the struts were instrumented with strain gauges, which could provide insight about potential preloading applied in the field, strain gauge readings were not collected until preloading was applied, and hence no baseline (aka “zero” data) is available. Preloading can limit localized displacement at the location of the strut that would otherwise occur; these influences will be evaluated within the numerical models to determine the likelihood of preloading occurring at any given strut level.

Excavation Construction Sequence

Figure 4 through Figure 8 illustrate the 14 stages of the top-down excavation construction derived from construction progress reports provided by Parsons Brinkerhoff [2]. Upon reaching a new excavation stage (depth), a new set of struts and walers was installed at an elevation approximately 2.5 to 4.5 feet above the current excavation level. This placement allowed for sufficient clearance for both, installation and instrumentation (see Figures 4 – 8). At each strut elevation, inclinometer readings were taken. These five elevations are represented through five specific dates (10/18/2013, 12/18/2013, 01/28/2014, 3/12/2014, and 08/28/2014), and five corresponding inclinometer readings as presented later in this thesis. The dates at which lateral displacements were recorded are displayed at the bottom of Figures 4 – 8. A single excavator was utilized to excavate lifts in 5-ft increments; no benching of the soil was noted. The soldier piles, which served as the primary vertical support to prevent lateral displacement, were constructed using wide flange steel sections (W21x93) embedded in cast-in-drilled-holes (CIDH) filled with concrete to create a composite

element. The CIDH pile consisted of lean concrete within the depth of excavation and continued as structural concrete below the excavation bottom.

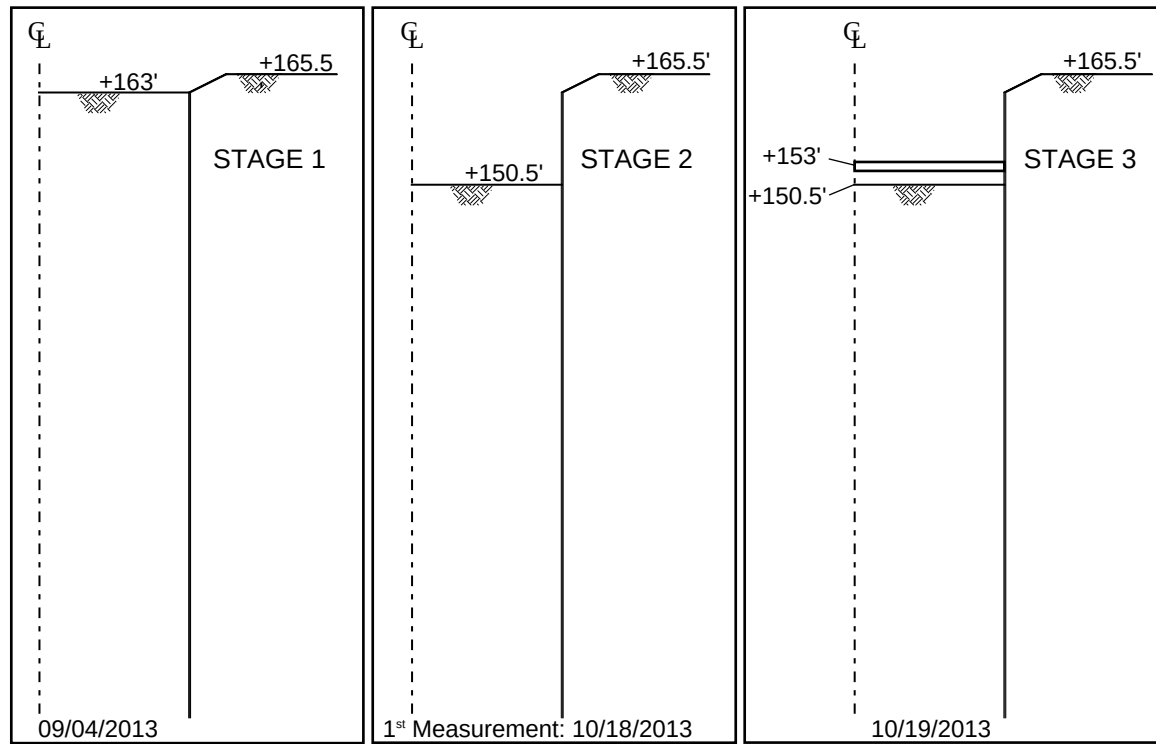


Figure 4: Construction sequence stage 1 – 3, elevations indicate excavation depth and strut elevation

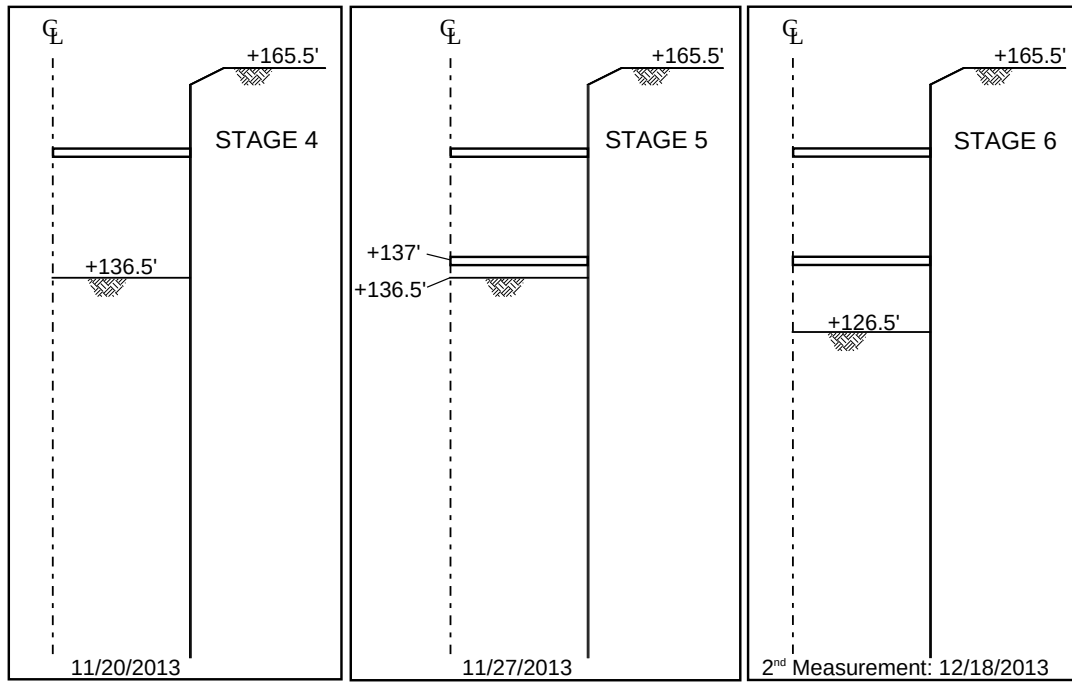


Figure 5: Construction sequence stage 4 – 6 showing excavation depth and strut levels

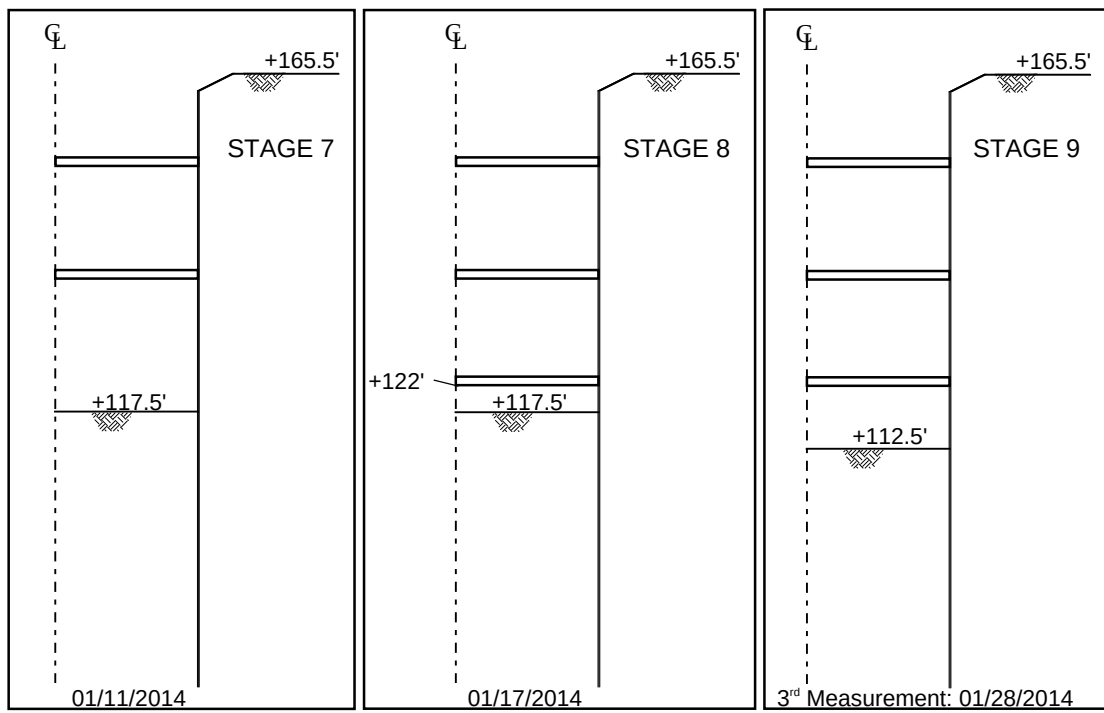


Figure 6: Construction sequence stage 7 – 9 showing excavation depth and strut levels

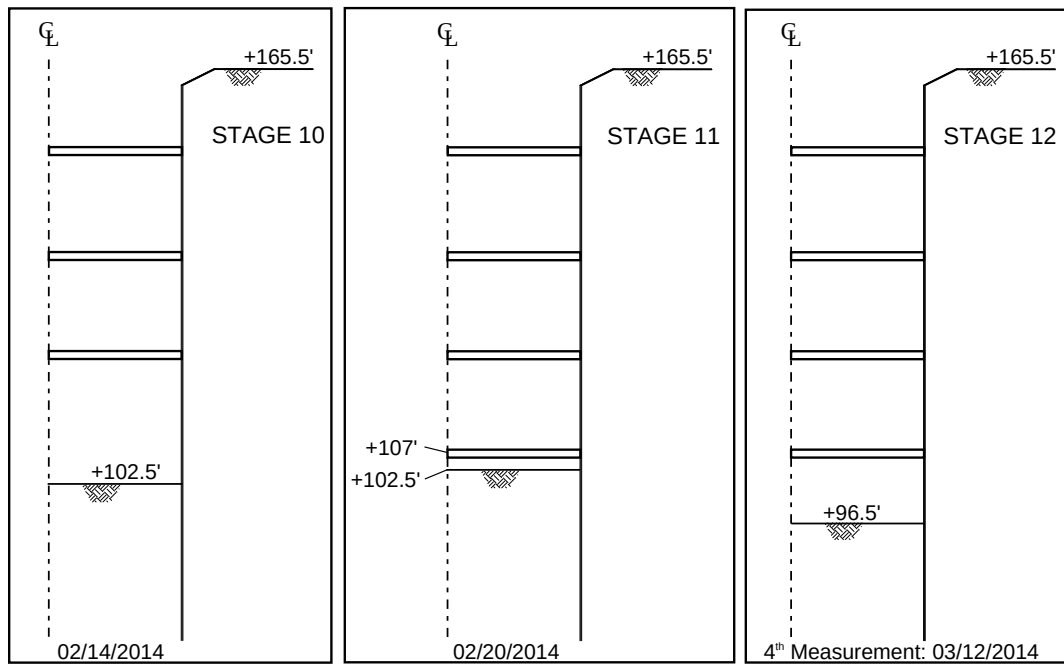


Figure 7: Construction sequence stage 10 – 12 showing excavation depth and strut levels

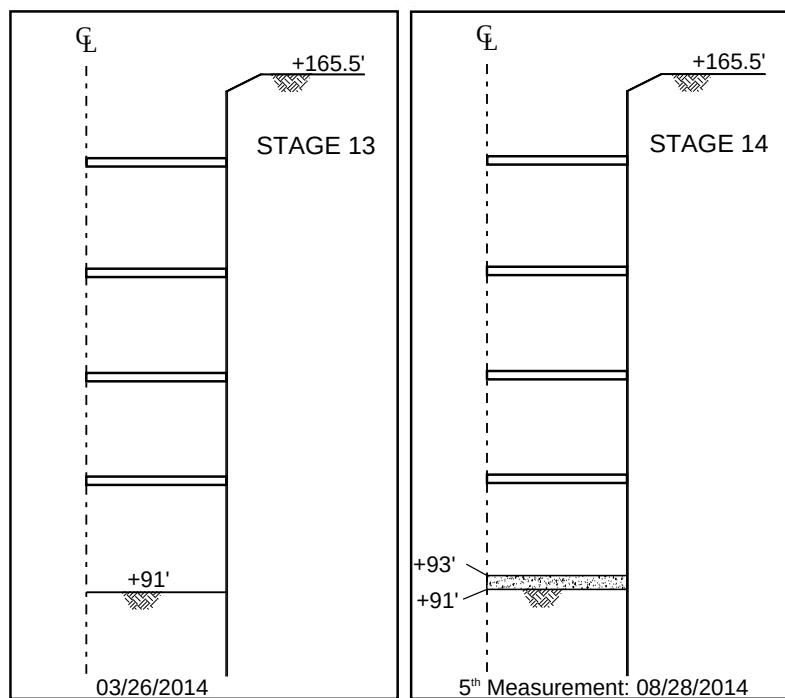
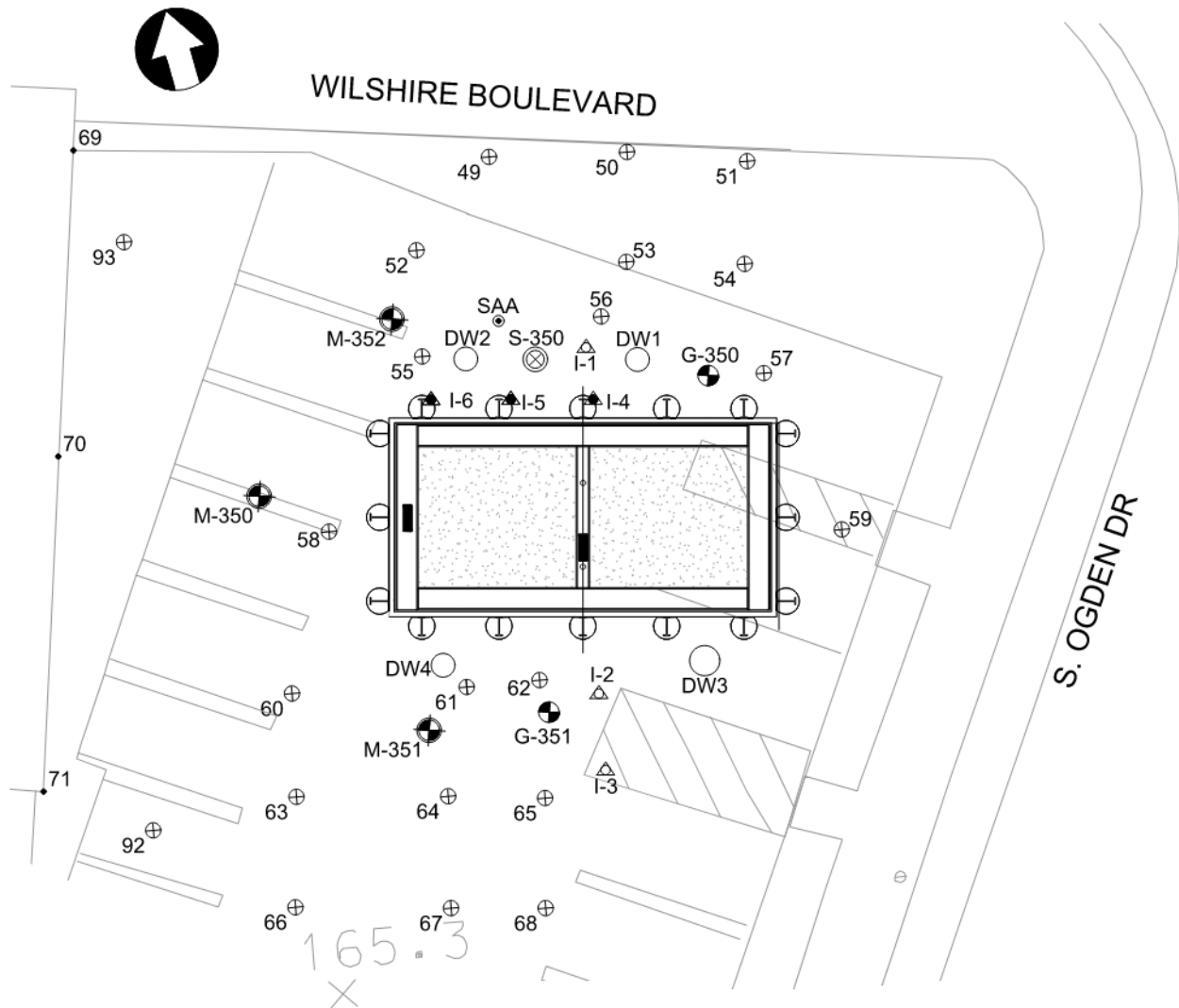


Figure 8: Construction sequence stage 13 – 14 showing final excavation depth and all four strut levels

Shaft Instrumentation and Monitoring

The instrumentation system within and around the exploratory shaft is shown in Figure 9 and consisted of six inclinometers (monitoring lateral excavation movements), 22 settlement markers (monitoring vertical surface deformation), seven gas sensors at the surface, 16 gas sensors within the exploratory shaft, four dewatering wells, and 34 strain gauges. Three inclinometers (I-1 through I-3) were placed within the soil surrounding the exploratory shaft. Three additional inclinometers, I-4, I-5, and I-6 were placed directly at the soldier piles, namely at mid-span, quarter-span, and near the corner respectively (Figure 9). The analysis herein focuses predominantly on inclinometers I-4 through I-6, as those represent the movement of the soldier piles within the exploratory shaft; settlement marker measurements are also utilized as a direct comparison of vertical displacements (i.e., surface settlements) outside the exploratory shaft.



LEGEND

- △ ROTARY-WASH BORING WITH A SLOPE INCLINOMETER AND VIBRATING WIRE PIEZOMETERS
- ▲ INCLINOMETER ATTACHED TO SOLDIER PILE
- ⊙ ROTARY-WASH BORING WITH VIBRATING WIRE PIEZOMETERS
- ⊕ HOLLOW-STEM-AUGER BORING WITH METHANE WELLS AND PROBES
- ⊙ ROTARY-WASH BORING WITH AN SAA INCLINOMETER
- ⊗ SONIC BORING WITH GAS MOTORING WELLS AND PROBES
- DEWATERING WELL
- BUILDING SETTLEMENT REFERENCE POINT
- ⊕ SURFACE SETTLEMENT MARKER
- ① SOLDIER PILE
- STRAIN GAUGE ON SUPPORT SYSTEM ELEMENT

Figure 9: Instrumentation Monitoring near Exploratory Shaft (redrawn after Parsons Brinkerhoff)

A total of four dewatering wells were installed in sets of two and placed along the long-side perimeter of the shaft. All four wells were in operation prior to the excavation of the shaft, removing approximately 1,500 gallons per day at their optimal performance. The observation report notes clogging of the filter fabric to impact the flow through the perforated casings; and further documents that various wells experienced caving. For instance, caving was reported in one well (dewatering well #2) at approximately 48 feet below finish grade. After six weeks of operation, all four wells were considered inoperable. All monitoring wells/bore holes denoted by G-#, S-350, and M-#, utilized vibrating piezometers and standpipes with dip meters to measure water pressures prior to and during the first few stages of construction. The water levels varied from 8 to 45 feet below finish grade based on measurements taken at G- and M- wells. Water levels in S-350 (5.5 feet away from the shaft) vary from surface level to 25 feet below finish grade. Further groundwater data from the Wilshire/Fairfax station (Geotechnical Data Report – Wilshire/Fairfax Station) [3] also indicates ranges of surface level to 29.5 feet below finish grade.

Site Conditions

Soil data from the site was collected through in-situ testing and soil sampling. Boring logs from various locations around the project site were also provided, including the G-350 boring adjacent to north face of the excavation. A total of six characteristic soil layers can be distinguished for the exploratory shaft site. Figure 10 provides the soil stratigraphy derived from borehole G-350 and its corresponding description. Note, the water table elevation shown in Figure 10 is only for the borehole G-350, the water table utilized for modeling is further described in Chapter 2.

Figure 11 presents the soil stratigraphy observed for the north face (plane of I-4 through I-6). This figure represents the geological exploration of the various soil types within the exploratory shaft

during the course of the excavation. The colored diagram on the left (Figure 11.a) indicates the range in soil types observed, such as alluvial deposits, Lakewood Formation, and San Pedro Formation. The gray-scale colored diagram on the right (Figure 11.b) shows the amount of tar-infused soil observed at the time of excavation; this ranges from slightly to highly infused tar soil. The green shading represents hydrocarbon stained zones occurring above the tar-infused soil. The white blocked out zones (resembling lagging), represent the zones that caved during excavation and needed to be immediately shored; identification and observation of tar-infused soil could not take place.

Based on archaeological findings and geotechnical explorations, tar-impacted soils were found below 15 feet of depth and varied from moderate to highly infused tar sand [1]. The influence of tar on a soil medium varies; however, tar presence generally reduces the permeability of the soil, increases the cohesiveness [4] and changes its phase (gas vs liquid) in dependency of pressure. A detailed study of the tar soil behavior and the influence of bitumen on the global construction performance can be found in Chrysovergis et al. (2024).

Laboratory testing of tar-infused soil is not included in this thesis; instead, field data are used to derive the overall material properties required for the specific soil models. These properties include parameters such as shear modulus G_{vs} , soil modulus E_{vs} , undrained shear strength S_u , and friction angle ϕ .

