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LONG-TERM PERFORMANCE OF DRILLED SHAFT RETAINING WALLS: ASSESSMENT OF EXISTING WALLS

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16. Abstract This report provides assessment from information and analysis for three drilled shaft walls in service in Houston, Texas. The three walls have been in service for 14, 9 and 2 years, respectively, and have cantilevered heights ranging from 5 to 23 feet. A field inspection of each wall revealed no obvious signs of significant distress. Based on L-Pile analyses, earth pressures greater than a linear increase of 80 psf/ft would likely be required to produce significant distress that could be readily observed in these walls.		
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Research Supervisor: Robert Gilbert

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1. INTRODUCTION

The objective of this project is to assess the design performance of existing drilled shaft retaining walls built for TxDOT in expansive clay soils. This report provide assessment information and analysis for three drilled shaft walls constructed in Houston, Texas in 1997, 2002, and 2009, respectively.

2. BACKGROUND INFORMATION

Cantilever drilled shaft retaining walls are common earth-retaining structures in Texas. They are well suited to use in urban environments where noise, space, and damage to adjacent structures is a major consideration (Wang and Reese 1986). Additionally, because of the prevalence of drilled shaft foundations in Texas, experienced contractors are readily available. The design of drilled shaft retaining walls has changed over time. While initial design methods were based on limit equilibrium calculations, more refined p-y analyses based on soil-structure interaction have been developed and are currently in use by TxDOT (Wang and Reese 1986, TxDOT 2009).

There is uncertainty in how to account for lateral earth pressures acting on drilled shaft walls installed through expansive clay. In Texas, some of the most problematic expansive clay deposits are also highly overconsolidated. For this reason, an examination of retaining wall design procedures for stiff, overconsolidated clay can provide a reference point for the design of walls in expansive clay deposits.

Commonly, the earth pressure on walls in stiff, overconsolidated clay is estimated using Coulomb active earth pressures with drained properties (Wang and Reese 1986). The TxDOT Design Procedure for Cantilever Drilled Shaft Walls employs this method with a recommended friction angle of 30 degrees for "medium to stiff clays" (TxDOT 2009). This approach results in earth pressures that correspond to an equivalent fluid density of approximately 35 to 40 pounds per cubic foot (pcf) for clays common in Texas.

In the current TxDOT design procedure, drilled shaft size and spacing is based on moment capacity. The computed groundline moment from the calculated earth pressures is multiplied by 1.5 to estimate the maximum moment. Then a load factor of 1.7 is applied to estimate the ultimate moment (TxDOT 2009). The final check for the TxDOT design procedure uses COM624 or LPILE to ensure that the base of the shafts is fixed and that the predicted top-of-wall deflection does not exceed 1% of the wall height. The design guide also notes that "deflections observed in the field seldom reach the predicted value" (TxDOT 2009).

There have been concerns raised over the potential effects of expansive soils on retaining structures. The most common of these concerns is the magnitude of horizontal swelling pressures exerted on the wall by the expansive soil. Lytton (2007) summarizes some relevant studies that seek to quantify this effect. Variously, the potential lateral pressures acting on a wall in expansive clay have been estimated to be four times the overburden pressure, 6000 psf at three feet of depth in a lab study, 8000 psf at three feet of depth in another lab study, and 1700 psf at three feet of depth in a field study. These studies are described in more detail in Lytton (2007). In general, the expansive soil pressure exerted on a wall is considered to be limited by the passive resistance of the retained soil (Pufahl et al. 1983 and Hong 2008).

In addition to the potential for high lateral pressures, other potential concerns have been identified for retaining walls in expansive clay. Pufahl et al. (1983) describe a hypothetical structure "ratcheting" out with wetting and drying cycles. During dry seasons, the soil could pull back from the wall, incompressible debris could fill the gap, and soil expansion could push the wall and debris further out with each new rewetting cycle. Puppala et al. (2011) describe that cracks near drilled shafts could create zones for moisture infiltration, increasing the depth of the active zone near the shafts.