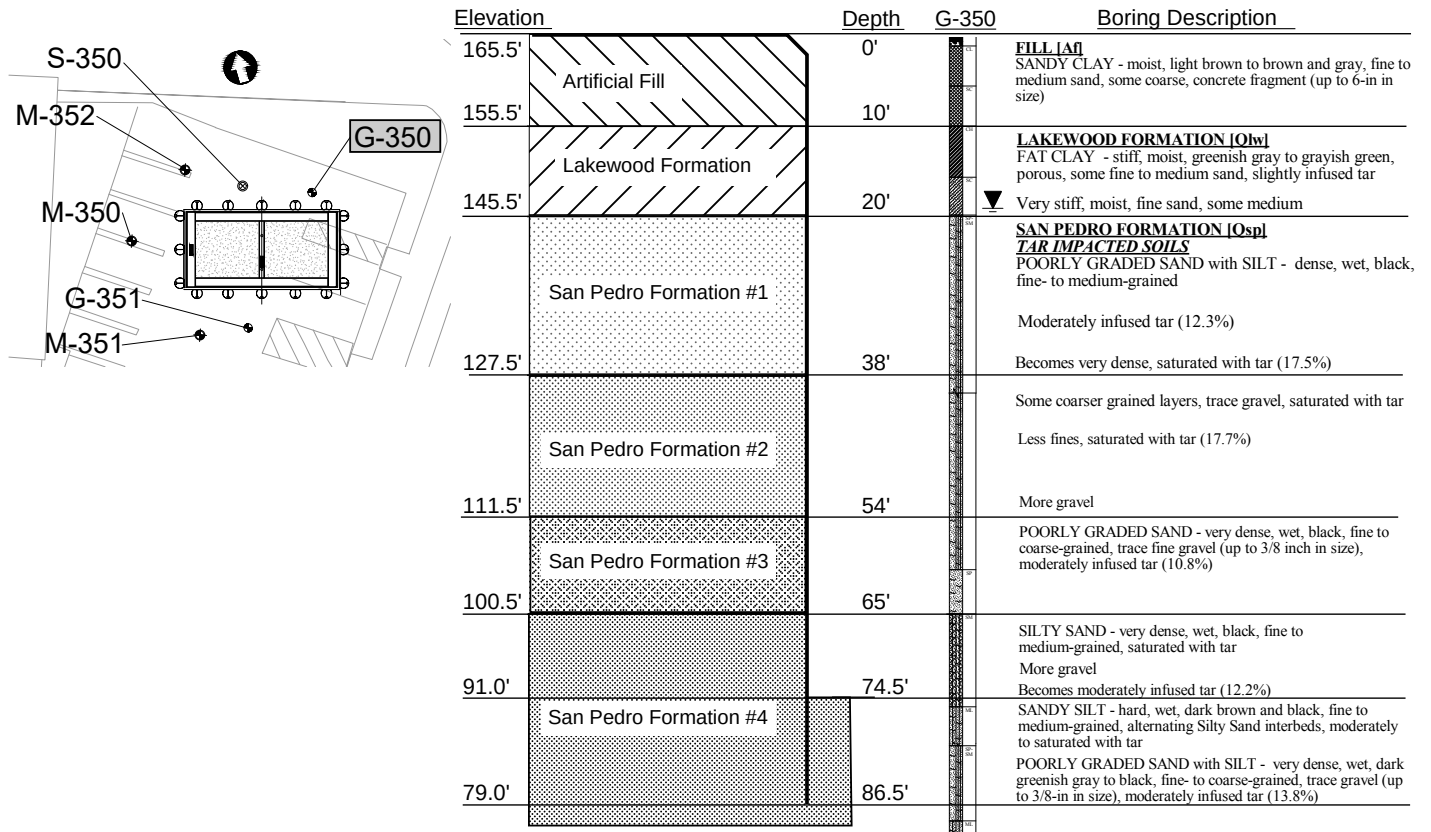
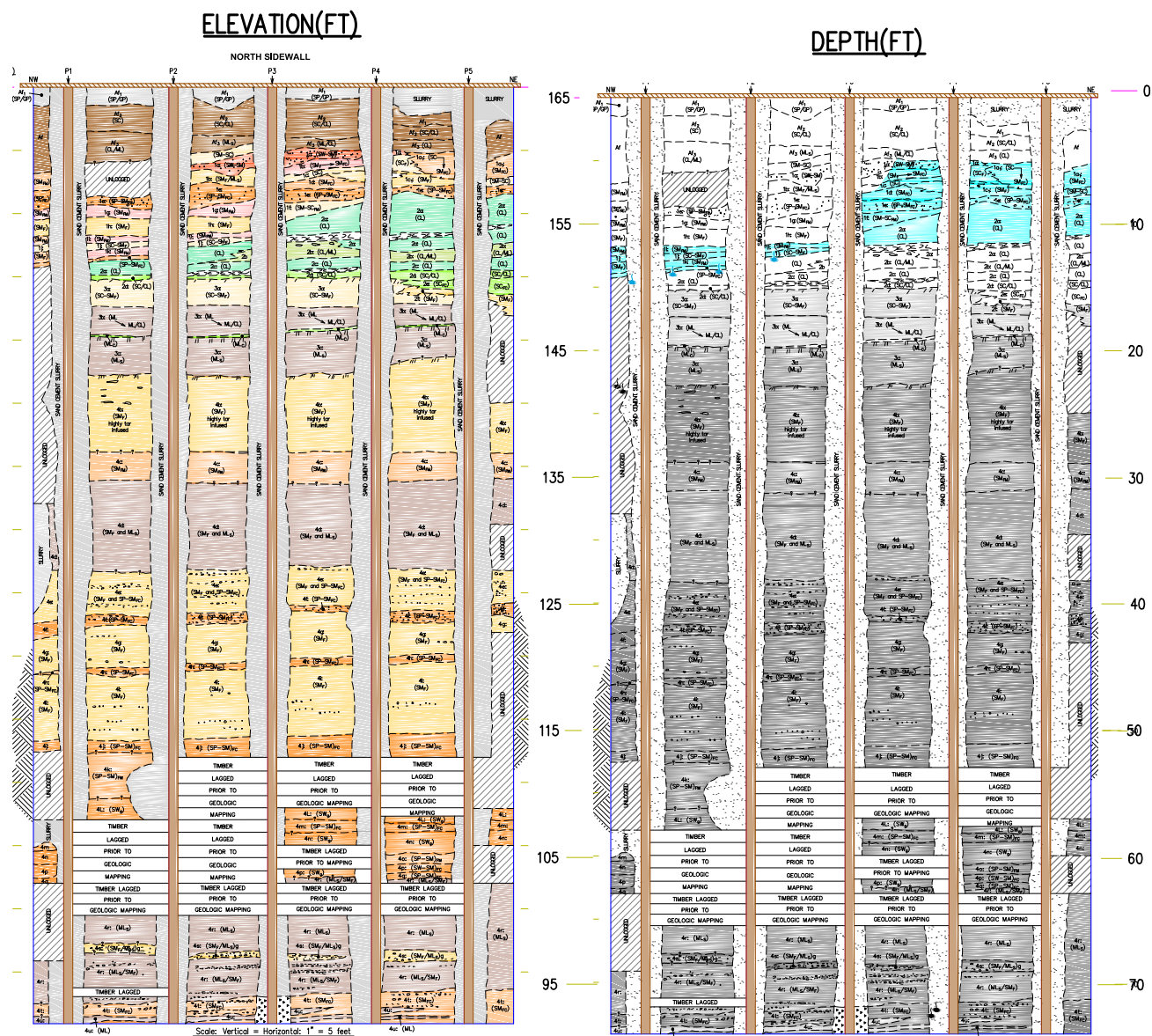


Figure 10: Soil Stratigraphy (a) Distinguished Layers and (b) G-350 Boring Data



Symbol

- Slightly tar infused
- Moderately tar infused
- Highly tar infused
- Apparent hydrocarbon stained zone

Figure 11: Geological Exploration (a) Identified Soil Layers and (b) Tar Infused Soils (shown in grey)

On-site Photos of Construction Sequence.

The following Figures (Figs.12 through17) show various photos of the construction sequence, as provided by the contractor. Each Figure displays representative elevations within the excavation and construction process.



Figure 12: Excavation to 5-ft within Exploratory shaft, Installation of Cap Beam (Photos by Parsons Brinkerhoff [2]: Sept-2013)



Figure 13: Installation of First Strut and Waler at 15-ft below ground surface (Photos by Parsons Brinkerhoff [2]: Oct-2013)



Figure 14: Installation of Second Waler at 29-ft below ground surface (Photos by Parsons Brinkerhoff [2]: -2013)



Figure 15: Installation of Third Strut and Waler at 48-ft below ground surface (Photos by Parsons Brinkerhoff [2]: Jan-2014)



Figure 16: Installation of Fourth Strut and Waler at 63-ft below ground surface (Photos by Parsons Brinkerhoff [2]: Feb-2014)



Figure 17: Installation of Invert Slab at 72.5-ft below ground surface (Photos by Parsons Brinkerhoff [2]: March-2014)

Field Measurements

Lateral Displacements

Lateral movements of the exploratory shaft were evaluated using inclinometers readings I-1 through I-6. Three inclinometers (I-1 through I-3) were installed outside of the shaft, placed within the soil at approximately 6 to 12 feet away from the excavation facing. Inclinometers I-4, I-5, and I-6 were directly placed on the mid-span, quarter-span, and corner steel soldier piles of the long side of the excavation (Figure 18).

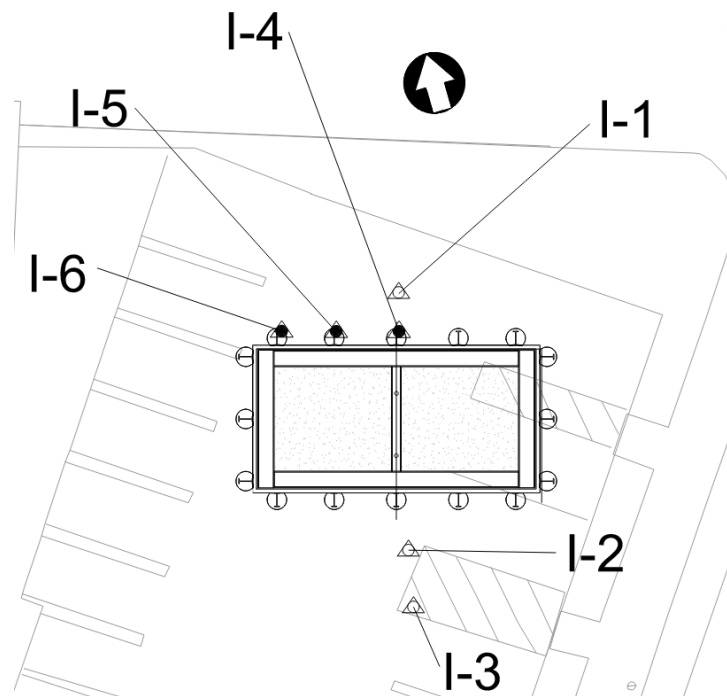


Figure 18: Location of Inclinometers I-1 through I-6

For the deflection monitoring of the shaft walls, inclinometer measurements I-4 through I-6 are most relevant for the analysis presented herein. Over the course of the 8-month construction, a total of 19 measurements (taken on 19 separate days) were recorded for I-4 through I-6. However, not all 19 days are needed to evaluate the lateral shaft displacement and establish a future numerical

model. Only five measurement dates, which correspond to the five critical excavation depths and five corresponding strut installation levels were selected. These dates and their corresponding excavation depths are shown in Table 1 and Figure 13, along with the maximum measured lateral displacement.

Table 1: Max Lateral Displacement Observed from Inclinerometers

Measurement Date	Excavation Depth [ft]	Max lateral displacement observed on that date		
		I-4 (center) [inch]	I-5 (mid-span) [inch]	I-6 (corner) [inch]
10/18/2013	15	0.16	0.19	0.02
12/18/2013	39	0.48	0.43	0.13
01/28/2014	53	0.61	0.54	0.14
03/12/2014	69	0.88	0.86	0.21
08/28/2014	74.5	0.95	1.0	0.20

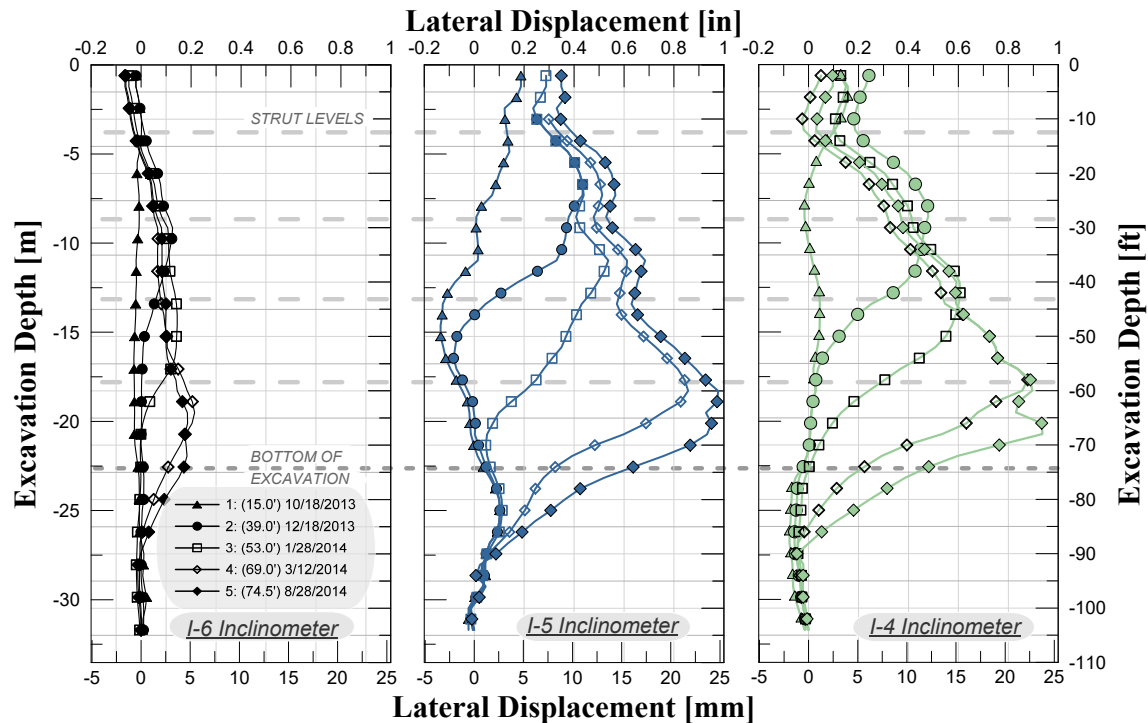


Figure 19: Inclinerometer measurements (i.e., lateral displacements recorded by I-6 (corner), I-5 (quarter-span), and I-4 (mid-span))

The maximum lateral displacements observed from Figure 19 is 0.95, 1.0, and 0.20 inch for I-4, I-5, and I-6, respectively. Figure 20 presents the maximum lateral displacements at each excavation depth for all inclinometers. All data points follow a relative linear trendline. When reaching the bottom of the excavation (74.5ft) the majority of inclinometers (I-1, I-2, I-4 and I-5) read lateral deformations around 0.1%H (where H is depth of excavation). These inclinometers are all near the center of the exploratory shaft; whereas I-3 and I-6 are furthest away from the center and correspondingly display much smaller displacements.

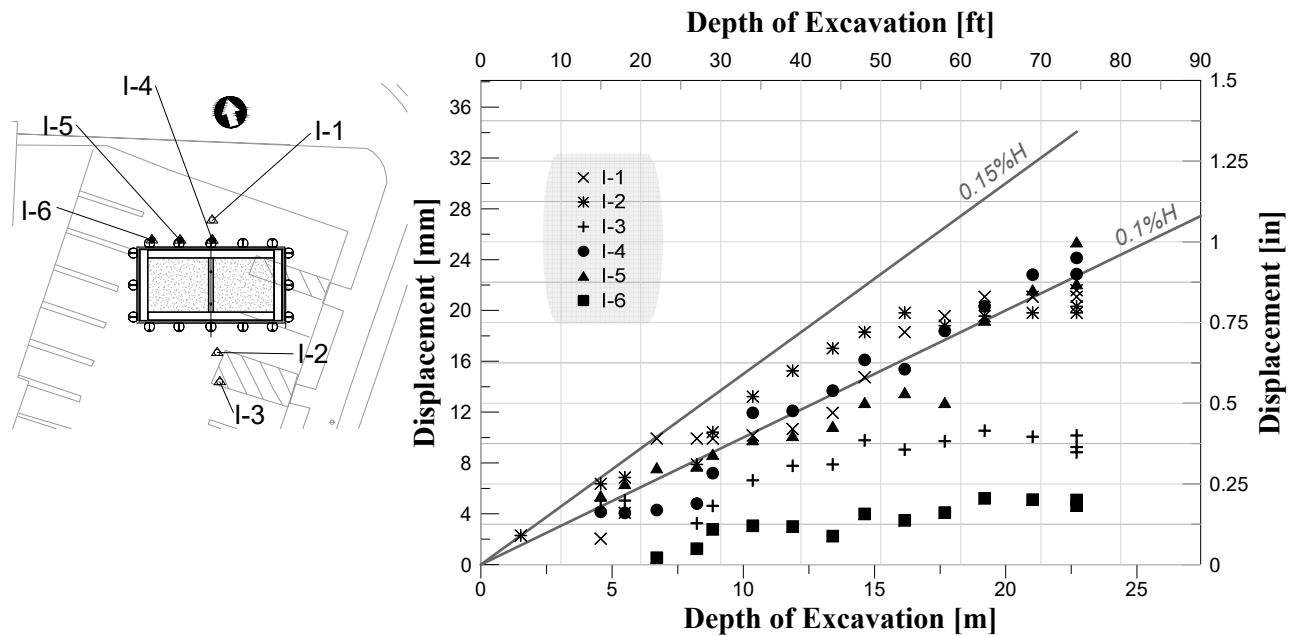


Figure 20: Maximum Inclinometer Displacements Measured at Piles (I-4 through I-6) and in Soil (I-3 through I-6) at Various Depths

To further evaluate the lateral deflections with respect to the overall excavation system, the plane strain ratio (PSR) is calculated. The PSR provides an approximate relationship between the lateral deflections and vertical settlements observed in a 2D plane-strain model and the estimated deformations or settlements due to 3D geometric effects. To obtain the PSR, the system stiffness “S” of the excavation, along with the Factor of Safety “FS” is determined first.

System Stiffness

The “system stiffness,” S , as defined by Clough *et al.* (1989) [5], is utilized to characterize the wall stiffness relative to other shoring systems such as sheet piles walls, diaphragm walls, and soil-nail walls. The system stiffness for the exploratory shaft is determined using the relationship shown in Equation 1. Hereby the stiffness EI should be taken as an equivalent stiffness which represents EI of the soldier pile (W section) distributed over the pile spacing.

$$S = \frac{EI}{h^4 \gamma_w} * \frac{1}{x} \quad (\text{Equation 1})$$

Where:

E = Elastic modulus of a single steel soldier pile (29,000,000 psi)

I = Area moment of inertia of a single steel soldier pile (2070 in⁴)

h = Average spacing between struts (186 in)

γ_w = Unit weight of water (0.0361 pci)

x = Spacing of soldier pile on center (96 in)

Following Clough *et al.* [5], the definition of stiffness ranges from 10–3000; the smaller the values of “ S ”, the more flexible the excavation system. Larger values of system stiffness represent very

stiff systems [6]. In the case of the exploratory shaft, the system stiffness is considered flexible at a value of $S = 14.47$ per Equation 1.

Factor of Safety

The factor of safety against basal heave, FS, is described as the ratio of resistance divided by the load and can vary based on unit weight of soil and undrained shear strength S_u [7]. The mechanics of basal heave failure are derived from bearing failure due to vertical loads from the excavation exceeding the undrained shear strength of the soil.

The following relationship (i.e., Bjerrum and Eide (1956) [8]) has been established in literature to calculate the FS through the simplified equation summarized in Table 2. This method was developed primarily for narrow excavations where the effect of wall stiffness is not considered. The depth below the bottom of excavation to bedrock (or hard stratum) must also be greater than $B/\sqrt{2}$. These conditions are met as D (depth to bedrock or hard stratum) goes beyond the limits of the borings (>40 ft below bottom of excavation), and $B/\sqrt{2} = 26.87$ ft; and B (19 ft) < L (37ft). The undrained shear strength S_u was determined for each layer through Laboratory testing. This allowed the stage-dependent determination of the respective FS at five different dates, as shown in Table 3. The bearing capacity factors were determined from Skempton (1951) [9], utilizing the ratio of depth of excavation over the width for a rectangular shape. It is important to note that the FS determined in Tables 2 and 3, assumes no construction surcharge (q), i.e., $q = 0$. This scenario is only applicable to the Exploratory Shaft, where no surrounding construction (e.g., superstructural elements) or other heavy construction equipment were present.

Table 2: Sample Factor of Safety Against Basal Heave Calculation for Bjerrum and Eide (1956) at an Excavation Depth of 74.5 ft

<i>Bjerrum and Eide(1956)</i>	
$FS = \frac{1}{H} * \frac{N_c S_u}{\gamma} =$	(Equation 2)
$FS = \frac{1}{H} * \frac{8.12 S_u}{\gamma} = \frac{1}{74.5 ft} * \frac{8.12(7,400 \text{ psf})}{122 \text{ pcf}} = 6.61$	
$\underline{FS = 6.61}$	
H = Excavation depth [74.5 feet] N_c = Bearing capacity factor based on Skempton (1951) varies by depth [9] D = Depth below bottom excavation to bedrock ($>B/\sqrt{2}$) B = Width of exploratory shaft (19 feet) S_u = Undrained shear strength of soil layer below [7,400 psf] γ = Unit weight of soil [122 pcf]	

Table 3 shows a summary of the factor of safety against basal heave calculated using Equation 2 for the five excavations depths.

Table 3: Factor of Safety Against Basal Heave at various excavation depths (see time stamps)

Date	Excavation Depth [ft]	γ_s [pcf]	$N_{c,rec}$ [-]	S_u [psf]	Factor of Safety Against Basal Heave
					Bjerrum and Eide
10/18/2013	15	129	6.69	3,500	12.09
12/18/2013	39	123	7.65	5,700	9.09
01/28/2014	53	122	7.93	7,900	9.69
03/12/2014	69	122	8.02	7,400	7.05
08/28/2014	74.5	122	8.12	7,400	6.61

Since the exploratory shaft is considered a narrow deep excavation, Equation 2 is an appropriate method to be utilized for further analyses. As such, the FS (per Bjerrum and Eide, Equation 2) and the system stiffness, S , are utilized to determine two distinct parameters: (a) the first being the plane-strain ratio (PSR) as defined in Finno *et al.* (2007) [10] and shown in Equation 3 (b) the second is the ratio of the theoretical maximum lateral movement relative to excavation depth.

$$PSR = (1 - e^{-kC(L/H)}) + 0.05 \left(\frac{L}{B} - 1 \right) = \frac{43D}{42D} \quad (\text{Equation 3})$$

$$k = 1 - 0.0001(S) \quad (\text{Equation 4})$$

$$C = 1 - [0.5(1.8 - FS)] \quad (\text{Equation 5})$$

Where:

L = Width of long side of shaft (37 feet)

C = Based on factor of safety against basal heave FS ($C = 1$ if $1.8 \leq FS$)

k = Based on system stiffness S ($k = 1$ for $S \leq 100$)

Table 4: Plane-Strain Ratio (PSR)

Measurement Date	Excavation Depth [ft]	FS Bjerrum And Eide	S	k	C	PSR	L/H
10/18/2013	15	12.09	14.47	1.0	1.0	0.96	2.47
12/18/2013	39	9.09	14.47	1.0	1.0	0.66	0.95
01/28/2014	53	9.69	14.47	1.0	1.0	0.55	0.70
03/12/2014	69	7.05	14.47	1.0	1.0	0.46	0.54
08/28/2014	74.5	6.61	14.47	1.0	1.0	0.44	0.50

The importance of the PSR is to provide an approximate relationship between the displacements observed in a 2D plane-strain model and the theoretical displacements due to 3D geometric effects, i.e., $PSR = \Delta_{3D}/\Delta_{2D}$. For a PSR of 0.5, the 2D displacements would be halved to determine the theoretical 3D displacement. Table 4 displays the PSR for the 5 different excavation stages, while Figure 21 shows the PSR for various excavation depths during which inclinometer readings were taken. The PSR approaches 0.44 as the depth of the shaft reaches its bottom level; this indicates that during this fully excavated stage, the 3D geometric effects of the exploratory shaft would allow for as much as a 44% reduction to the maximum displacements calculated with a 2D model. The PSR can be later compared to the numerical results from the 3D and 2D models.

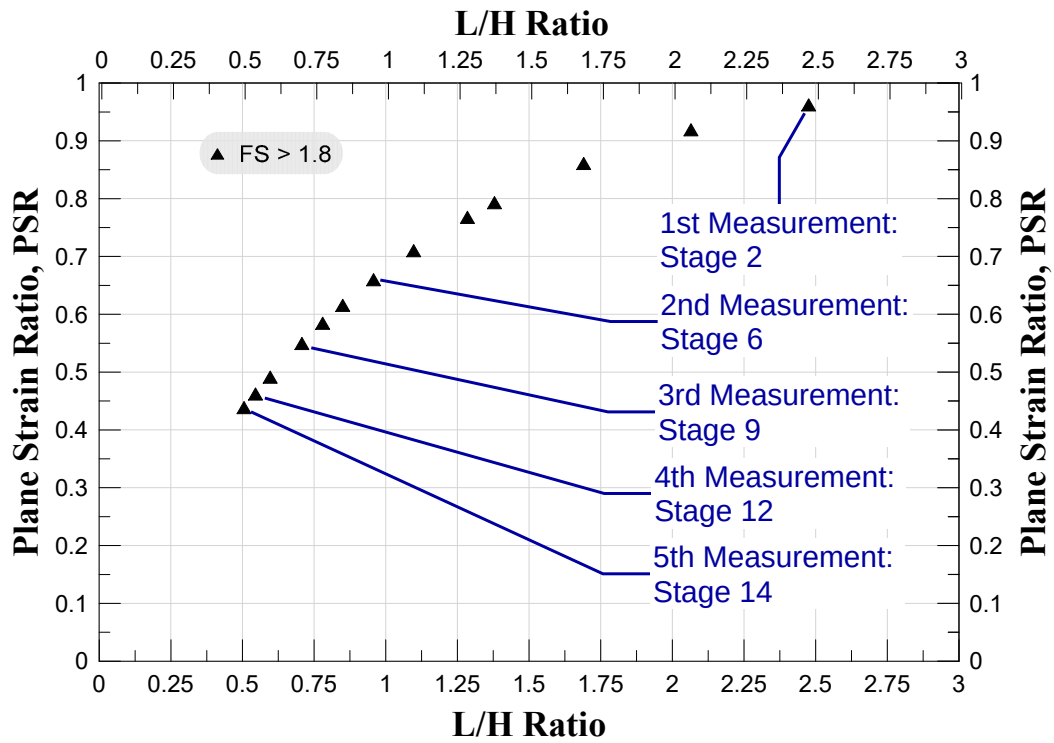


Figure 21: PSR for the Various Excavations Depths

The second characterization to be derived from the preceding analysis is the ratio of the theoretical maximum lateral movement to excavation depth, as defined in Clough and O'Rourke (1990) [11] and shown in Figure 22. The maximum lateral movement ratio is correlated to the factor of safety against basal heave as well as the system stiffness. This value can be compared to the observed measured resultants as a reference check; however, both the PSR and the theoretical maximum lateral movement are developed for soft to medium clay soils as opposed to the stiff clays observed at the exploratory shaft.

Following Figure 22, a system stiffness of 14.47 and a FS of 6.61 would suggest a theoretical maximum lateral movement of 0.16%H (1.45 inch) when reaching the maximum excavation depth

of 74.5 feet in the exploratory shaft. Maximum measured movements, however, (e.g., from I-4 and I-5) were recorded to be only 0.95" and 1.0" respectively). The theoretical overprediction could be attributed to the much larger soil stiffnesses (E , G) observed near the exploratory than those based on which the original model of Figure 16 was derived for (i.e., soft to medium clays.)

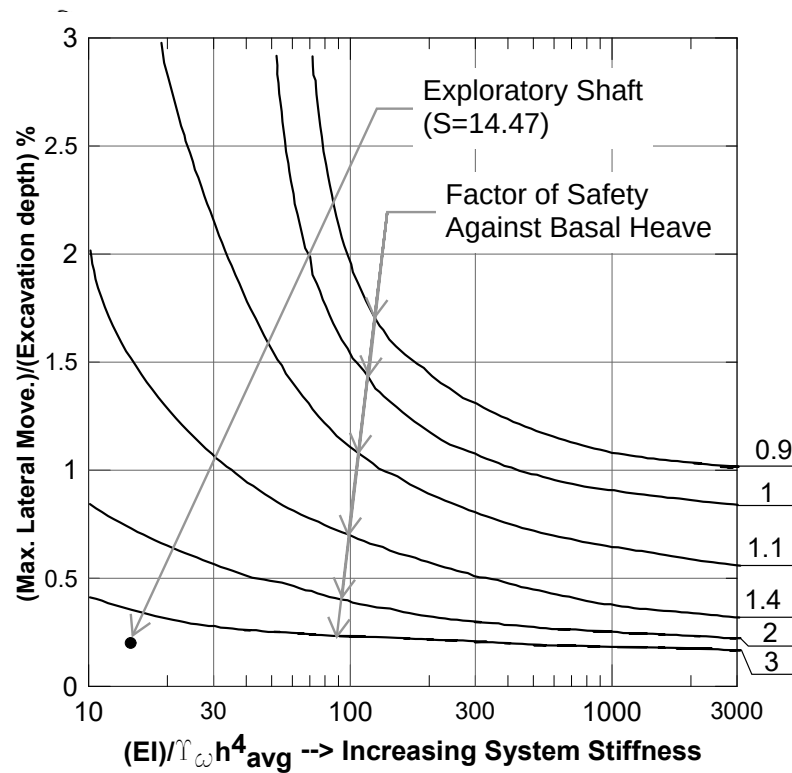


Figure 22: Maximum Lateral Wall Movement (or Soil Settlement), after Clough & O'Rourke (1981), plotted with the Exploratory Shaft Stiffness and FS against Basal Heave

Surface Settlements

A total of 22 surface settlement markers were installed to measure vertical deformations (i.e., surface settlements) near the exploratory shaft. Settlement markers were located between 5 and 30 feet away from the north-face of the exploratory shaft, where the maximum surface settlement was measured to be 0.312 inches at settlement marker 56. All settlement measurements shown in Figure 23 correspond to surface settlements recorded when the full depth of excavation was reached (i.e., 74.5 feet on 3/21/14). The largest surface settlements were recorded at various locations relative to the exploratory shaft, specifically settlement markers 56, 61, 62, 53, and 55. The smallest surface settlements were observed furthest away from the exploratory shaft.

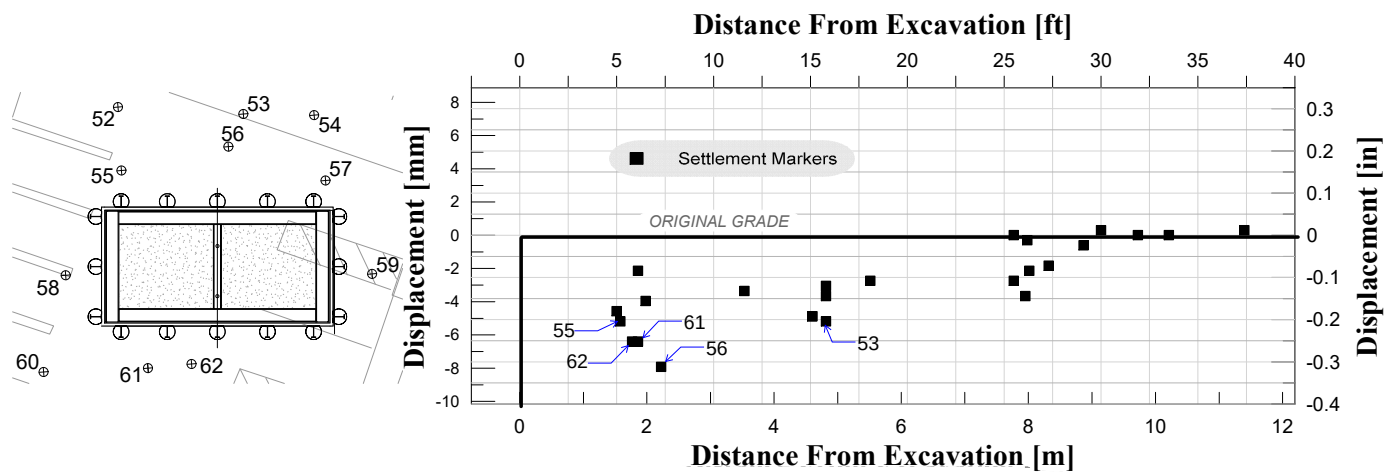


Figure 23: Vertical Deformations (i.e., Surface Settlements) at Maximum Excavation Depth

As described in Clough et. Al [6], maximum settlements in stiff to very hard clays typically occur within a distance of $0.25H$ away from the face of the excavation, where H is the maximum excavation depth. In the case of the exploratory shaft, the maximum measured surface settlement occurs at $0.1H$ away from the face excavation, namely at 7.45 feet from the exploratory shaft. Figure 24 plots surface settlements vs. excavation depth for settlement markers (49–57) on the

north side of the shaft. It can be observed that settlement markers closer to the face of excavation indicate larger settlements with ratios approaching $0.04\%H$.

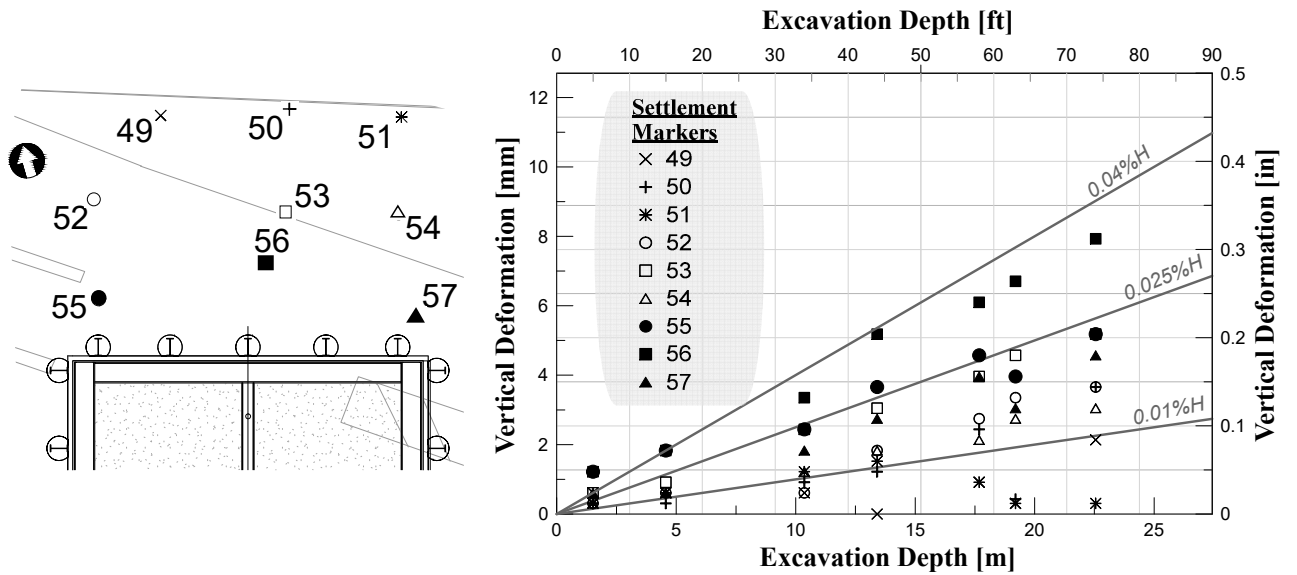


Figure 24: Vertical Deformation at Maximum Excavation Depth at the North side (Marker 49-57)

2 NUMERICAL MODELING

Numerical predictions of excavation settlements and displacements are dependent on-site stratigraphy, geometric complexity, and stiffness of the structural system [5]. In addition, post construction numerical modeling, such as performed herein, requires high quality monitoring data to calibrate soil models and to reliably represent excavation performance behavior. Challenges include the appropriate selection of soil properties, the selection of the appropriate soil constitutive models, the location and numerical implementation of the ground water table (GWT) and its time-dependent variation, a proper representation of the system's structural parameters, and the correct assumption of model boundary conditions. This study considers four different soil models with varying complexities: (1) a linear-elastic soil model (LE), (2) the Mohr-Coulomb model (MC), (3) the Hardening Soil model (HS), and (4) the small-strain stiffness soil model (HS_{small}). Material models that account for small-strain nonlinearity of soil and have robust, yet simple parameter calibration schemes, are particularly suitable for capturing soil deformations.

This chapter will provide a detailed description of how model parameters were selected using field and laboratory data, and how the numerical model was created and calibrated to simulate comparable field performance. It is important to highlight that there is no constitutive material model for tar-infused soils available in current literature.

General Model Setup

2D and 3D numerical models of the exploratory shaft were developed within the finite element (FE) software, PLAXIS ([12]), to study the excavation performance at different excavation stages. The geometric boundaries and water table are identical for both models, except for the 2D model being configured to be symmetric about the strut midpoint (Figure 25). The horizontal model boundaries extend $2H$ (150 feet) from the excavation face; the vertical boundary extends $1.5H$ (115.5 feet) below finish grade. Since most excavation-induced ground movements are confined within approximately two times the excavation depth ($2H$) behind the wall; prior studies have shown that boundary interaction were deemed negligible beyond this distance (Hsieh and Ou (1998) [11], Boonyarak et al. (2024) [13]). Accordingly, the FEM's lateral boundaries were placed at $\geq 2H$ from the wall. In addition, meshing and boundary distancing have been independently verified by PLAXIS technical support. As previously noted, the GWT is placed 15 feet below finish grade with the drawdown occurring near the perimeter of the shaft.

The setup for both the 2D and 3D numerical models begins with defining the soil stratigraphy for the site by assigning the start- and end elevation for each soil layer. Ground water table and soil properties are also assigned in this step and are identical for 2D and 3D. The second step consists of defining the structural components which vary between 2D and 3D. For example, waler elements cannot be defined in 2D, and are only included in the 3D model. Once structural components are defined, the third step involves defining the finite element mesh of the soil layers and structural elements. The fourth step consists of defining the dewatering conditions for each construction stage. Both 2D and 3D models utilize the same parameters for the dewatering sequence. The fifth and final step consists of defining the construction sequence. These steps and their detailed assumptions will be presented within this Chapter.

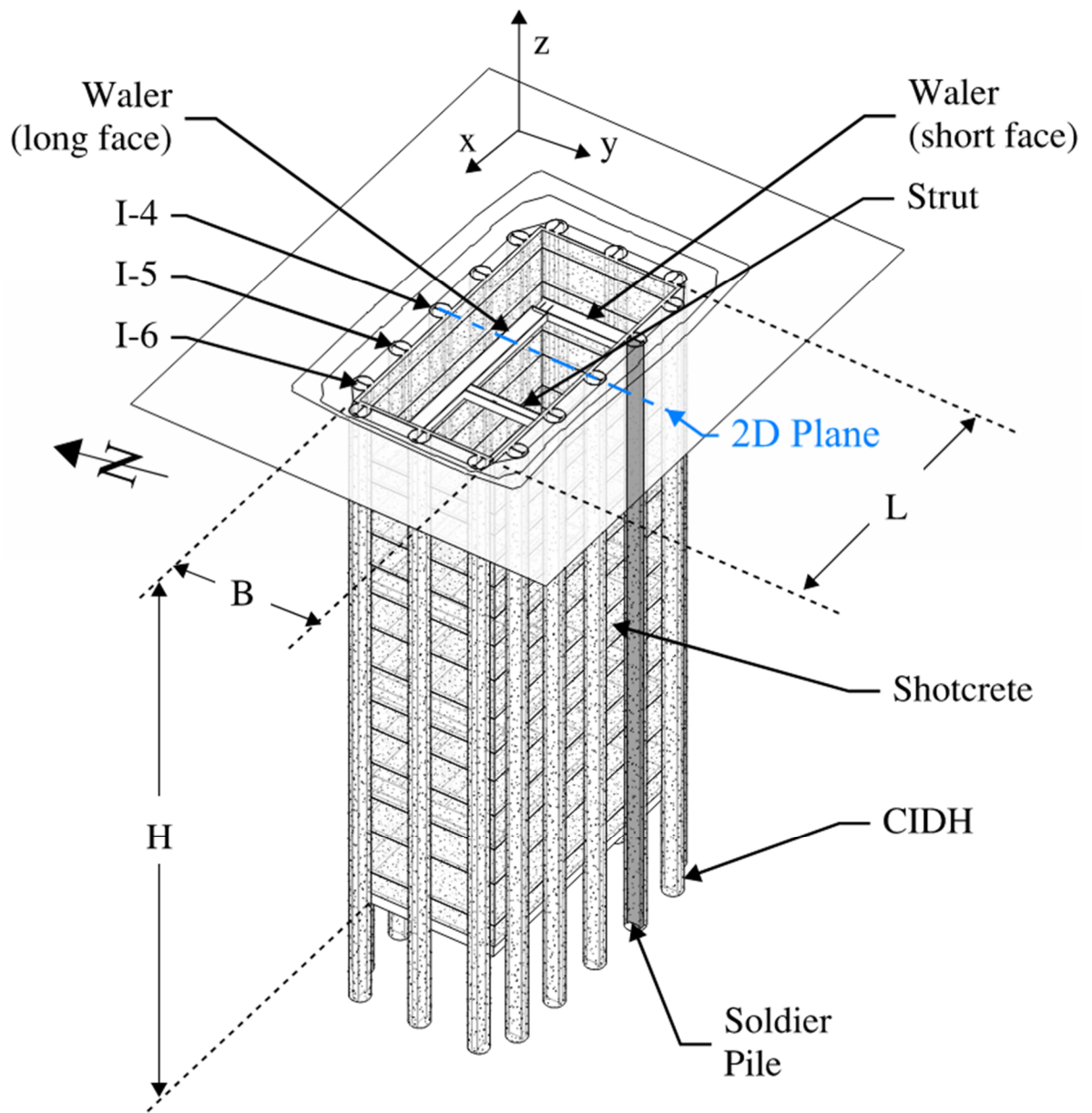


Figure 25: 3D Isometric View of Exploratory Shaft

2D Numerical Model

Figure 26 shows the plan view of the exploratory shaft, and Figure 27 shows the 2D PLAXIS model. The 2D model assumes plane strain behavior and symmetry of the shaft around the midpoint of the long face. Hence, inclinometer I-4, located at mid-span of the long side will be representative of the largest lateral deformations in the shaft.

The 2D simulation is a limited replication of the 3D configuration. The model only considers a cross-section along the strut, therefore walers parallel to the long face of the excavation are omitted within the 2D model. Similarly, the shotcrete facing is omitted in the 2D model, as its stiffness is negligible compared to the stiffness of the soldier pile. Since the 2D simulation requires an input for the interval spacing of the strut, this distance is taken as the spacing between the strut and waler parallel to the short face (16.78 feet, see Figure 26). As a result, the model is limited in its ability to consider the rigid waler/strut framing and corner effects of the exploratory shaft. The remaining structural elements, such as soldier pile and struts, are defined as plates and anchors respectively.

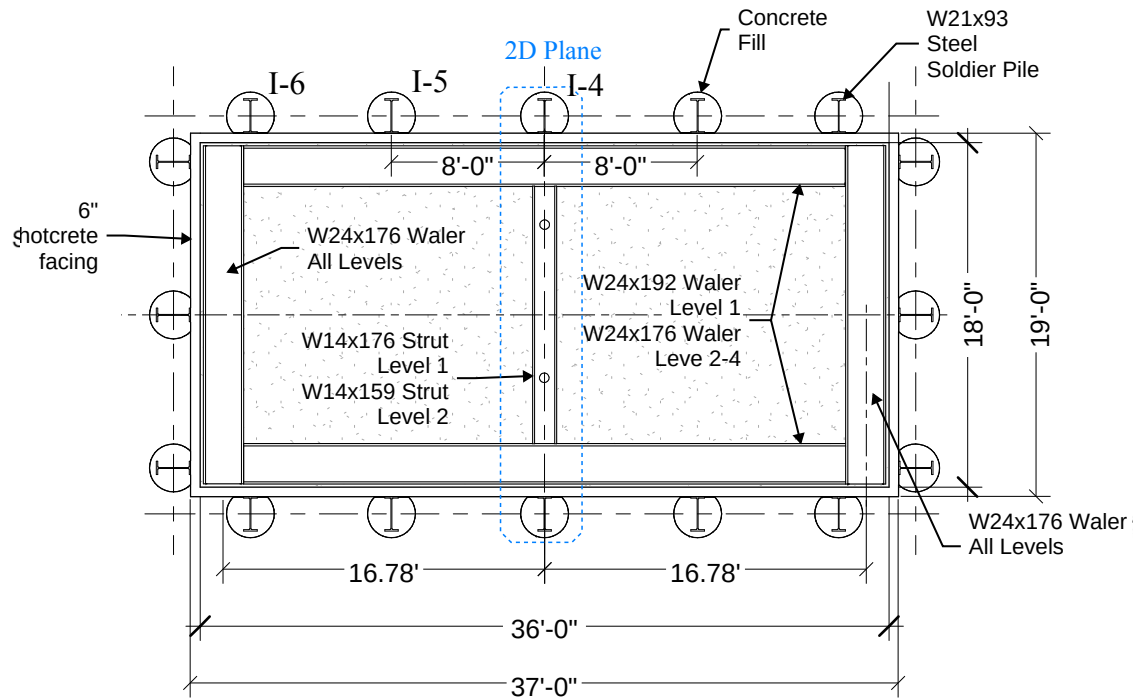


Figure 26: Plan View of Exploratory Shaft

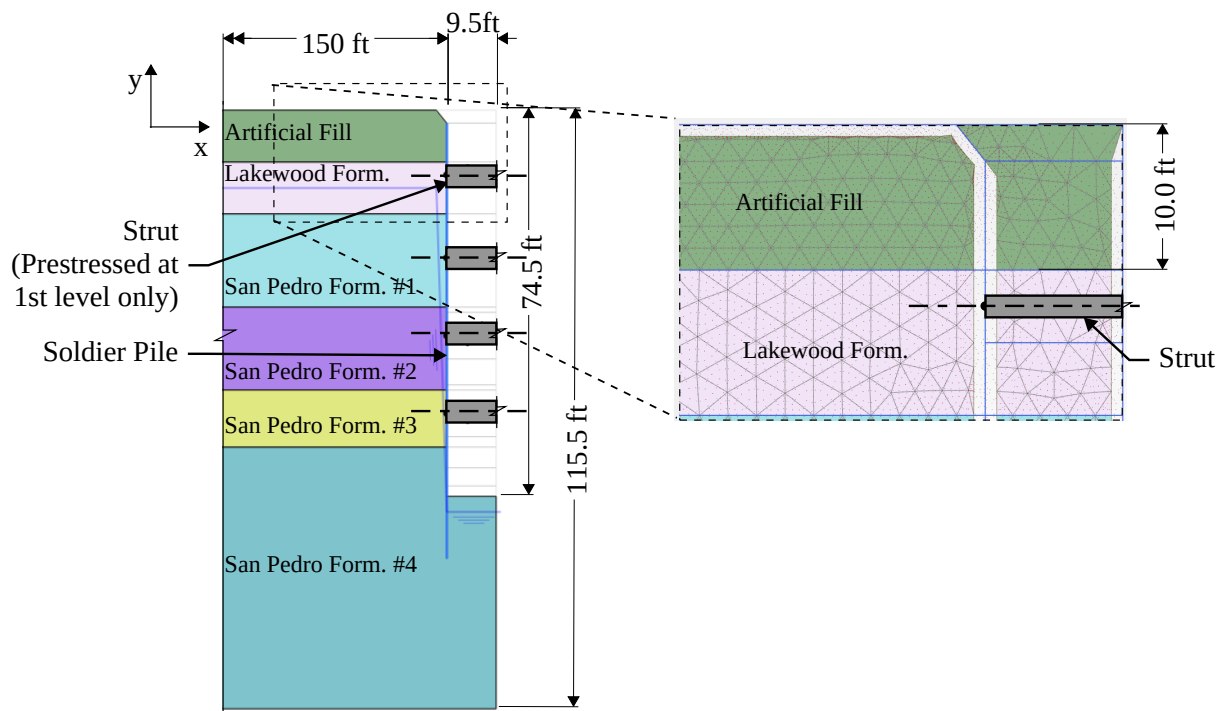


Figure 27: Numerical Model and Enlarged View Mesh in PLAXIS 2D

A portion of the soil mesh is shown in Figure 27 (right). The entire soil mesh is automatically generated within PLAXIS and consists of a total of 10,562 triangular elements with a maximum and minimum size of 3.35 feet and 0.50 feet respectively. The size of the elements is smallest near the soldier pile and begins to increase as the mesh approaches the boundaries. The average element is 2.0 feet in size and includes 15 nodes. Interfaces between soil layers and structure elements provide the option for defining permeability, as explained later. The continuity of mesh elements is preserved across soil interfaces, by aligning one leg of each element parallel to the boundary interface, so that the element nodes are shared to both sides of the interface.