The behavior is complicated because expansive soils in Texas are also heavily overconsolidated. In overconsolidated clay, in-situ horizontal stresses can be very large. When the unloading associated with retaining wall excavation takes place, these large horizontal stresses can impact wall performance. Furthermore, the residual strength of overconsolidated clay can be very low – residual friction angles of 18 degrees or less have been widely reported. The transition from peak-drained strength to residual-drained strength could influence the increase in lateral earth pressures with time (Wang and Reese 1986). The lateral swell pressures from moisture changes in overconsolidated clay have been reported to be higher than those in normally consolidated clay (Ellis 2011).

Because the potential for expansion and a high degree of overconsolidation coexist in expansive clays in Texas, it is difficult to separate the effects of swelling from the effects of overconsolidation. Smith et al. (2009) examine the failure of a bridge deck completed using top down construction in the overconsolidated, expansive Eagle Ford shale near Dallas, TX. In this case, the bridge deck was installed before complete excavation of the underpass and installation of tiebacks. Ultimately, an estimated four inches of inward movement caused the failure of the bridge deck. The authors concluded that the major issue was the use of a Ko value of approximately 0.7; actual values of Ko for the Eagle Ford shale and other overconsolidated clays are often reported to be approximately 3.0. Expansive soil movement was cited as a "likely" contributing factor (Smith et al. 2009).

Another failure in the Eagle Ford shale, this time of a VERT wall system, is detailed by Adil Haque and Bryant (2011). This paper indicates that the high Ko values and low residual strengths of overconsolidated clay, as well as expansion from moisture changes, should have been considered in design. The paper also states that "the swell pressure due to unloading could also exert a significant pressure on the wall, much greater than the swell pressure on the walls from moisture changes" (Adil Haque and Bryant 2011).

Despite the numerous problems potentially associated with the expansive soils in Texas, relatively few failures of drilled shaft retaining walls have been observed. There are several possible explanations for the general lack of problems associated with drilled shaft retaining walls in expansive clays in Texas.

First, the load factors and deflection requirements used by the TxDOT design procedure will result in drilled shafts that can withstand higher pressures than the nominal values used in design. After calculating the maximum moment in the shaft, a load factor of 1.7 is applied to estimate the ultimate moment for design. All other things being equal, the result of this load factor is that shafts designed using an equivalent fluid pressure of approximately 40 psf/ft (a value commonly used for expansive clays in Texas) could withstand the bending moments induced by a pressure of approximately 60 psf/ft (for reference, a Coulomb analysis using a residual friction angle of 18 degrees results in an equivalent fluid pressure of approximately 60 psf/ft). While the top-of-shaft deflections might exceed one percent of the wall height, the structural integrity of the shafts may be preserved and there may be no distress to the wall. Furthermore, the drilled shafts may have greater capacity than the minimum allowed by design due to other factors such as constructability.

Additionally, pavement and drainage systems behind drilled shaft walls may limit the severity of moisture changes causing shrinking and swelling. In pavements with expansive subgrades, moisture contents tend to increase from their natural moisture content to a "steady state" value after the installation of pavement (Snethen et al. 1975, Wise et al. 1971). While the subgrade is still subject to moisture changes, the magnitude of these changes may be smaller than those of exposed soil. The presence of pavement near the shaft can also prevent the problems associated with water and/or debris entering the gap between the shaft and the soil (Puppala et al. 2011).

Finally, despite the potential to generate very large swell pressures under confinement, swell pressures can be reduced by allowing relatively small wall deformations to take place (Thomas et al. 2009). For projects as large as the typical TxDOT drilled shaft retaining wall, it is possible that expansive soil pressures are being accommodated by small wall deformations that would not be noticed without careful instrumentation.

3. CANDIDATE WALLS

Three existing walls were selected for assessment with the cooperation of TxDOT managers. While there may be additional walls throughout the state of Texas that warrant further study, the following three walls from the Houston district were the only candidates that could be identified with the information provided by TxDOT.