3D Numerical Model

The 3D numerical simulation accounts for all structural members within the exploratory shaft and is grouped into elements such as beams, plates, and fixed anchors. The properties for each element are defined through their modulus of elasticity, moment of inertia, area, and weight. Figure 28 shows the components of the 3D model (see Figure 26 for plan view). Later in this chapter, the structural properties of all components are explained in detail.

The mesh of the 3D model consists of a total of 35,810 triangular elements with a maximum and minimum size of 77.37 feet and 0.005 feet respectively. The size of the elements is smallest near the structural components and begins to increase as the mesh approaches the model boundaries. The average element size is 12.48 feet and includes 10 nodes. Like the 2D simulation, interfaces between soil layers and structure elements provide the option for defining permeability. The continuity of mesh elements across soil interfaces is similarly established through a single, shared leg of an element that is located along the interface. The nodes of this element bridge the soil respective soil layers.

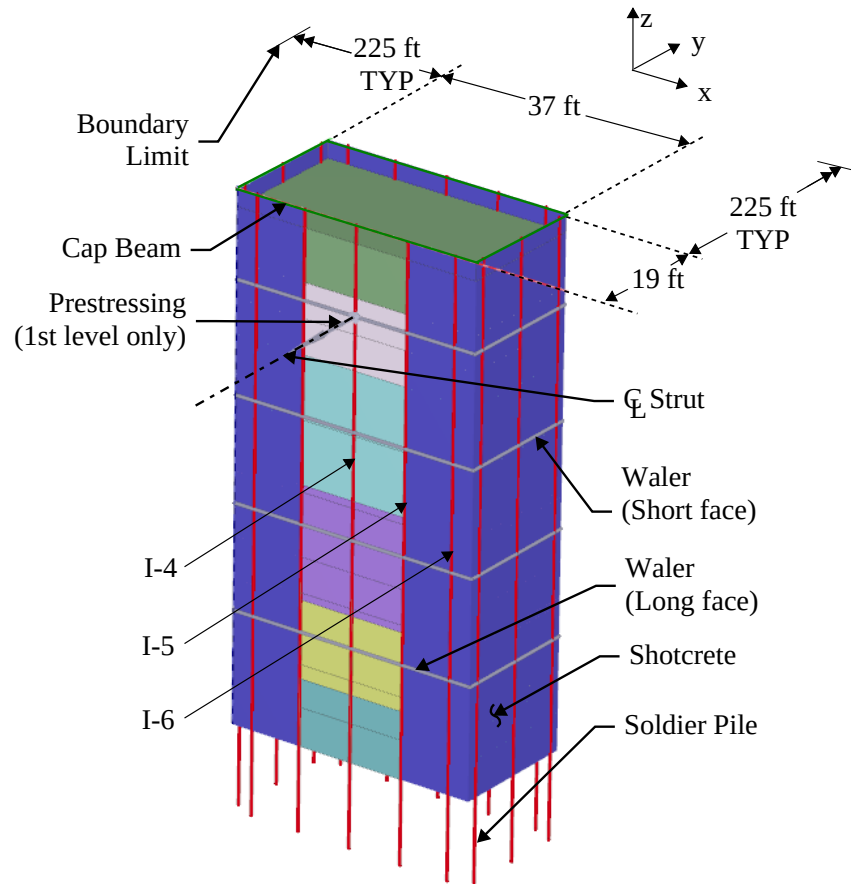


Figure 28: Numerical Model in PLAXIS 3D

The key limitations in the 3D model consist of properly modeling the stiffness of the connections between the various structural elements, especially at the system corners. For instance, the stiffeners and welds themselves cannot be properly modeled in PLAXIS as stand-alone elements and are only represented through fixed connections (typical for all connections). Another challenge arises in placing the walers at their centerline within the model. The in-situ shaft has small "stand-off" beam elements (depicted in Figure 29) that are present at all soldier piles. These 8-inch "stand-off" elements are welded between the soldier piles and walers. These beam elements (along with the depth of the waler) create an offset between the face of excavation and centerline of waler.

However, placing the waler elements at their centerline and incorporating these "stand-off" beams within the model results in an overestimation of system stiffness in the model for cross-sections I-4 and I-5. This configuration more accurately represents conditions along the I-6 cross-section, rather than the stiffness at the system centerline. Furthermore, the rigidity of the moment-resisting steel connection requires comprehensive modeling of a rotational spring to properly capture the stiffness. Since PLAXIS cannot model rotational springs, the "stand-off" beam elements are omitted and the waler elements are modeled within the same plane as the shotcrete (plate element) and soldier pile (beam element). As a result, the rigidity at the corners is underestimated, while the rigidity of along the center (i.e., along I-4 and I-5) are more accurately represented without further modeling of a rotational spring.

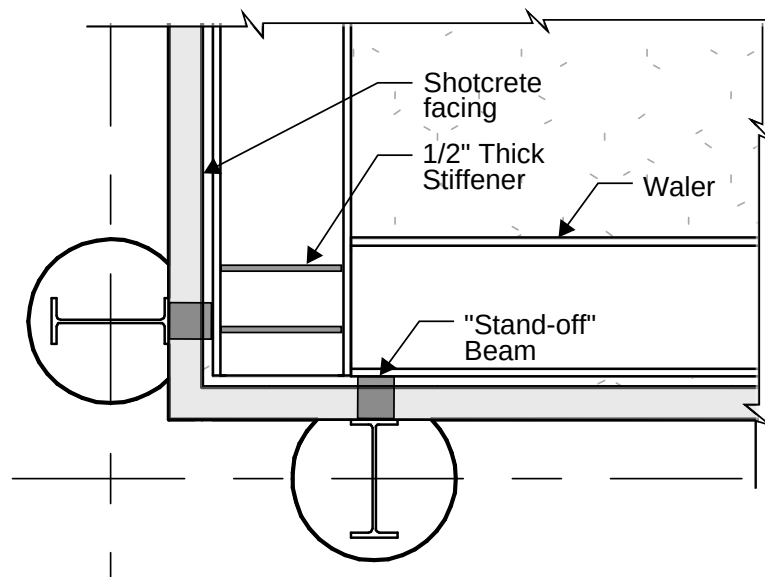


Figure 29: Stand-off Beam and Stiffener at Corner Connection

Modeling of Construction Sequence

The construction sequence is modeled numerically following the excavation sequence depicted in Figures 4 through 8. The initial stage represents the condition prior to excavation

construction, where only the soil layers are defined. In the following stage (Stage 1), all the soldier piles are defined along with any corresponding elements constructed at that excavation depth, namely the shotcrete facing. Each subsequent stage thereafter defines a new excavation depth along with updated structural elements (including struts and walers) and dewatering condition. All 14 construction stages outlined in Chapter 1 are modeled in PLAXIS. However, the primary focus is on the stages where numerical results can be directly compared with in-situ inclinometer measurements. Five stages are considered for this purpose. The global GWT is taken at 15 feet below the finish grade for every stage. Excavated soil layers are then manually assigned as dry at each stage as described in the Modeling of Ground Water Table and Dewatering section. Following the available construction plans, only the 1st level strut will be preloaded by 176 kips within the numerical models, all other levels maintain zero preloading throughout construction (Table 5). Preloading for the strut is applied only once within the construction stage that it's installed and omitted in subsequent stages. Note, the impacts of preloading will be investigated later within this chapter.

Table 5: Preloading at 1st Level

Strut Level	Construction Stage			
	1	2	3	4-14
1 st Level	-	-	176 kips	-

Modeling of Soil Conditions

The soil stratigraphy and soil material properties are identical for the 2D and 3D numerical models. Since the GWT is located at 15 feet below finish grade (at surface level), the soil layers are considered drained above the GWT and undrained below the GWT. The simulations also omit permeable interfaces between the soil layers and shotcrete plate elements, thus negating the influence of permeability parameters. Dewatering conditions are described in more detail in the ‘Modeling of Ground Water Table and Dewatering’ section. The following in-situ and laboratory tests were utilized to characterize the soil stratigraphy and derive the geotechnical input parameters: Cone Penetration Tests (CPTs), Seismic CPTs, Standard Penetration Tests (SPTs), consolidated undrained (CU) triaxial tests, and OYO suspension logging. In addition, boring log G-350, located 4 feet away from the shaft, and its geologic description, were used to assist with defining material parameters for the respective stratigraphy. Shear wave velocities, V_s , were obtained from Seismic CPT and subsequently used to determine shear modulus, G , and elastic modulus E per Equation 6 and Equation 7.

$$G_{vs} = \rho V_s^2 \quad (\text{Equation 6})$$

$$E_{vs} = 2(1 + \nu)G_{vs} \quad (\text{Equation 7})$$

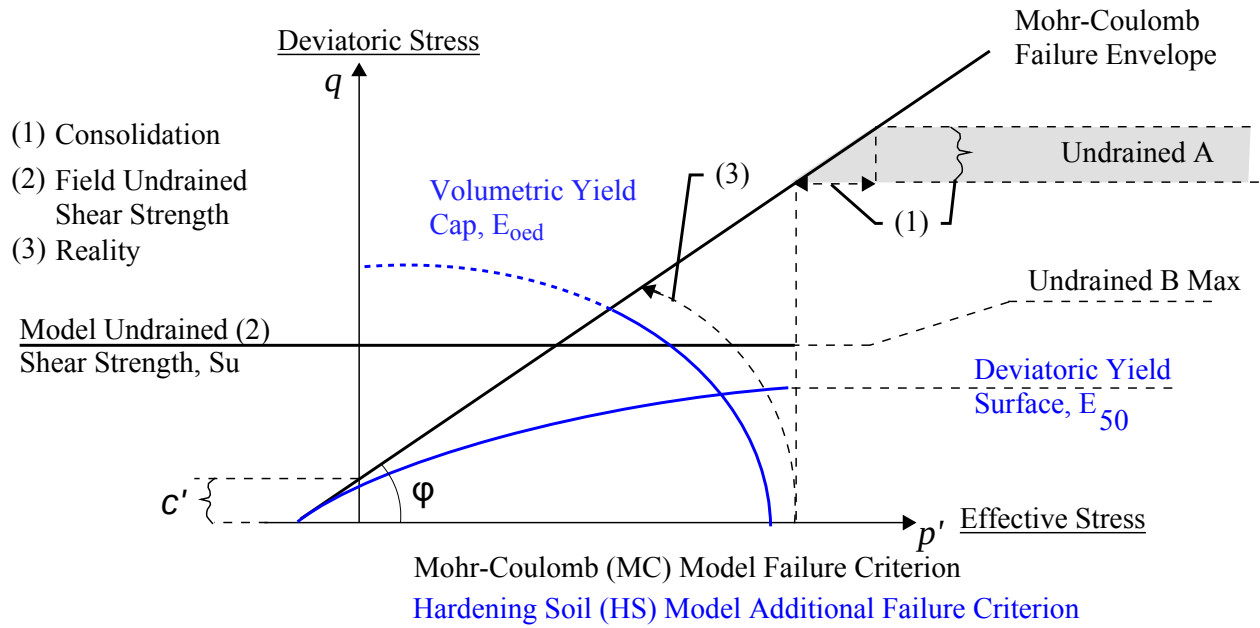
Where:

ρ = Soil density

ν = Poisson's ratio (taken as 0.3).

The undrained shear strength, S_u , cohesion, c , friction angle, ϕ , and HS model stiffnesses E_{50} , were determined from the CU triaxial test. The undrained shear strength was directly taken from the Mohr-Coulomb failure envelope provided for various soil samples subjected to a range of confining pressures corresponding to their sampling depths. Between borings G-350 and G-351, a total of 15 cylinders (3 samples/ soil layer) were tested by an external laboratory.

The triaxial loading stiffness or secant stiffness, known as E_{50} , is determined through the relationship of the deviator stress and axial strain. The deviator stress can be defined as the differential stress between the major and minor axes of the cylinder, i.e., $\sigma_1 - \sigma_3$. The value of E_{50} reflects the modulus of the cylinder at 50% of the maximum deviator stress, where maximum deviator stress is defined as the stress when failure of the cylinder occurs. Figure 30 provides a visual representation of the failure criterion between the Mohr-Coulomb and HS models. The relationship between the initial elastic modulus E , E_{50} , and E_{ur} is best summarized schematically in Figure 32 a & b [14].



As previously noted, both parameters, E_{50} and E_{ur} , represent stiffness of the soil derived from a stress-strain relationship of the CU triaxial test. Similarly, E_{oed} represents the stiffness derived from an oedometer test that measures the axial strain versus axial stress along a single axis (Figure 31). The value of E_{oed} is taken at the confining pressure, also referred to as the effective pressure. This effective pressure is equal to the pressure at which the E_{50} stiffness is observed within Figure 32(a). A typical relationship observed between the three parameters is defined within the numerical model as follows:

$$E_{oed} \cong E_{50} \quad (\text{Equation 8})$$

$$E_{ur} \cong 3E_{50} \quad (\text{Equation 9})$$

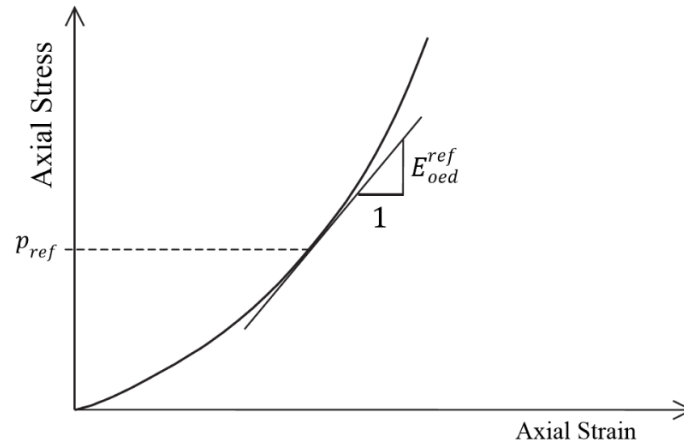


Figure 31: Schematic presentation of compression hardening and the derivation of axial oedometer stiffness E_{oed}

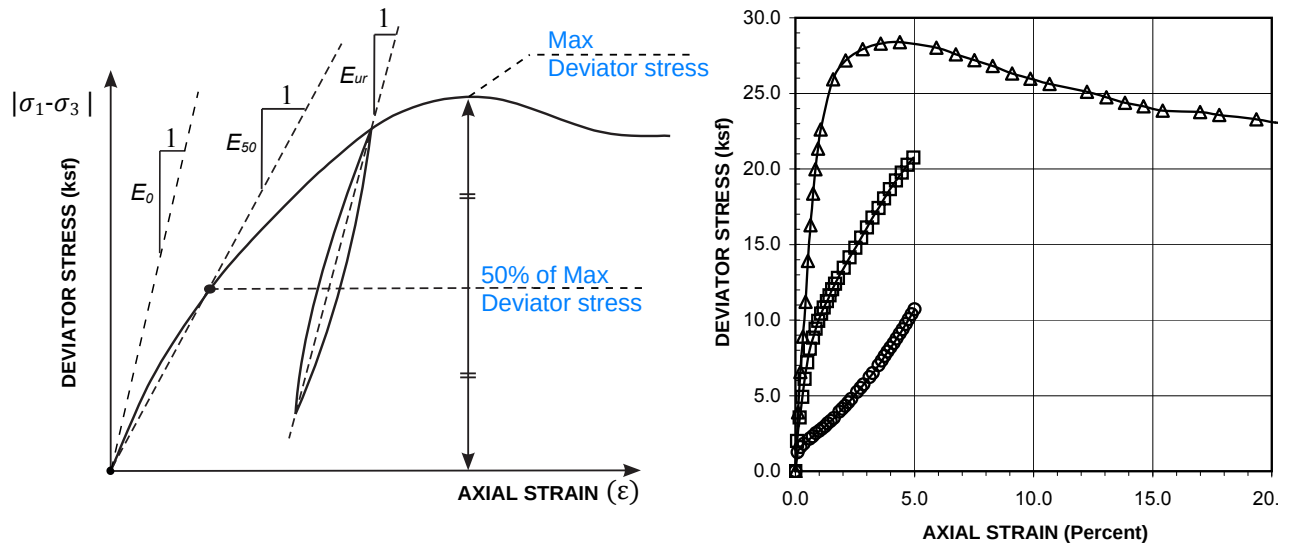


Figure 32: Stress vs Strain relationship (a) Schematic illustration of E-moduli obtained from a stress strain curve, Source: PLAXIS Materials Model Manual [14]; (b) Stress strain measurements obtained from triaxial soil testing using samples from borehole G-350

Figure 33 summarizes the input parameters required for all four soil models used in the numerical analysis of this thesis. Poisson's ratio is taken as 0.30 for all six layers and for all four soil models. The linear elastic (LE) model defines its elastic stiffness parameters based on Hooke's Law, utilizing the initial elastic modulus, E , and Poisson's ratio, ν , as the primary means to calculate deformations. The MC model further defines the strength of the soil through the friction angle ϕ , cohesion c' , and dilatancy angle ψ . The Mohr-Coulomb model employs a failure criterion for the stress-strain relationship which recognizes plasticity. To better characterize the soil stiffness within the constitutive material model, the HS model integrates the secant stiffness E_{50} , the oedometer stiffness E_{oed} , and the unloading/reloading stiffness E_{ur} , as shown in Figure 32. The HS_{small} model utilizes a strain dependent shear modulus and applies two additional parameters, $\gamma_{0.7}$ and the corresponding $G_{0.7}$. The $\gamma_{0.7}$ is derived from modulus reduction curves [15], and $G_{0.7}$ is taken as 70% of G_{vs} . The primary benefit of the HS/HS_{small} models is a more accurate characterization of the soil based on strain and stiffness parameters, in addition to stress parameters, as done by the MC model.

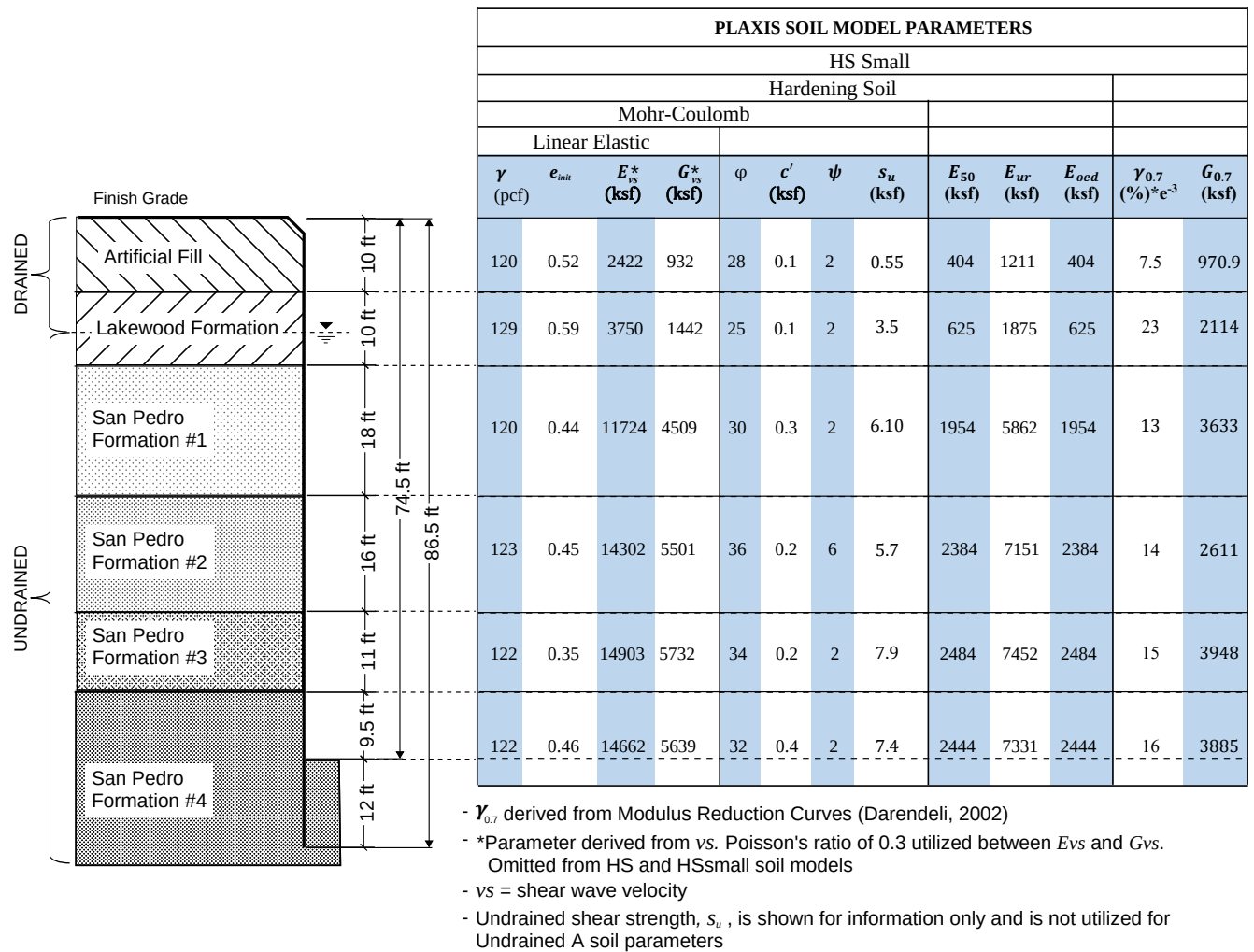


Figure 33: Model Parameters for the four soil models considered in the PLAXIS model

Modeling of Ground Water Table and Dewatering

Due to caving of the dewatering wells and the variation in groundwater table measured during the excavation, the groundwater level is conservatively modeled at 15-feet below finish grade (i.e., finished surface). The geocomposite drainage system installed behind the walls is omitted within the models, therefore the influence of groundwater on the excavation face is defined through the dewatering approach described below (Figure 34). Groundwater table measurements

indicate relatively steady levels; major drops are caused by inflows from caving. For the numerical model however, the GWT level remains 15-feet below finish grade while the dewatering near the excavated faces is user-defined for each stage. While all layers are defined as undrained, the top layer, Artificial Fill, and 5-feet into the next layer, Lakewood Formation, are manually assigned as dry within PLAXIS; this condition effectively analyzes the assigned layers as drained. The remaining layers are then subsequently assigned either dry, interpolated, or submerged- i.e., subjected to the GWT. The assignment of the above-mentioned dewatering conditions (dry, interpolated, submerged) within the exploratory shaft varies with construction staging and is exemplarily defined in Figure 34 for three separate stages. All other stages not shown in Figure 33 follow an identical dewatering approach. The interpolated layers observe a linear interpolation of the hydrostatic pressures between the adjacent drained layer and values generated internally by PLAXIS within the submerged layer.

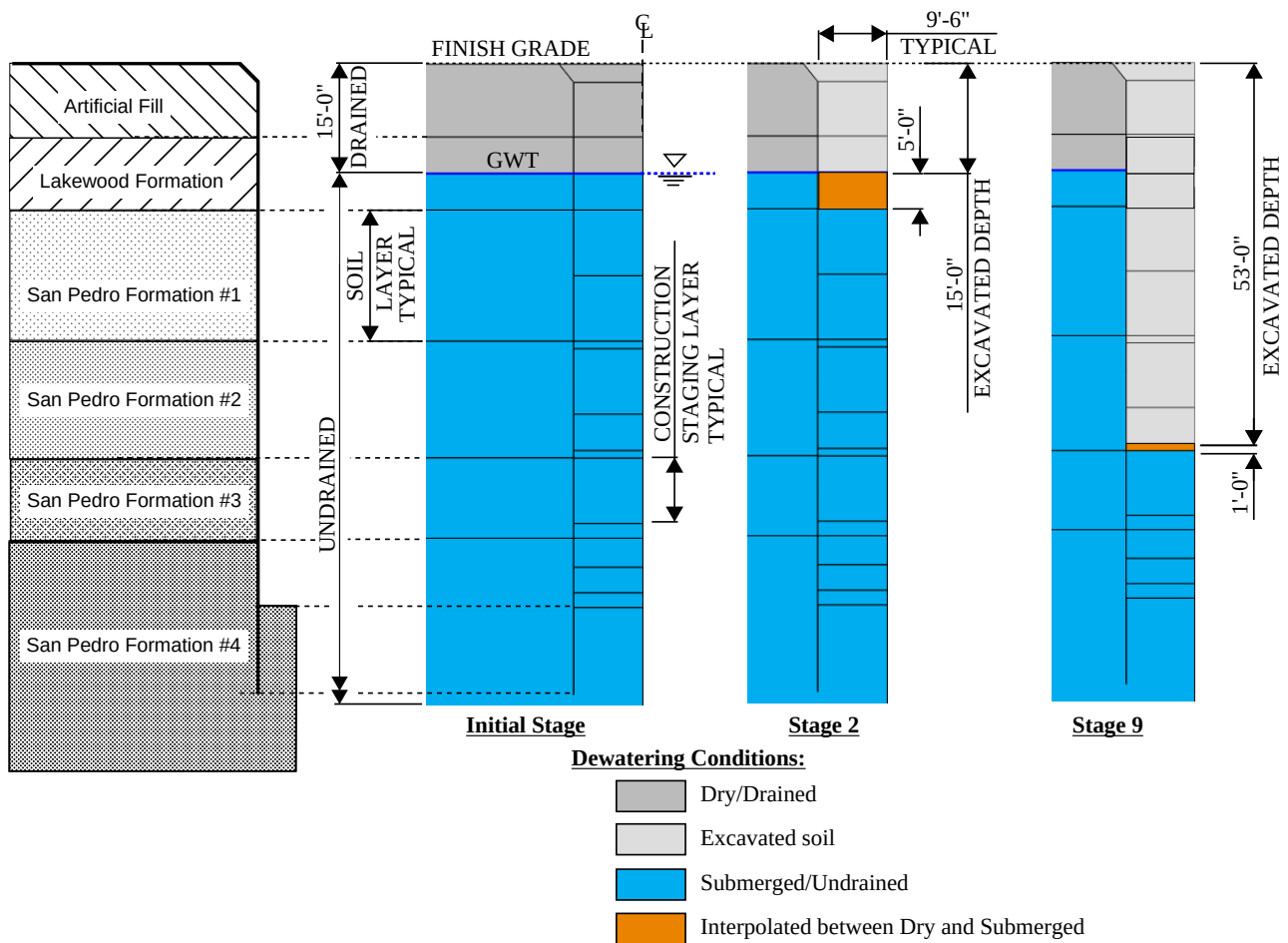


Figure 34: GWT and Dewatering Approach for 2D (Initial Stage, Stage 2, and Stage 9)

Modeling of Drainage Conditions

In addition to defining the GWT, PLAXIS recognizes different soil drainage types. The drainage type 'Undrained A' utilizes effective strength parameters, i.e., friction angle ϕ' , cohesion c' , and dilatancy Ψ' , to calculate the undrained shear strength S_u . The 'Undrained B' type of analysis allows for direct input of the undrained shear strength S_u and therefore omits the effective strength parameters and an internal calculation thereof. PLAXIS recognizes the limitations of Undrained A by correlating its dependency on the effective load path that is not always correctly

generated within the model. Both drainage types are applicable to the MC, HS, and HS_{small} soil models, however the HS and HS_{small} models are adversely affected when applying Undrained B. The S_u provided from CU triaxial tests can be directly entered within Undrained B. However, compression hardening, represented by the oedometer stress E_{oed} , is no longer observed within the model for Undrained B, in addition to the model omitting the stress-dependency of the stiffness. This results in smaller vertical displacements relative to Undrained A, which establishes a boundary for E_{oed} . A comparison between the two drainage types (A and B) will be investigated for the 2D model, both, using HS and HS_{small}, to assess and compare the mobilized shear strength in the two analysis methods. Drainage type “Undrained A” will then be utilized for all subsequent 3D models due to the limitations described for Undrained B.

Modeling of Structural Components

Soldier Piles

Table 6 lists the structural and geometric properties of the soldier piles as modeled in 2D and 3D.

The subsequent text describes how these properties were obtained for the respective model.

Table 6: Soldier Pile Properties in PLAXIS

Model Type	2D	3D	Units
Location	Cross section at I-4	perimeter (all levels)	-
Model Element	Plate	Beam	-
Element Length	84.5	84.5	ft
Elastic Modulus (E)	-	4.176E+06	kip/ft ²
Moment of Inertia (I)	-	0.09983	ft ⁴
Area (A)	-	0.1896	ft ²
Effective thickness (d_{eq})	2.514	-	ft
Bending Stiffness (EI)	5.21E+04	-	kip*ft ²
Axial Stiffness (EA)	9.90E+04	-	kip
Weight (w)	0.0116	-	kip/ft ²
Unit Weight (γ)	-	0.491	kip/ft ³

2D

In the 2D model, the steel sections (W-shapes) of the soldier piles are modeled as an equivalent plate element (Fig. 34) with its material stiffness distributed over a length of 8 feet (i.e., corresponding in-situ spacing of the soldier piles). The effective thickness of the plate is based on the ratio of bending stiffness to axial stiffness and is defined within PLAXIS through the following equation:

$$d_{eq} = \sqrt{12EI/EA} \quad (\text{Equation 10})$$

Where:

E = Elastic modulus of a single steel soldier pile

I = Area moment of inertia of a single steel soldier pile

A = Area of a single steel soldier pile

Properties for the plate element are defined per linear foot. Figure 35 (bottom), highlights the W-section of the structural system that is converted into the equivalent plate. Only the steel sections are considered in defining the plate's structural properties. The shotcrete facing and lean/structural concrete within the 24-inch diameter CIDH backfill are omitted due to the negligible bending stiffness of the concrete in comparison to the structural steel. The influence of connections between the struts, walers, and soldier piles will be investigated within the 3D simulation since connections within 2D cannot be properly defined. While 2D considers fixed connections between plates and fixed-end anchors (struts), walers that exist outside of the 2D section cannot be modeled. Therefore, the influence of walers and their fixed connections are not adequately represented and the system stiffness in 2D is lower relative to the 3D model. The 3D numerical model recognizes boundary connections of the plates and calculates the load path through their connections to the soldier piles and walers to determine their influence on the deformations.

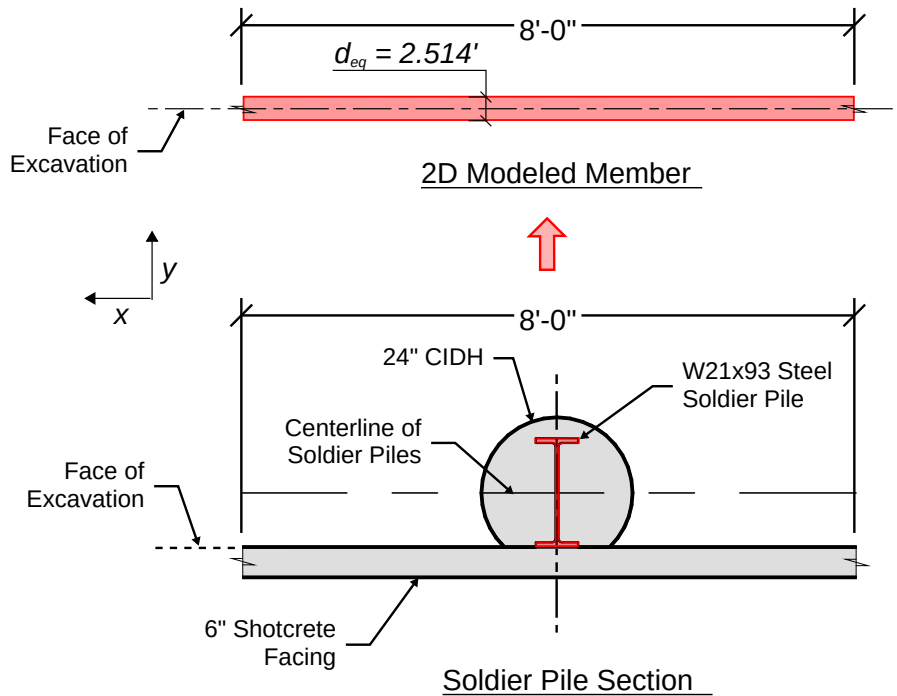


Figure 35: Simplified plate for 2D analysis

3D

The 3D model represents soldier piles as “Beam” elements rather than “Plate” elements. This choice is based on how PLAXIS recognizes these elements. A significant advantage of employing “Beam” elements, rather than “Embedded Beam” elements, is their ability to establish connections at any point along the member, a feature that is especially crucial for the shaft. Note, “Embedded Beam” elements only provide connections at the ends of the element, resulting in discontinuities within the model where fixity should be occurring between the beams, plates, struts, and walers. The beam elements are defined through area, moment area of inertia (weak and strong axis), elastic moduli, and unit weights. Note, parameters for plasticity of the beam can be provided, however these are omitted due strains being well within the elastic region of the steel.

Struts and Walers

The structural properties of the struts and walers are summarized in Tables 7 and 8. Walers are only modeled in PLAXIS 3D, as these members exist outside of and/or are perpendicular to the 2D-plane section and cannot be modeled in PLAXIS 2D.

Table 7: Waler Properties in PLAXIS

Model Type	3D [long face]	3D (short face)	Units
Location	W24x192 – Level 1 [W24x176] – remaining	W24x176 – all levels	-
Model Element	Beam	Beam	-
Elastic Modulus (E)	4.176E+06	4.176E+06	kip/ft ²
Moment of Inertia (I)	0.02556 [0.0231]	0.0231	ft ⁴
Area (A)	0.3910 [0.3590]	0.3590	ft ²
Unit Weight (γ)	0.491	0.491	kip/ft ³

Table 8: Strut Properties in PLAXIS

Model Type	2D	3D	3D	Units
Location	W14x176 – Level 1 [W14x159] – remaining	*W14x176 – Level 1 [W14x159] – remaining	*W14x176 Level 1	-
Model Element	Fixed-end anchor	Beam	Fixed-end anchor	-
Elastic Modulus (E)	4.176E+06	4.176E+06	-	kip/ft ²
Moment of Inertia (I)	-	0.10320 [0.09163]	-	ft ⁴
Area (A)	-	*0.17985 [0.3243]	-	ft ²
Length Spacing (L_{sp})	16.78	-	-	ft
Axial Stiffness (EA)	1.502E6 [1.354E+06]	-	*7.511E+05	kip
Unit Weight (γ)	-	0.491	-	kip/ft ³

* 1st Level strut shares the stiffness between the fixed-end anchor and beam element

2D modeling of Struts

Each strut level is represented by a single fixed-end anchor (Figure 36) and only requires the following inputs from the strut: axial stiffness, anchor spacing, equivalent length, and preloading (can be varied with construction stage). Fixed-end anchors are defined as point elements and exhibit one single degree of freedom in the direction of the element's length. The equivalent length of fixed-end anchors is defined by the location of zero strain within the strut, typically considered to be at the center of the strut. For the exploratory shaft, the equivalent length is taken as half of the strut length which is equal to half the width of excavation (9.50 ft). The on-center spacing of the

fixed-end anchors is taken as 16.78 feet and represents the as-built spacing between the strut and the adjacent walers that are symmetric about the strut. Preloading of the fixed-end anchor is applied in the same stage as the installation of the strut and is omitted thereafter. As previously noted, preloading was not well defined in available construction documentation, however its influence will be investigated within the numerical models, primarily for the strut located in level one, which was documented to have a preload of 176 kips. The waler elements themselves are omitted from the 2D simulation due to their location and direction relative to the 2D plane modeled.

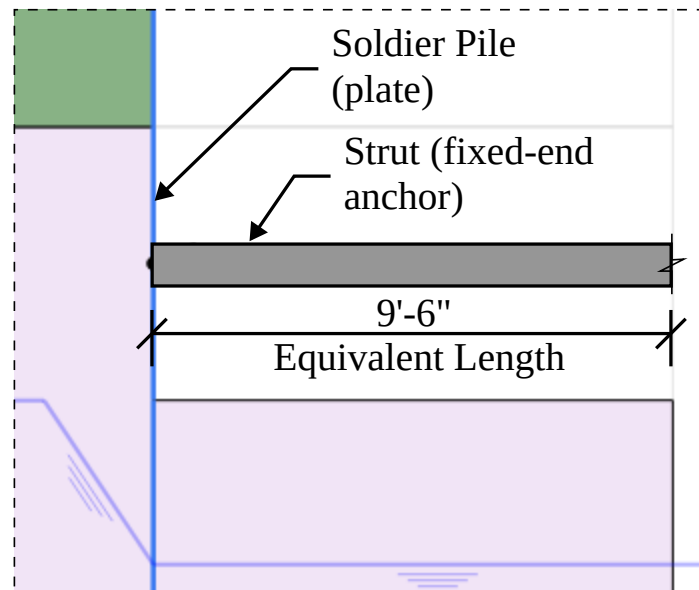


Figure 36: Soldier Pile and Strut in 2D Model

3D modeling of struts and walers

In the 3D model, the struts and walers (Figure 36) are represented as beam elements that are independently connected between each soldier pile. PLAXIS automatically defines fully fixed connections at the nodes where different elements meet. These fixed connections occur at the ends of the strut elements and the soldier piles, reflecting the welded connections present among all structural members. For the 1st level strut, the beam (strut) element is coupled with a fixed-end anchor at both ends. Similar to the 2D model, the fixed-end anchors are applied as points and provide supplemental axial stiffness. The anchors also allow an input for preloading, which is not permitted through beam elements alone. Since the stiffness of fixed-end anchors cannot be zero, the axial stiffness of the strut is divided between the fixed-end anchor and the beam strut element (50/50) to establish the effective stiffness of a single strut. The direction of the fixed-end anchors' equivalent length is taken relative to the face of excavation. Magnitude of preloading is entered as 176 kips for both fixed end-anchors. Verification of the direction of preloading can be observed through viewing the numerical meshing of the shaft within PLAXIS.

As previously noted, there are limits to modeling the rigidity of the corner connections. Typical structural analysis programs allow for (1) rigid links that effectively negate deflections and rotations at specified locations where rigid connections occur, or (2) define rotational springs that describe the rigidity of the connection. Without these rigid links and/or springs, PLAXIS observes displacements and rotations in the walers near the corner soldier pile (I-6) that will result in a potential overestimation of displacements compared to field measurements (I-6).

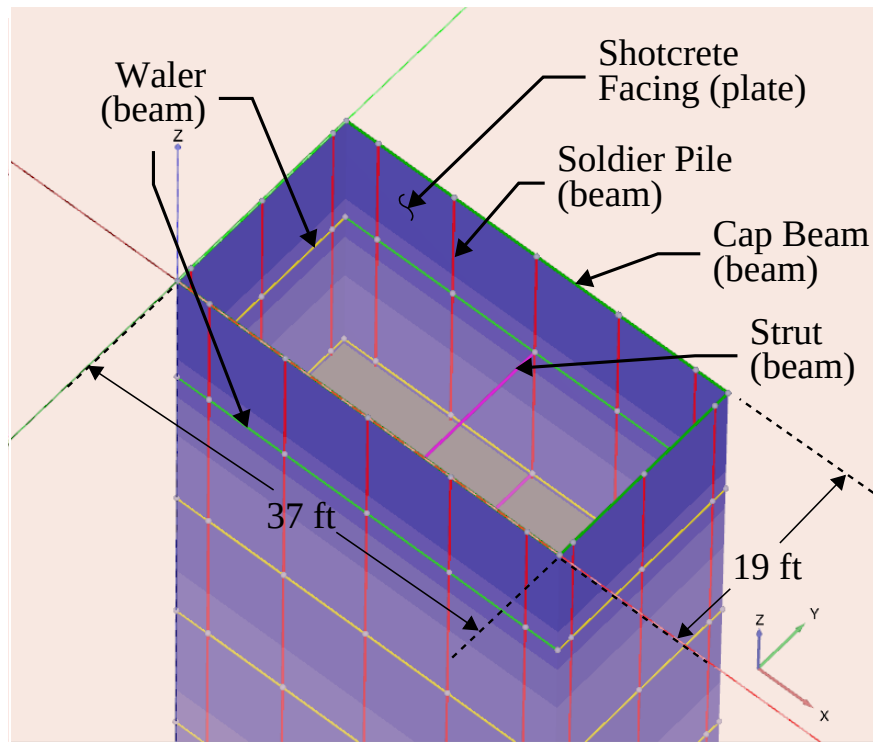


Figure 37: Soldier Pile and Strut in 3D Model with respective elements (in parentheses)

Shotcrete Facing

The properties of the shotcrete facing are described in Table 9. As explained prior, shotcrete is not modeled in PLAXIS 2D; only parameters relevant for the 3D model are included in Table 9.

Table 9: Shotcrete Properties in PLAXIS

Model Type	3D (long face)	Unit
Location	All perimeter (0.70)	ksi
Model Element	Beam	-
Elastic Modulus (E)	2.310+05	kip/ft ²
Thickness (d)	0.5	ft
Area (A)	0.3910 [0.3590]	ft ²
Density (γ)	0.491	kip/ft ³

In the 3D model, the shotcrete facing is simulated as a plate element (Figure 37). During construction, the shotcrete facing utilized welded wire mesh in a six-inch-thick section to limit deflections of the facing. The concrete compressive strength after 12 hours of placement was recorded to be 0.7 ksi. The concrete strength reached an average of 4.2 ksi after ~72 hours and the 28-day concrete compressive strength (f'_c) of the concrete measured 5.0 ksi. Much of the deformation recorded in the shotcrete wall occurred before the 28-day curation period, with some measurements occurring on the same day as excavated lifts were removed and shotcrete was placed. Since not all deformation occurs immediately after excavation, some variance in the stiffness and deflection of the shotcrete facing is expected. As such, the average strength between the 12 and 72-hour intervals (2.45 ksi) is utilized within the 3D model. The connections between the plates are considered fixed in four-degrees of freedom: three translational directions and one rotation about

the longitudinal axis. Note, a custom connection was originally applied between the plates to simulate free rotation about the soldier piles, effectively preventing moments from transferring between plates along the longitudinal axis of the piles. However, due to the number of connections occurring in a single geometric point/line in space, errors in meshing arose and connections were subsequently omitted from the analysis. This omission does not affect displacements or settlements, only the magnitude of moments and shear within the plates themselves.

Cap Beam

The cap beam (as shown in the photograph of Figure 12) is only modeled in the 3D model with properties described in Table 10. The cap beam was placed after the installation of the soldier piles and before the 1st construction stage.

Table 10: Cap Beam Properties in PLAXIS

Model Type	3D	Units
Location	Top of soldier piles	-
Model Element	Beam	-
Elastic Modulus (E)	4.176E+06	[kip/ft ²]
Moment of Inertia (I)	0.014660	[ft ⁴]
Area (A)	0.14583	[ft ²]
Density (γ)	0.491	[kip/ft ³]

The cap beam consists of an HSS 10"x10"x5/8" rectangular section welded to the top of each soldier pile flange and web. The cap beam spans the long and short face of the exploratory shaft. The cap beam is omitted in the 2D analysis, as the element is in the perpendicular plane to the 2D section.