1. FM 1960 @ Kuykendahl (CSJ # 1685-01-082)
2. US 59 @ Hazard Street (CSJ # 0027-13-165)
3. IH 45 @ GREENS ROAD (CSJ # 0110-06-102)

3.1. FM 1960 @ Kuykendahl (CSJ # 1685-01-082)

This retaining wall is located in north Houston, TX, where Kuykendahl Road passes under FM 1960 (Figure 3.1). The underpass was built to relieve congestion at the intersection. Based on TxDOT's payment records and satellite imagery, excavation was likely completed in late 2008 (Ozuna 2011). The area had been developed prior to construction, and the project represents a change to an existing roadway that was already covered with pavement (Figures 3.2 and 3.3). The wall is a hybrid structure consisting of a cantilever drilled shaft wall over the middle depths and a tieback wall over the deeper depths. As of June 30, 2011, no obvious signs of distress have been observed.

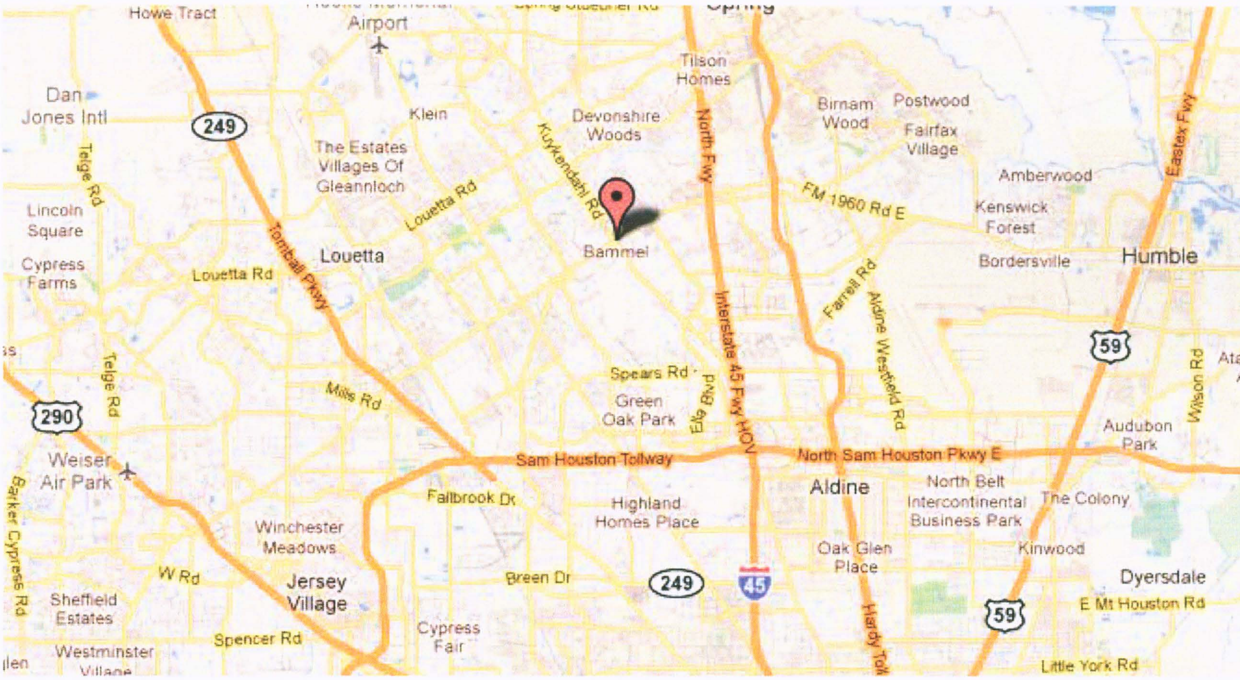


Figure 3.1: View of wall location within greater Houston (Google Inc., 2011).

Figure 3.2: Aerial view of project area before excavation (image date: January 2008) (Google Inc., 2011).