Numerical Results

2D – Lateral Deformations

The 2D numerical predictions for the lateral deformations obtained with all four soil models (i.e., (1) Linear Elastic, (2) Mohr- Coulomb, (3) Hardening Strain, and (4) Hardening with Small-strain) are depicted in Figure 38. All results shown are obtained using the analysis type “Undrained A”. Figure 38 also shows the elevations of the struts (dashed lines). The solid black lines for models HS and HS_{small} provide a comparison with in-situ measurements and represent the maximum lateral deflections recorded at inclinometers I-4 and I-5 at the maximum depth of excavation.

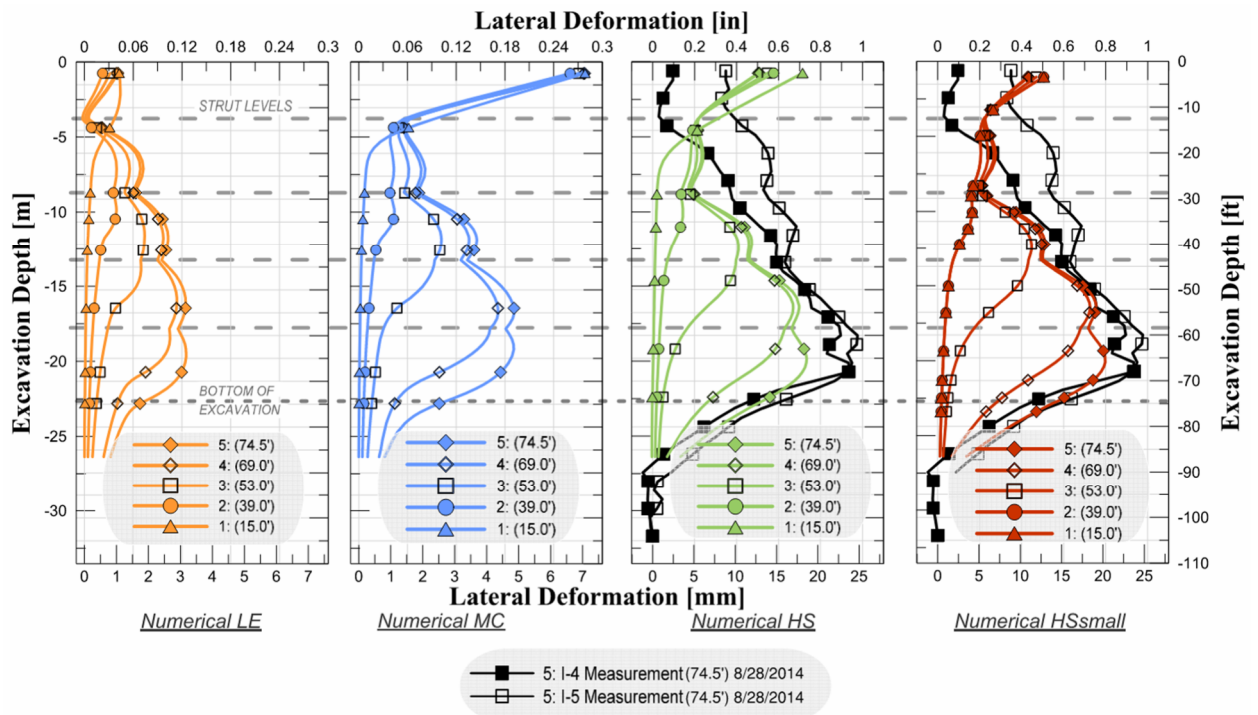


Figure 38: 2D Numerical Model LE, MC, HS, and HS_{small} for Undrained A, Compared to I-4 and I-5 Inclinometer Measurements

The maximum displacements for the LE, MC, HS, and HS_{small} soil models are shown in Table 11.

The following observations can be drawn for the LE and MC models from Figure 38 and Table 11.

- The LE model shows the peak deflection at a depth of around 70 feet, while the MC model shows a peak at the top of the soldier pile. The HS and HS_{small} models provide a assimilation, in shape, depth and magnitude of the max measured deflections. Both models predict the max lateral deflections to occur at around 65 feet of depth. The in-situ maximum displacements for I-4 and I-5 occurred between 62-68 feet.
- The large displacement observed at the top of the soldier pile for the MC model (as well as the HS and HS_{small} models) are attributed to exceeding the soil strength in the upper Artificial Fill layer prior to the installation of the first strut (Stage 2). Note, the displacement at the top of the pile reduces by a negligible amount as the excavation depth increases.

Pile displacements below the depth of 15 feet for the LE and MC models suggest magnitudes within 50% of each other for all five stages, however, both magnitudes of lateral displacement underestimate the field measurements by a factor of 5-7. The significant underestimation observed in the LE and MC models arises from their limitation by representing stiffness solely with a single, constant modulus E_{vs} . This modulus, (E_{vs}), derived from an in-situ shear wave velocity test near the shaft is 7–15 times larger than E_{50} , which is obtained from laboratory tests and relates to stress at failure. The larger stiffness yields much smaller deformations.

The availability of relevant input parameters is the primary limiting factor for both types of analysis. Additionally, the use of linear-elastic, perfectly-plastic strength parameters based on the Mohr-

Coulomb failure criterion does not significantly constrain the soil response—except near the top of the soldier pile, where the Artificial Fill layer is present. Below a depth of 12.5 feet, this limitation is largely irrelevant. The internal stresses and strains mobilized during excavation are low, and all stress levels remain below the threshold needed to trigger plasticity or increased displacement. As a result, these strength parameters have minimal influence on the exploratory shaft behavior below 12.5 feet and are not expected to meaningfully affect the analysis outcomes.

Table 11: 2D Numerical Results for Undrained A

Measurement Date	Excavation Depth [ft]	2D Numerical Max Lateral Deformation for the Following Dates in Comparison with In-situ data from I4 & I5					
		LE_A* [inch]	MC_A* [inch]	HS_A* [inch]	HS _{small} _A* [inch]	I-4 [inch]	I-5 [inch]
10/18/2013	15	0.04	0.05	0.21	0.20	0.16	0.19
12/18/2013	39	0.04	0.04	0.19	0.21	0.48	0.43
01/28/2014	53	0.07	0.10	0.41	0.45	0.61	0.54
03/12/2014	69	0.11	0.17	0.67	0.73	0.88	0.86
08/28/2014	74.5	0.13	0.19	0.73	0.80	0.95	1.0

*Considers numerical displacements below depth of strut (12.5 feet) for comparison of the pile behavior

The HS_{small} model provides the closest prediction to the inclinometer measurements in the field, showing results within 16% to 20% of the maximum displacements measured at I-4 and I-5, respectively. This closer prediction can be attributed to the fact that the Mohr-Coulomb failure envelope is reduced within the HS/HS_{small} model by considering shear and compression hardening that induces irreversible strains within the soil medium.

The 2D HS_{small} model suggests approximately 10% larger lateral deflections compared to the HS model. The larger lateral displacement predictions relative to the HS model can be attributed to the small strain parameters. By refining the correlation between the shear modulus and strain, the stiffness is effectively decreased compared to the HS model for any given strain. Figure 39 shows

test results of LA tar sands correlating shear modulus and shear strain determined from resonant column and torsion shear testing (RCTS), as documented in the field report (Parson's Brinkerhoff). Since RCTS was not performed on soil samples adjacent to the exploratory shaft, modulus reduction curves (Darendeli, 2001) provide an alternative for determining the shear strain $\gamma_{0.7}$ that corresponds to 70% of G_{vs} ($G_{0.7}$). The increase in lateral deformation from HS to HS_{small} suggests that the numerical model is calculating larger shear strains than the provided input parameter, $\gamma_{0.7}$, and is thus exhibiting a softer soil response as a result. HS_{small}'s numerical results are small, and were regarded negligible for these simulations.

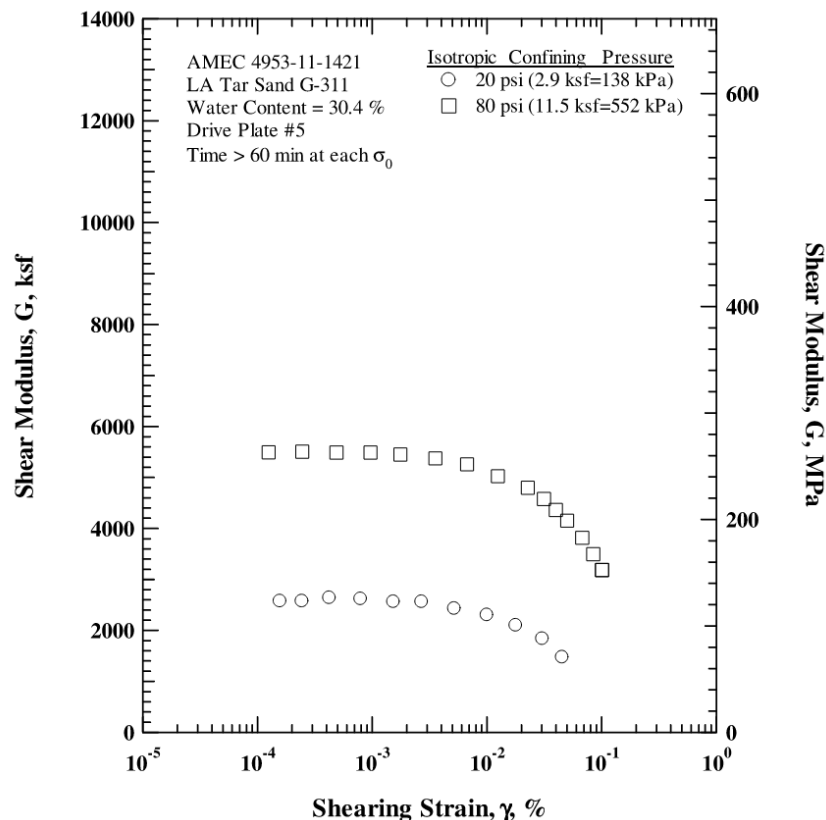


Figure 39: Resonant Column and Torsion Shear Testing Results for G-311 Soil Sample
(Source Report: Parson's Brinkerhoff ([1])

Comparison of Drainage types within the 2D analysis: Undrained A vs Undrained B

As previously mentioned, a comparison of drainage types Undrained A and Undrained B is conducted to verify the influence on the mobilized shear strength. Table 12 and Figure 40 present a comparison of lateral deformations obtained with both analysis types in 2D: Undrained A and Undrained B for HS, and HS_{small} soil models. Maximum displacements obtained with Undrained A are substantially larger than their Undrained B counterpart, i.e. by 52%, and 74% for HS and HS_{small} respectively. The observed increase in displacements for the Undrained A models is expected, as the drainage model calculates lower undrained shear strengths, which would typically result in larger displacements.

Table 12: 2D Numerical Results for Undrained A and B for HS and HS_{small}

Measurement Date	Excavation Depth [ft]	2D Numerical Max Lateral Deformation for the Following Dates in Comparison with In-situ data from I4 & I5					
		HS_B* [inch]	HS_A* [inch]	HS _{small} _B* [inch]	HS _{small} _A* [inch]	I-4 [inch]	I-5 [inch]
10/18/2013	15	0.17	0.21	0.11	0.20	0.16	0.19
12/18/2013	39	0.20	0.19	0.18	0.21	0.48	0.43
01/28/2014	53	0.32	0.41	0.30	0.45	0.61	0.54
03/12/2014	69	0.44	0.67	0.42	0.73	0.88	0.86
08/28/2014	74.5	0.48	0.73	0.46	0.80	0.95	1.0

*Considers numerical displacements below 15 feet depth for more direct comparison

This contrast is further supported by Figure 40 which plots the shear strength versus effective stress for two separate layers, the Lakewood Formation and San Pedro Formation #2. For both layers, the

calculated shear strength utilizing effective strength parameters (Undrained A) is smaller than the undrained shear strength determined from the CU triaxial test (Undrained B).

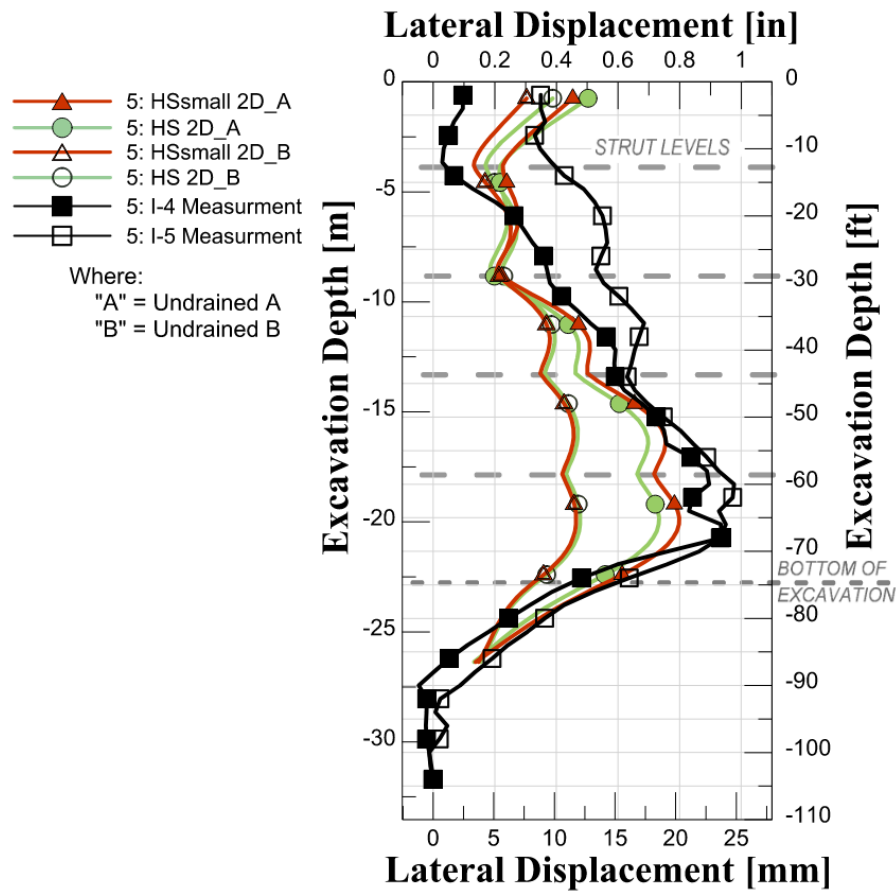


Figure 40: A Comparison of Numerical Lateral Displacement Results for HS and HS_{small} in 2D using Undrained A (solid symbols), vs. Undrained B (empty symbols) and in-situ measurements from I-4 and I-5 Inclinometers.

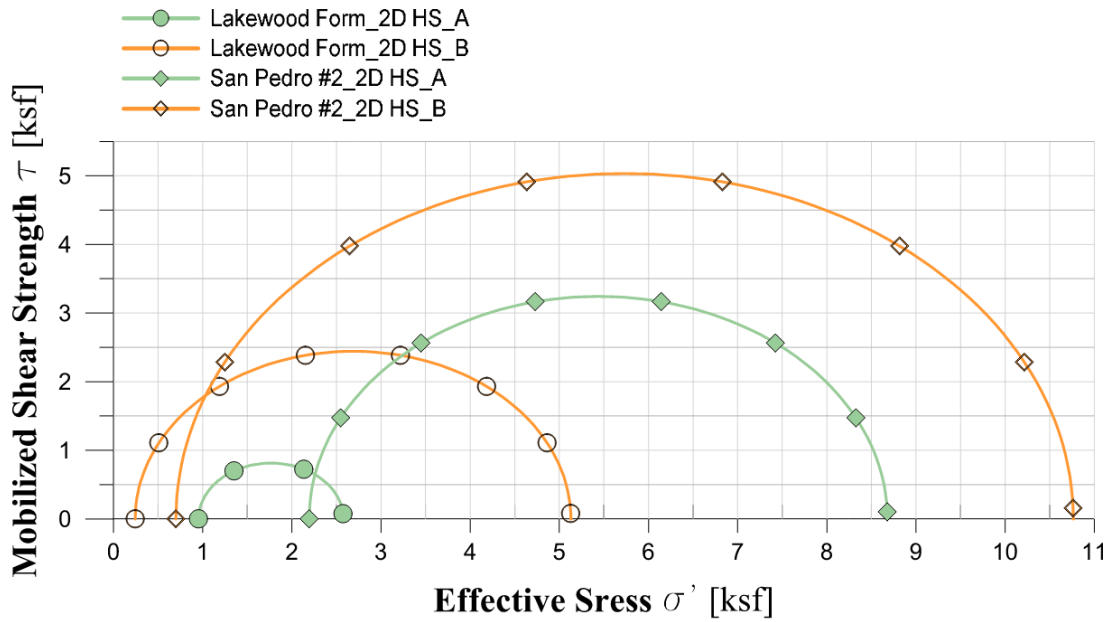


Figure 41: Shear Strength versus Effective Stress for Undrained A and B for HS Soil Layers

As previously discussed in the ‘Modeling of Drainage Conditions’ Section, there are various limitations to utilizing Undrained B, especially when displacements are driven by stiffness and strain versus the strength failure criterion. From a numerical standpoint, the Undrained B type omits stress-dependency and compression hardening, thus limiting the capabilities of the HS model to capture strain-dependent deformations. The presence of tar-sands further increases the variability of undrained shear strengths determined from CU triaxial samples. The primary differences between the two drainage types are highlighted in the following Table 13 for the HS models. Due to these results and limitations, Undrained A will be utilized for all subsequent models.

Table 13: Soil Parameters Utilized for Undrained A and B for HS Soil Model

Soil Parameters											
Drainage Type	ϕ	c' (ksf)	ψ	E (ksf)	G (ksf)	s_u (ksf)	E_{50} (ksf)	E_{ur} (ksf)	E_{oed} (ksf)	m	K_o
Undrained A_HS	x	x	x	-	-	-	x	x	x	0.5	c
Undrained B_HS	-	-	-	-	-	x	x	x	-	0.5	1.0

Where:

- m = power coefficient for stress-dependent stiffness
- K_o = lateral earth pressure coefficient
- x = input parameter – varies with layer and is identical between drainage types unless noted otherwise
- c = internally calculated parameter – varies with depth $[1 - \sin(\phi)]$
- = parameter omitted

The power coefficient for stress-dependent stiffness, m , did not have any influence on mobilized shear strengths or deflections for either drainage type.

Modeling of strut preloading

The sensitivity of the 2D model to strut preloading was investigated by varying the application of a 176-kip preload. Although this load was not consistently documented during construction, as-built plans indicate that preloading was applied to the first-level struts. To evaluate its impact, the HS and HS_{small} models were run, and three numerical scenarios were considered: (1) a 176-kip preload applied solely to the first-level strut, (2) a 176-kip preload applied to struts on levels one

through four, and (3) no preload applied to any strut. Numerical results, shown in Figure 42, indicate that all three preloading configurations result in a similar overall deformation shape. Note, the results for “3-no preload” are nearly identical to “1-preload at 1st strut” and are omitted from Figure 42 for clarity. The models considering all strut levels to be preloaded reduce the maximum deformation by 6% compared to the model with no preload. A 6% reduction is not consistent with field measurements at other braced cuts. Preloading of the strut typically tightens the connections to the waler and stand-off beams and system response increases nonlinearly as preloading increases. This is an indication that the preloading and/or stiffness of the structural connections are not captured adequately by the numerical model. While there is negligible variation in maximum deformation between preloading conditions below the first strut level (12.5 feet depth), there is a considerable change in the shape above the location of the first level. Therefore, the preloading conditions will consider preloading at the first strut only for the subsequent 3D model analysis.

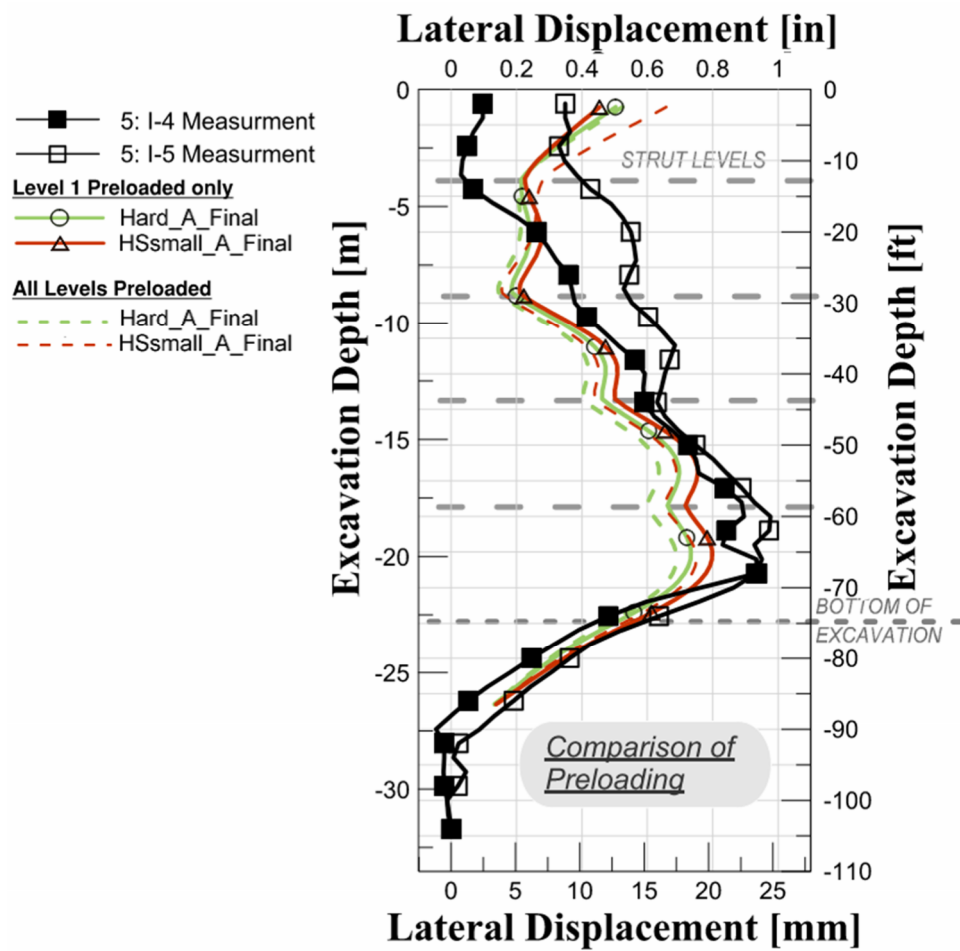


Figure 42: 2D Preloading Lateral Displacement Results for both HS Model Drainage types, Compared to I-4 and I-5 Inclinometer Measurements

2D – Vertical Deformations (Surface Settlements)

Vertical deformations (surface settlements) were extracted from the model and plotted in Figure 43. The surface settlement magnitudes correspond to the time step when the maximum excavation depth H (74.5 feet) was reached. Figure 43 shows the comparison between the numerical model results and measurements obtained from in-situ settlement markers. The maximum settlements for the LE, MC, HS, and HS_{small} models are provided in Table 14, along with the location of the settlement relative to the excavated face. Location of maximum settlements align with the location observed from the field measurements.

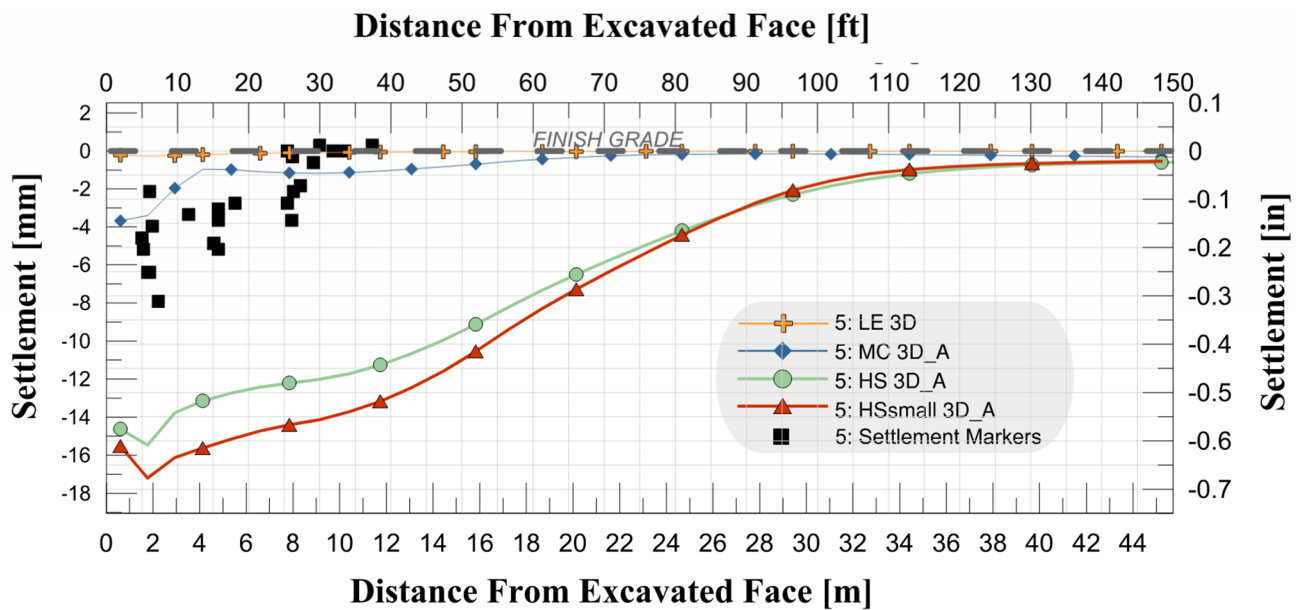


Figure 43: Settlement Comparisons Between LE, MC, HS and HS_{small} 2D Numerical Models for Final Depth of Excavation (Stage Five) and Settlement Markers

The LE model is unable to capture any surface settlements. The settlements obtained with this model correspond to only 3% of the maximum settlements observed in the field. The maximum settlements predicted using the MC model represent 44% of the maximum settlement measured on-

site; the predictions align with a lower bound of the observed data. The HS models overpredict the max measurement settlement by a factor of 1.95, and 2.15 for HS and HS_{small}, respectively. The in-situ maximum measured settlement (0.312 inches) is located approximately 7.3 feet away from the face of the excavation. This location is well predicted (within 20%) by both HS models. Measured surface settlements approach zero at 38ft (0.51*H) away from the excavation, while the numerical model does not assume settlement to converge to zero until about 150ft+ away from the excavation (~4 times farther). While the four models demonstrate comparative results in terms of location, the 2D numerical models do not adequately capture the settlement behavior due to over-simplification of 3D geometric effects.

Table 14: Maximum Settlements for 2D Numerical Model

	2D Numerical Max Vertical Settlements at Final Depth of Excavation (74.5 feet) for Undrained A				Settlement Marker Maximum
	LE	MC	HS	HS_{small}	
Max Settlement [inch]	0.011	0.14	0.61	0.68	0.312
Distance from face of excavation [ft]	5.79	2.00	5.79	5.79	7.3

As anticipated, preloading has a similarly negligible effect on the vertical settlements as it does on lateral deformations. Figure 44 presents a comparison of all vertical settlements obtained from all preload scenarios discussed in the previous section. For the HS_{small} model with all strut levels preloaded, the settlement within 15 feet of the excavation face increases by 19% maximum when compared to a level 1 preloading only. The HS model results in a decrease in settlement of 4% when compared to its counterpart with only level 1 preloaded. In summary, the HS_{small} models result

in a larger settlement for all levels preloaded relative to no-levels preloaded, while the HS models result in the opposite; this indicates an influence from the small-strain parameters

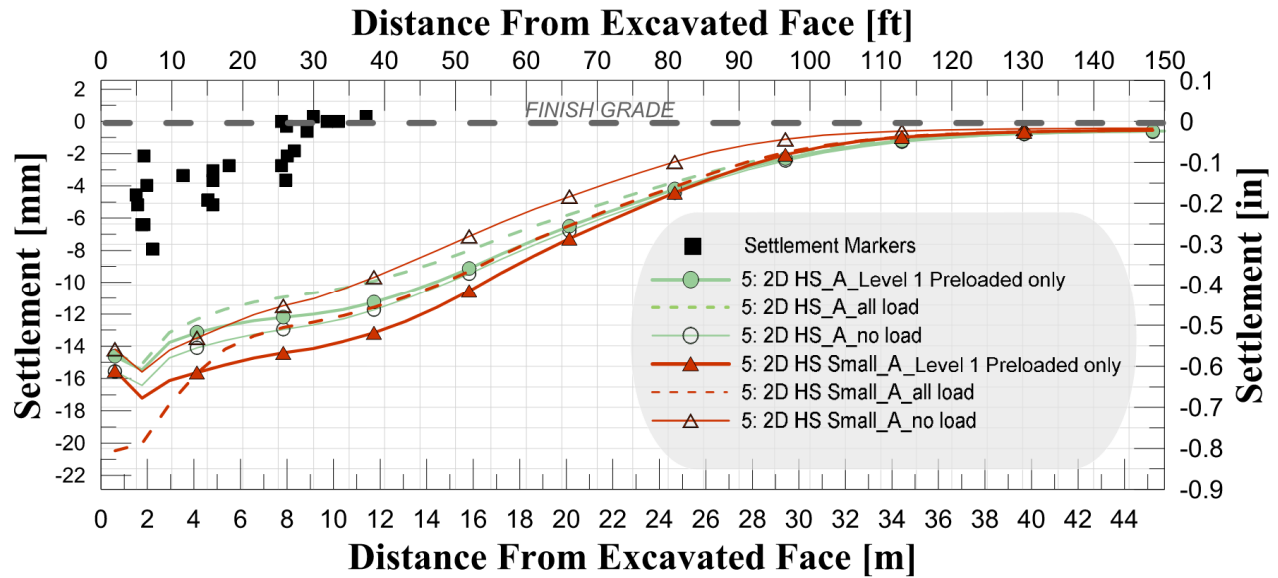


Figure 44: 2D Preloading Settlement Results for both HS Model Drainage types, Compared to Settlement Maker Measurements

3D – Lateral Deformations

In contrast to the 2D model, which primarily represents the I-4 plane, the 3D model allows for the observation of both vertical and lateral displacements throughout the entire shaft. However, analysis will concentrate on the I-4 through I-6 planes to maintain focus on the most relevant areas and enable comparison between 2D and 3D model results. The 3D geometry allows for a better representation of structural members and connections and therefore yields a stiffer structural system that reduces lateral deformations and better reflects real-world structural behavior.

The 3D model results presented below focus on the analysis type *Undrained A*, as this drainage condition yields more accurate magnitudes of mobilized undrained shear strength, as demonstrated in previous sections. Figure 45 shows the maximum displacements for the LE and MC models, recorded at the I-4 inclinometer, with values of 0.073 inch and 0.109 inch, respectively. The maximum lateral displacements for the HS and HS_{small} models are 0.45 inch and 0.47 inch, respectively. These two models exhibit a displacement pattern similar to the experimental data; however, they substantially underestimate deformations at all depths below the first strut level (12.5 feet). The maximum deformation is underestimated by a factor of 2. Only the deformations near the top of the exploratory shaft are more accurately captured by the 3D HS and HS_{small} models compared to their 2D counterparts.

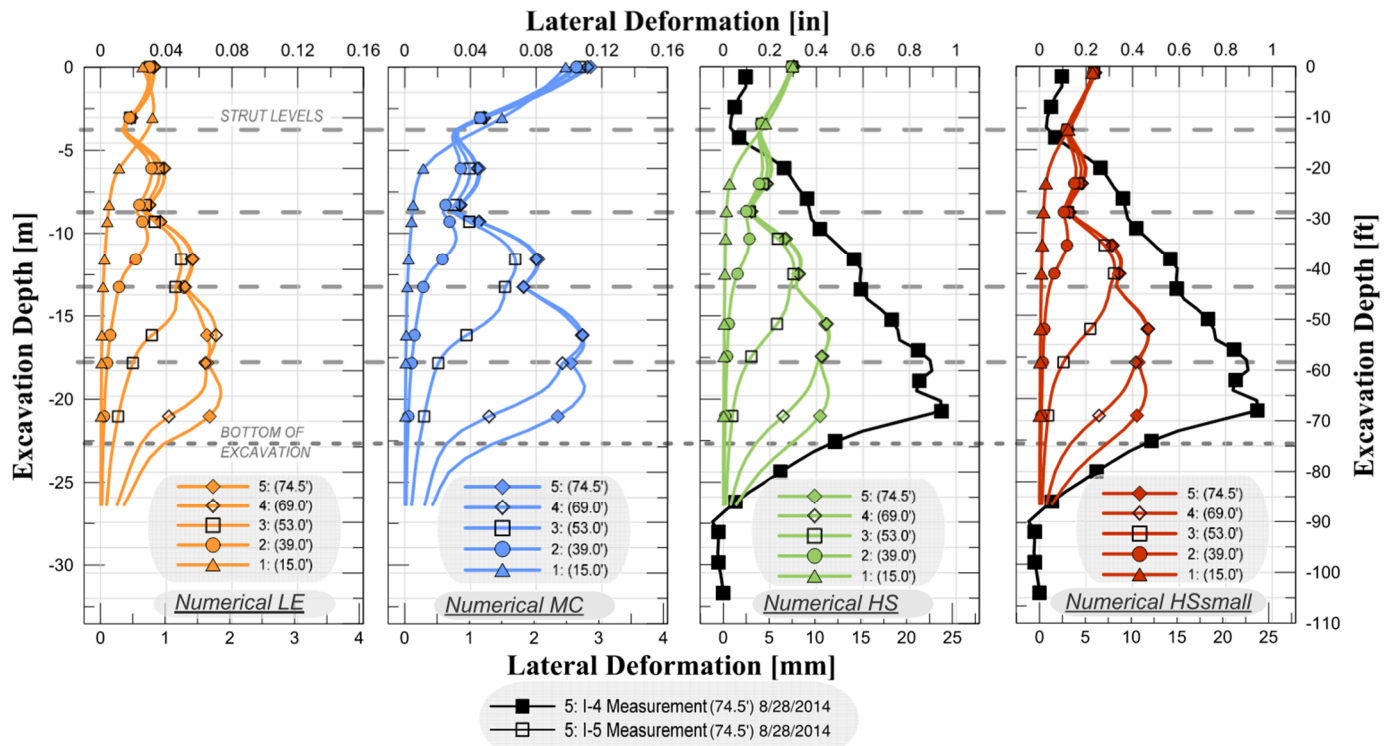


Figure 45: 3D Numerical Model LE, MC, HS, and HS_{small} Undrained A Results, Compared to I-4 Measurements

Table 15: Lateral Deformations for the 3D Numerical Model for Undrained A

Measurement Date	Excavation Depth [ft]	3D Numerical Max Lateral Deformation at I-4 for the Following Dates				
		LE_A [inch]	MC_A [inch]	HS_A [inch]	HS _{small} _A [inch]	I-4 [inch]
10/18/2013	15	0.007	0.007	0.16	0.13	0.16
12/18/2013	39	0.030	0.032	0.17	0.16	0.48
01/28/2014	53	0.049	0.068	0.30	0.32	0.61
03/12/2014	69	0.070	0.108	0.46	0.47	0.88
08/28/2014	74.5	0.073	0.109	0.45	0.47	0.95

Table 15 provides the numerical results for the maximum lateral displacement reached in each model. The HS_{small} Undrained A model's maximum displacements for I-4, I-5, and I-6 reach 0.47, 0.52, and 0.22 inches, respectively. These numerical displacements are within a range of 50%, 48%,

and 10% of the field measurements for I-4, I-5, and I-6 respectively (Figure 46). Especially deformations in I4 and I5 substantially underpredict the field measurements and the HS_{small}_A model is unable to capture the field data reliably. Numerical model results along the I-6 inclinometer plane provide a much better fit; the maximum displacements are within 10%, with the most likely reason being the stiffness of the walers near the corner. The waler beam elements provide a direct load path at the corner of the exploratory shaft that help minimize the load transferred to the soldier piles, and thus decrease the lateral displacements at the corner.

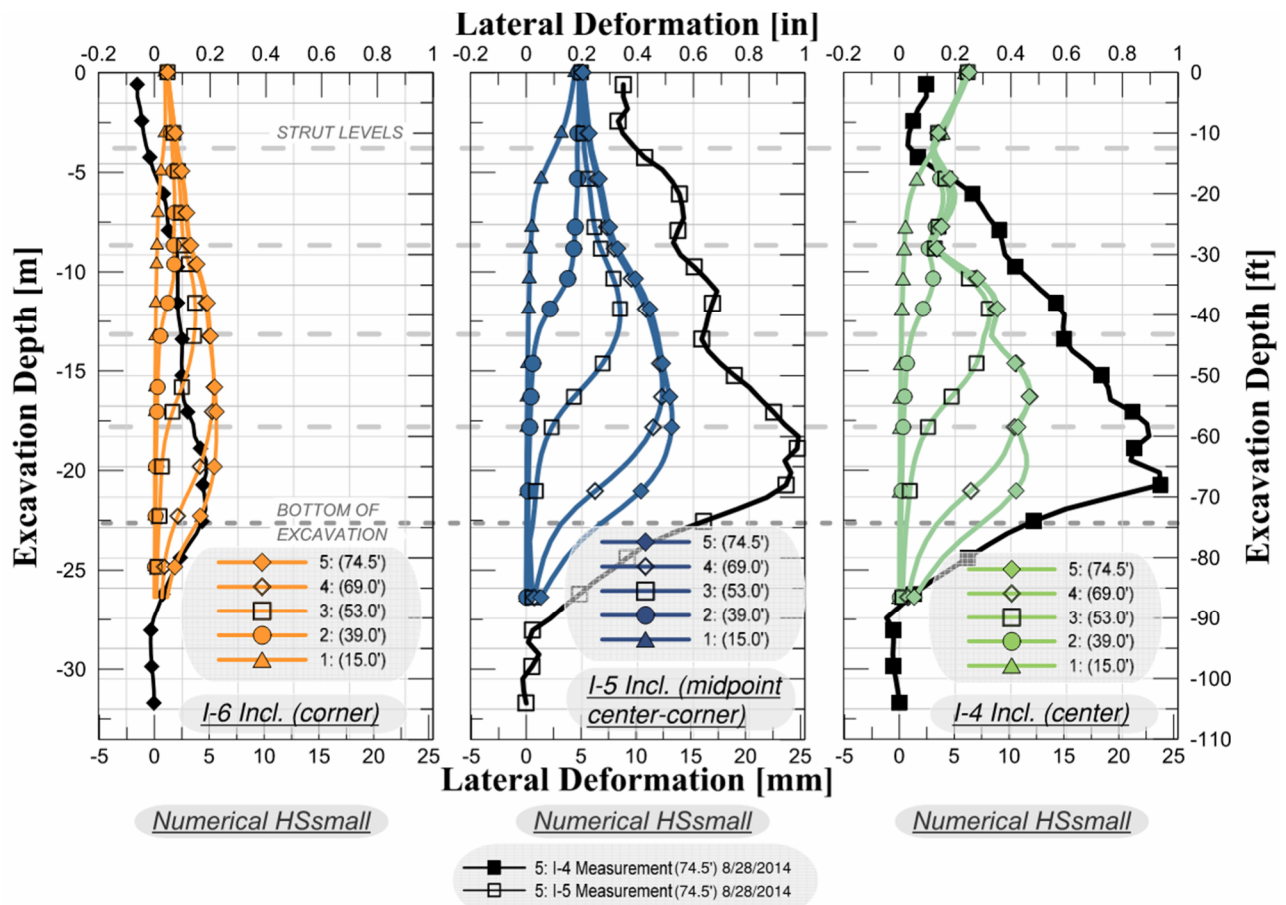


Figure 46: 3D Numerical deformations obtained for locations I-4 (center), I-5 (halfway center-corner), and I-6 (corner) for HS_{small} with analysis type Undrained A; numerical results are compared inclinometer measurements at stage 5 (74.5 ft of excavation)

Table 16: Lateral Deformations for the 3D Numerical Model for HS_{small} Model (Undrained A)

Measurement Date	Excavation Depth [ft]	3D Numerical Max Lateral Deformation at I-4 for the Following Dates					
		HS_A I-4 [inch]	HS_A I-5 [inch]	HS_A I-6 [inch]	I-4 [inch]	I-5 [inch]	I-6 [inch]
10/18/2013	15	0.13	0.03	0.02	0.16	0.19	0.02
12/18/2013	39	0.16	0.18	0.07	0.48	0.43	0.13
01/28/2014	53	0.32	0.34	0.15	0.61	0.54	0.14
03/12/2014	69	0.47	0.49	0.21	0.88	0.86	0.21
08/28/2014	74.5	0.47	0.52	0.22	0.95	1.0	0.20

3D – Vertical Deformations (Surface Settlements)

Figure 47 and Table 17 compare the measured surface settlements around the shaft with numerical predictions. All settlement values correspond to the time at which the maximum excavation depth H of 74.5 feet was reached. The location of the maximum settlements is within $10\%H$ away from the face of the excavation (i.e., 7.5 ft), consistent with the location of maximum settlements measured in the field. Similar to the lateral displacements, the HS and HS_{small} model vertical displacements are nearly identical and result in a maximum settlement of 0.35 inches; this maximum is within 12% of the maximum measured in the field (0.312 inches). The small difference in settlement values between the HS and HS_{small} model indicates a negligible contribution from the small-strain parameters, $\gamma_{0.7}$ and $G_{0.7}$.

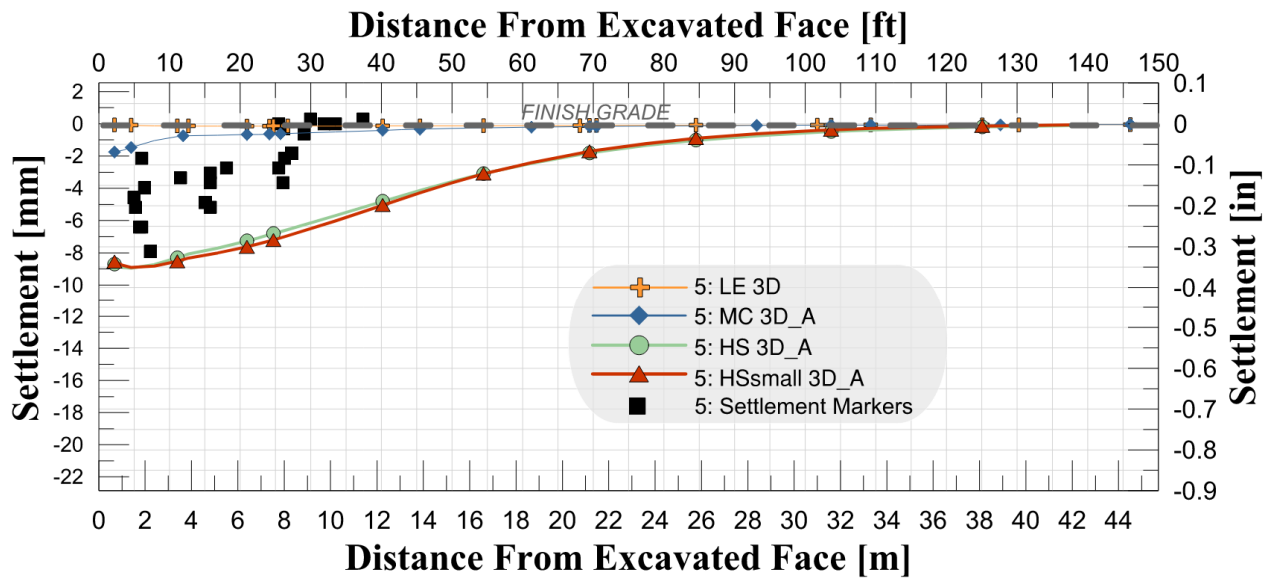


Figure 47: 3D Numerical Vertical Deformation for LE, MC, HS, and HS_{small} Models taken at the I-4 Inclinator Plane (along the center)

Table 17: Maximum Settlements for 3D Numerical Model

	3D Numerical Max Vertical Settlements at Final Depth of Excavation (74.5 feet) for Undrained A				Settlement Marker Maximum
	LE	MC	HS	HS _{small}	
Max Settlement [inch]	0.005	0.069	0.352	0.350	0.312
Location away from excavated face [ft]	20.98	2.25	4.58	4.58	7.3

Comparison and Discussion of Numerical Results

The comparison between the 2D and 3D models will focus on the model results of the HS and HS_{small} soil models at the final excavation depth of 74.5 ft and consider both, lateral and vertical displacements. To reiterate, the choice of drainage type significantly impacts the numerical deformation and overall soil response. For the exploratory shaft, *Undrained A* was deemed the most appropriate drainage condition due to previously mentioned limitations with *Undrained B*. Specifically, in the Undrained B model, compression hardening—represented by the oedometer modulus E_{oed} —is no longer captured, and the stress-dependency of stiffness is omitted. Additionally, internal triaxial tests performed in PLAXIS indicate that the mobilized shear strength is smaller for Undrained A compared to Undrained B, further highlighting differences in strength parameters.

Comparison of Lateral Displacements

Figure 48 presents a comparison of lateral deformation profiles obtained with HS_{small} in 2D and 3D for the center inclinometer, I-4. The 3D model represents an overall stiffer structural system compared to the 2D model which omits walers and geometric effects; as such deformations of the 3D model are smaller relative to the 2D model. As suggested by Figure 48, the 2D model clearly out-performs the 3D model when comparing the overall fit of the numerical results to the in-situ data in terms of magnitude and shape of the lateral excavation deflections.

In the 2D model, deformations at the top of the pile are largely overpredicted, which can be attributed to the soil exceeding its designated strength prior to the installation of the 1st strut. In turn, the 3D model recognizes the cap beam element at the top of the soldier piles and enables a better load distribution that prevents the same enlarged displacements. As such, the 3D model

captures the lateral displacement at the very top of the shaft more accurately, but substantially underpredicts the max lateral deflection magnitude at greater depths. The HS_{small} soil models underestimate the maximum measured inclinometer lateral deformations by 16% and 51% for the 2D and 3D models, respectively. From the Field Observations Summary Report [1], the inclinometers were installed within the soil mass adjacent to the pile. Due to the soil conditions, such as sloughing/caving due to the presence of tar, the soil movements exceeded the lateral displacements of the steel soldier piles, especially at depths below 38 feet. This behavior partly explains why the 2D numerical analyses match the inclinometer measurements more accurately than the pile displacements. Moreover, tar-infused soil also adversely impacts the soil strength and stiffness, resulting in a softer soil response that would, in turn, yield larger displacements for both the 2D and 3D numerical models. The potential influence of tar is only conditionally captured in the model, as no constitutive relationship is available to include the complex mechanism associated with tar excavations.

All numerical models properly recognize the location and influence of the struts. For both 2D and 3D, the magnitude and application of preloading has negligible impact below the first strut level (12.5 feet). Figure 48 presents a comparison of measured lateral deformations in the exploratory shaft at an excavation depth of 74.5 ft with numerical predictions obtained from PLAXIS for the HS and HS_{small} Undrained A analysis in 2D and 3D.

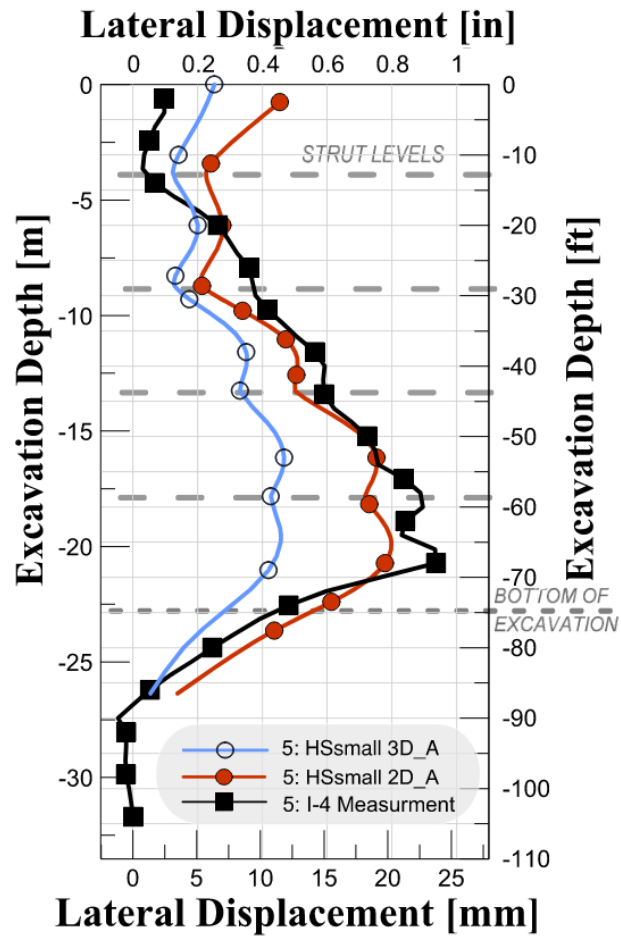


Figure 48: Comparison of lateral deformations obtained with HS_{small} 2D and 3D Numerical Models at the center of the exploratory shaft (I-4 Inclinator Plane)

To further compare the differences between 2D and 3D results, the *Plane-strain Ratio* (PSR), previously defined as the maximum displacement ratio for 3D/2D, will be revisited. A total of 3 PSR ratios are compared in Table 18:

- (1) A numerical PSR ratio, which was obtained by dividing the 3D numerical results by the 2D numerical results (i.e., $\frac{3D}{2D}$), as shown in Table 18 and referred to as “numerical PSR”.

(2) A semi-empirical PSR ratio obtained by dividing field-measured deformations over 2D numerical results.

(3) A literature based PSR ratio as defined by Finno *et al.* (2007) [10] and repeated from Equation 3 below.

$$PSR = \left(1 - e^{-kC(L/H)}\right) + 0.05 \left(\frac{L}{B} - 1\right) = \frac{\Delta 3D}{\Delta 2D} \quad (\text{Equation 3})$$

Where:

L = Width of long side of shaft (37 feet)

H = Excavation depth [feet]

C = Based on factor of safety against basal heave FS ($C=1$ if $1.8 \leq FS$)

k = Based on system stiffness S ($k=1$ for $S \leq 100$)

In addition to the PSR ratios, Table 18 also shows the 2D and 3D numerical lateral displacements for HS and HS_{small}, in addition to the I-4 inclinometer field measurements.

Table 18: Comparison of Lateral Displacements at I-4 for HS and HS_{small} Numerical Models

Measurement Date	Excavation Depth [ft]	L/H [-]	Lat. Displ. from 2D Analysis		Lat. Displ. from 3D Analysis		Field Data I-4 [inch]	PSR <i>Field/2D</i>	PSR per Eq. (5)	Numerical PSR	
			HS [inch]	HS _{small} [in]	HS [inch]	HS _{small} [inch]				HS (3D/2D)	HS _{small} (3D/2D)
10/18/13	15	2.47	0.21	0.20	0.16	0.13	0.16	0.80	0.96	0.76	0.65
12/18/13	39	0.95	0.19	0.21	0.17	0.16	0.48	2.29	0.66	0.89	0.76
01/28/14	53	0.70	0.41	0.45	0.30	0.32	0.61	1.36	0.55	0.73	0.71
03/12/14	69	0.54	0.67	0.73	0.46	0.47	0.88	1.21	0.46	0.69	0.64
08/28/14	74.5	0.50	0.73	0.80	0.45	0.47	0.95	1.19	0.44	0.62	0.59

The PSR ratio from Finno et al (Equation 3) starts out with a strong match between 3D and 2D results (PSR of 0.96) but reduces quickly to only 0.44 as the final depth of excavation is approached. This ratio is primarily influenced by system stiffness, S , the factor of safety against basal heave, FS , and shaft geometry as discussed in the Field Measurements section of Chapter 1. An improvement of Finno's PSR is difficult to achieve for the exploratory shaft, as an increase in system stiffness will decrease the PSR value, while decreasing the system stiffness will have negligible influence given that the current system stiffness is already considered extremely flexible ($S = 14.47$). Likewise, an increase to the FS will have no impact on the PSR value, as the maximum $FS = 1.8$ is already exceeded for the exploratory shaft ($FS = 6.61$); decreasing the FS will result in a smaller PSR value. It is also worth repeating that Equation 3 developed for excavations placed in soft to stiff clays, whereas the soil stiffness/strength parameters defined for the exploratory shaft are a sandy-silt and almost one order of magnitude larger than the clays of the original case history

that yielded Equation 3. As such the current case history is outside of the boundaries for which the Finno PSR was developed.

A comparison between the numerically obtained PSR ratios suggests the HS_{small} soil model to yield smaller PSR ratios than the HS model at any given depth. This implies the HS_{small} parameters, $\gamma_{0.7}$ and $G_{0.7}$, to be sensitive to geometric effects, especially at smaller depths of excavation. Numerical PSR values for both soil models indicate roughly a 40% reduction to the 2D numerical results at the final depth of excavation (74.5 feet) as shown in Figure 49. One potential explanation for the lower HS_{small} ratio is the overestimation of structural stiffness within the 3D model. This discrepancy may arise due to the modeling assumptions adopted for structural components, as outlined in the preceding section on *Modeling of Structural Components*. An overestimated stiffness would generally result in a stiffer response, thereby yielding a lower PSR. Furthermore, the HS_{small} numerical PSR (3D/2D) for the final depth provides a larger value than the PSR per Equation 3 by more than 30% (0.59 for HS_{small} vs. 0.44 for the PSR derived from Equation 3).

Figure 49 represents an updated version of Figure 21 from Chapter 1 to show the numerical PSR values for both, the HS and HS_{small} models. The PSR determined by Equation 3 is also plotted as reference, where the final depth (74.5 feet) represents a 66% decrease from 2D to 3D. All three plots within Figure 49 support a consistent trend where the PSR value increases as L/H increases.

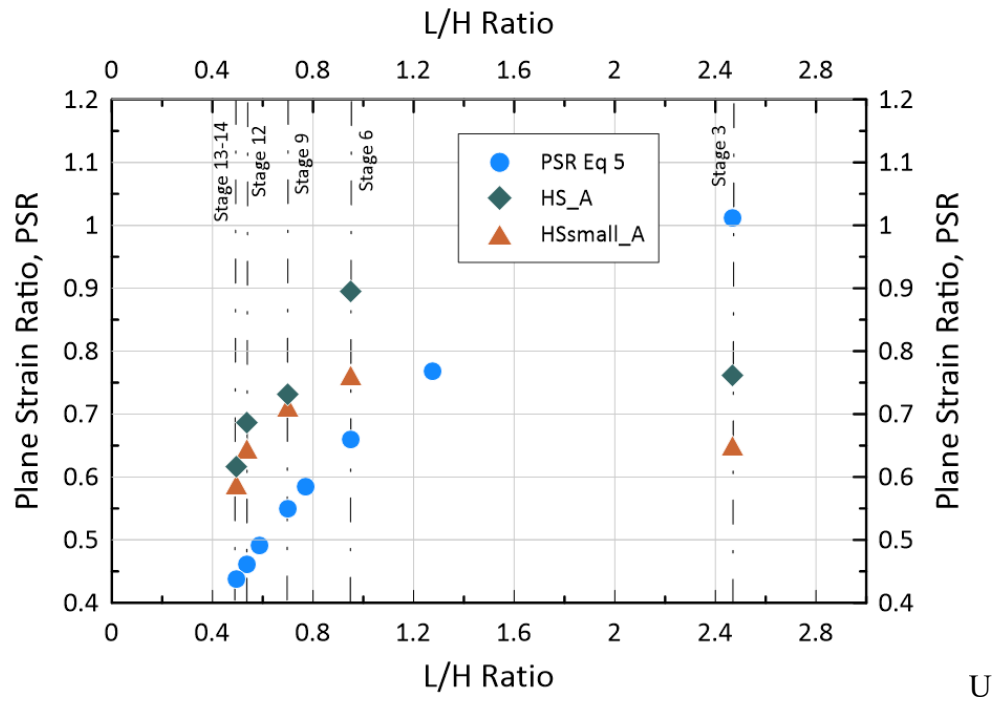


Figure 49: Comparison of Numerical PSR Values for HS and HS_{small} Numerical Models at the I-4 Inclinometer Plane (along the center) along with the PSR per (Equation 3.3 ; Trendline shown for HS_{small} and Equation 3 only

Table 19 below shows the comparisons of displacements occurring at the final depth of excavation (74.5 ft, 08-28-2014) between the HS_{small} numerical models (2D and 3D), and the inclinometer field measurements taken at I-4. The ratio between the field data and the 2D numerical model provide the most accurate results, maintaining a ratio of 1-1.5 for depths below 15 ft.

Table 19: Comparison of Displacements at I-4 (occurring at final depth of excavation 08-28-2014) per the HS_{small} Numerical Model Results for 2D and 3D, Including Inclinometer Field Measurements

Depth Along Pile [ft]	2D HS _{small} [in]	3D HS _{small} [in]	I-4 (Field) [in]	3D/2D	Field I-4/ 2D	Field I-4/ 3D
5	0.40	0.21	0.0969	0.52	0.24	0.46
10	0.26	0.14	0.0347	0.53	0.13	0.25
15	0.24	0.15	0.0679	0.63	0.28	0.45
20	0.28	0.20	0.2628	0.72	0.95	1.31
25	0.26	0.17	0.3222	0.65	1.24	1.91
30	0.25	0.15	0.3788	0.61	1.51	2.48
35	0.44	0.28	0.4647	0.65	1.06	1.65
40	0.51	0.35	0.5922	0.69	1.17	1.69
45	0.53	0.33	0.5911	0.62	1.11	1.79
50	0.69	0.44	0.7247	0.63	1.05	1.65
55	0.75	0.46	0.7578	0.61	1.00	1.65
60	0.73	0.43	0.8962	0.58	1.23	2.10
67.5	0.79	0.44	0.934	0.56	1.18	2.10
70	0.74	0.42	0.7646	0.57	1.03	1.82
74.5	0.57	0.29	0.4817	0.51	0.85	1.66

Vertical Settlements

Table 20 and Figure 50 present a comparison of numerical and measured surface settlements. The 2D model is unable to capture vertical settlement magnitudes appropriately. Both models (HS and HS_{small}) overpredict the measured settlements by 1.96 to 2.18 times compared to the field settlement marker measurements. As shown in Figure 50, the 2-D models predict settlement that extend to about 130 ft from the face of excavation, whereas the field measurements show settlements that extend only to 30 ft. Both the measured settlement magnitude and settlement distribution from the face of excavation differ substantially from the magnitude and distribution settlement of the 2D model.

The 3D simulations yield an adequate boundary for both soil models (HS and HS_{small}). The curve captures the maximum settlement magnitudes well and is within 11% of the measured settlements (Table 20, 0.312 inch vs. 0.35 inch). As observed from Figure 50, the 3D model provides an upper bound on settlement, however, the measured settlements are on average 50% of the 3D model settlements. The 3D model settlements extend 100 ft from the face of excavation whereas the measured settlements are confined to 30 ft from the face of excavation.

Both models, HS and HS_{small}, overestimate the extent of measured settlements in 2D and 3D, and predict vertical deformations to occur substantially beyond the recorded settlements (i.e., beyond 40ft from the excavation surface, Figure 50).

Settlements for the 3D models are 58% and 52% of their 2D counterparts for the HS and HS_{small} numerical models, respectively. These magnitudes are reversely consistent with the results obtained for the numerical lateral deformations where 2D model results resemble lateral excavation deformations more accurately and 3D results underestimate deformations by about 40%.

Table 20: Comparison of 3D Vertical Displacements at I-4 for HS and HS_{small} Numerical Models

	Max Vertical Settlements at Final Depth of Excavation (74.5 feet) for Undrained A				Settlement Marker Maximum
	2D Analysis		3D Analysis		
	HS	HS _{small}	HS	HS _{small} _	
Max Settlement [in]	0.61	0.68	0.352	0.350	0.312
Location from excavated face [ft]	5.79	5.79	4.58	4.58	7.3

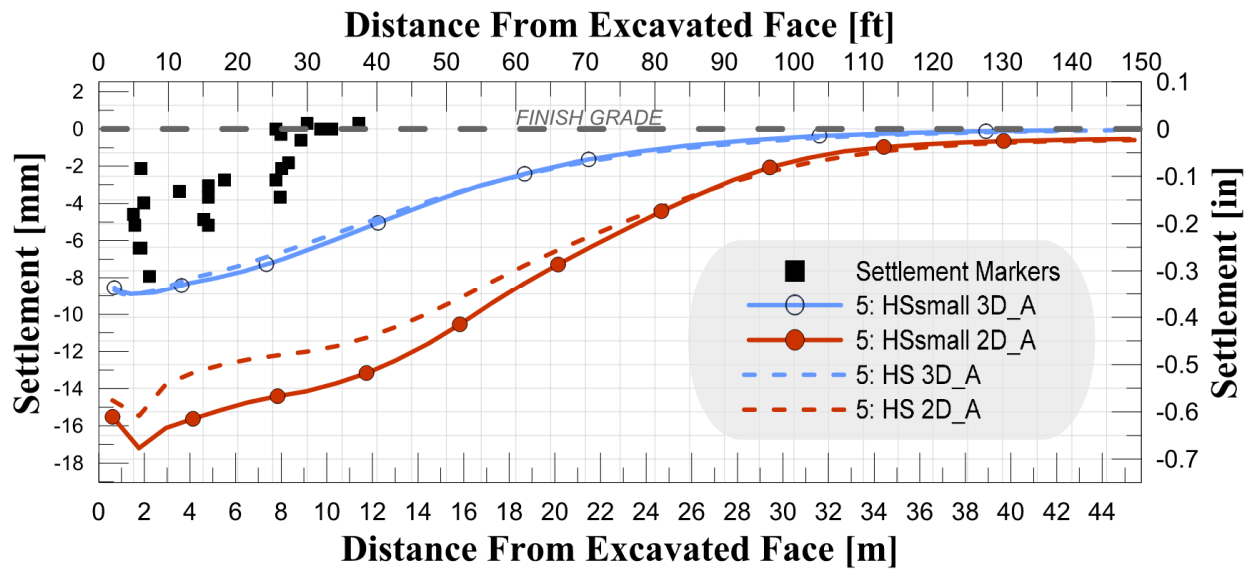


Figure 50: Comparison of 2D and 3D Vertical Displacements for HS and HS_{small} Numerical Models at the I-4 Inclinometer Plane (along the center)

Limitations of this study

Numerical models defined herein provided results for both 2D and 3D deformations utilizing four separate soil models. These results were individually compared to in-situ field measurements, in addition to being compared to one another to identify limitations with the model assumptions and provide possible correlations. By identifying soil parameters from in-situ and laboratory testing conducted with soil samples from the exploratory shaft, the required input for the soil models, linear-elastic, Mohr-Coulomb, hardening soil, and hardening soil with small-strain parameter, were defined. Key parameters include the elastic modulus, cohesion, friction angle, and secant modulus.

The Linear Elastic and Mohr Coulomb soil models substantially underestimated the lateral and vertical displacements. This discrepancy supports the apparent non-linear correlation of stress and strain when considering the deformation response of soil.

Limitations on the model include both in-situ identification of properties, and input limitations for defining structural response numerically. These can be summed up in the following:

- Determining correct soil layers and corresponding parameters are most critical for obtaining accurate model results. As noted from the results, underestimated soil strength in the Artificial Fill can result in premature shear failures, especially prior to the installation of the first strut when cantilever behavior is observed. Strength parameters have negligible impact below the first strut level, indicating that strains and corresponding stiffness have a substantial impact on displacement behavior. Therefore, maximum displacements observed in the exploratory shaft are primarily due to the defined stiffness parameters within HS and HS_{small} soil layers.
- Small-strain parameters defined in HS_{small} have minimal influence on vertical and lateral settlements, relative to the HS model results, if the internally calculated shear modulus, at the defined strain, $\gamma_{0.7}$, is roughly equal to the defined parameter $G_{0.7}$. The internally calculated shear modulus is based on the defined relationships of stress/strain with E_{50} , E_{oed} , and E_{ur} , all of which are equivalent for both HS and HS_{small}. If the defined $G_{0.7}$ parameter is larger, as is typically expected from in-situ lab results, the HS_{small} model results in smaller displacements (or less settlement). The opposite is observed when the $G_{0.7}$ parameter is smaller than the internally calculated shear modulus at that same strain, $\gamma_{0.7}$.
- Fixity of connections between structural elements for the 3D model, specifically at the corners and longitudinal boundaries of plates, can either overestimate or

underestimate the structural stiffness depending on the location of interest (center versus corner). Increased stiffnesses at the corner would be more consistent with welded field connections and would redirect the load path into the piles, thus acting similar to a strut. Longitudinal connections between plates, specifically at corners, should be free to rotate to omit moment transfer within plates that can absorb displacement that would otherwise be directed into soldier piles.

- Drainage parameters and defined drainage conditions/dewatering method are critical to correctly estimating lateral and vertical deformations. Drained characteristics near the excavation face and above the GWT result in less soil stiffness (loss of pore water pressure), thus increasing deformations. A more refined analysis of the that includes dewatering wells or the exploratory shaft itself acting like a dewatering well could define a more realistic behavior.
- Numerical omission of compression hardening within the PLAXIS software for Undrained B substantially limits the inclusion of undrained shear strength. Since Undrained A relies on an internal calculation that errs on the larger side for estimating undrained shear strength, careful consideration should be taken in choosing strength parameters, both effective and undrained.
- Understanding tar-infused soil and its influence on the soil are beyond the scope of this thesis. These properties also vary in terms of the amount of tar encountered at any given soil depth, posing a further challenge in defining the soil numerically. The influence of gas exsolution in tar soils upon being exposed to atmospheric pressure

changes the phase relationship of the in-situ tar and contributes to localized modifications in soil strength and stiffness of the tar infused strata. It is very likely that accounting for tar-infused soil would result in softer soil response. A softer soil response would yield larger displacements for both 2D and 3D numerical results. This expectation would be consistent with what is expected through the *Plane-strain Ratio*, where the 3D numerical results should be a better fit for the field measurements.

Summary

This thesis investigates the performance of a deep exploratory shaft in Los Angeles, part of Metro's Purple Line Extension project, through extensive numerical modeling. Using the finite element program PLAXIS, both two-dimensional (2D) and three-dimensional (3D) analyses were conducted with four soil constitutive models: Linear Elastic (LE), Mohr-Coulomb (MC), Hardening Soil (HS), and Hardening Soil with small-strain stiffness (HS_{small}). The modeling results were calibrated and compared against field measurements from inclinometers and settlement markers to evaluate accuracy and predictive capability.

The work provides important insights into soil–structure interaction and modeling limitations for deep excavations in stiff clayey and tar-infused soils. Many interesting observations emerge, including:

Lateral Displacements – Comparisons of 2D and 3D models across LE, MC, HS, and HS_{small} show that 2D analyses generally matched inclinometer measurements more closely, while 3D models

tended to underpredict maximum deflections at depth due to stiffer boundary representation. Structural fixity assumptions in 3D analyses also introduced discrepancies, as corner stiffness was potentially overrepresented.

Vertical Movements – Settlement predictions differed substantially. The 2D models overpredicted magnitudes and extents of settlements, while 3D models more closely captured measured values, with discrepancies reduced to within 11%.

Drainage Effects – Drainage type strongly influenced numerical deformation. Undrained A conditions better reflected soil strength mobilization, while Undrained B omitted compression hardening and proved less reliable.

2D vs 3D Model Accuracy – While 2D models captured lateral deformation magnitudes more effectively, 3D models provided better settlement predictions and more realistic representation of structural member interactions.

Plane-Strain Ratio (PSR) – Numerical PSR values (3D/2D) showed about a 40% reduction in displacement at full excavation depth, aligning partially with semi-empirical ratios from Finno et al. (2007). Field-measured PSR values further highlighted geometric and stiffness effects.

Strut Preloading – Preloading struts had negligible influence below the first strut level (12.5 ft). Differences between HS and HS_{small} models were linked to small-strain stiffness effects.

Constitutive Soil Modeling – Tar-infused soils may likely contribute to softer responses not fully captured in the numerical models.

Overall, the study demonstrates that neither 2D nor 3D models alone provide a complete picture, but their combined use, calibrated with field data, can significantly improve predictive confidence for excavation performance. A critical limitation of this study is the lack of a constitutive model tailored to tar-infused soils. The numerical analyses were performed using soil models originally developed for traditional clays and sands, which do not capture the unique effects of bitumen, such as reduced permeability, altered stiffness, and phase-dependent behavior under stress. As a result, specialized characteristics of tar soils were not represented in the models, leading to differences between predicted and measured performance. This underscores the need for future research aimed at developing constitutive models specifically calibrated for tar-impacted geomaterials.

In summary, the thesis highlights the strengths and limitations of numerical approaches for deep excavation design, underlining the importance of selecting appropriate soil models, drainage assumptions, and calibration with monitoring data to achieve safe and reliable infrastructure in complex urban ground conditions.

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