Figure 3.3: Aerial view of project area after completion (image date: March 2011) (Google, Inc., 2011).

3.2. US 59 @ Hazard Street (CSJ # 0027-13-165)

This retaining wall is located in Houston, TX, on US Highway 59 between South Shepherd Street and Mandell Street (Figures 3.4 and 3.5). Based on satellite imagery and available information from TxDOT, excavation was likely completed in mid-2002 (Figures 3.6 and 3.7). The wall consists of several sections of similar cantilever drilled shaft walls, interrupted by bridge abutments at regular intervals. During an assessment performed June 30, 2011, some gaps between the retained soil and the wall were observed, but no signs of wall distress were clearly present.

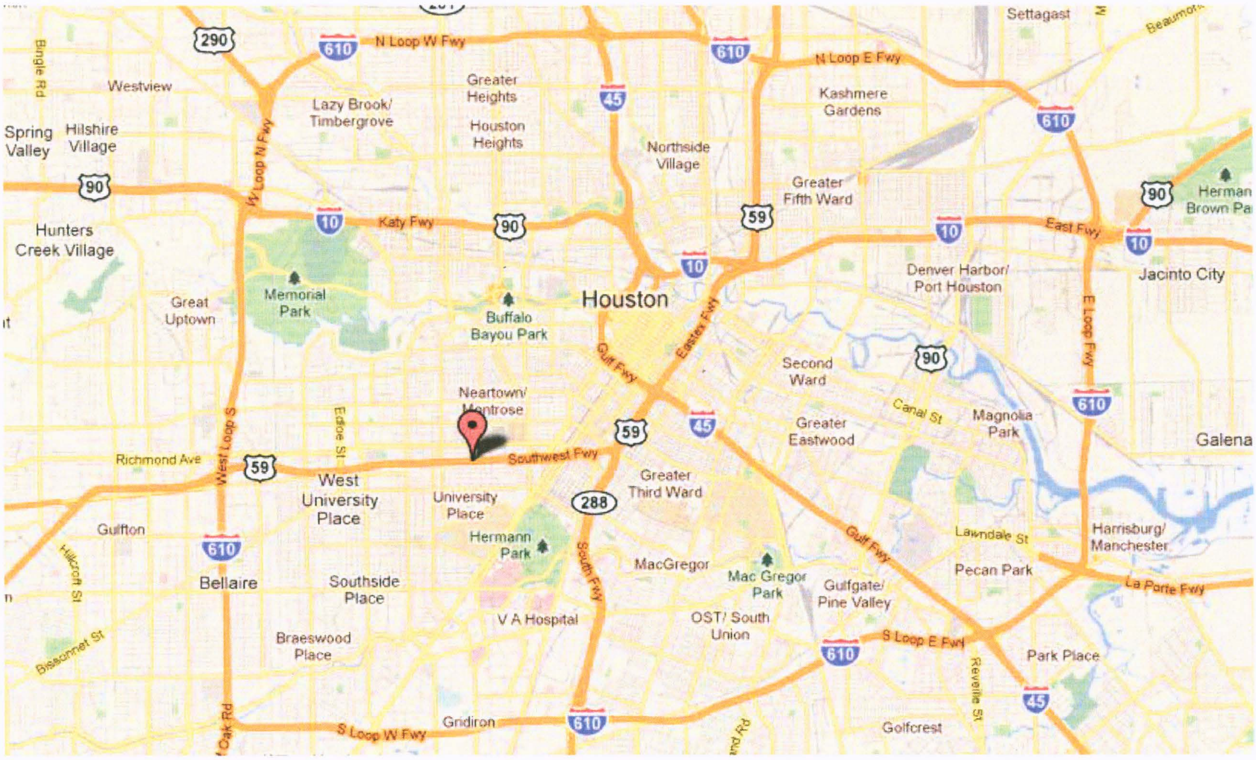


Figure 3.4: View of wall location within Houston (Google Inc., 2011).

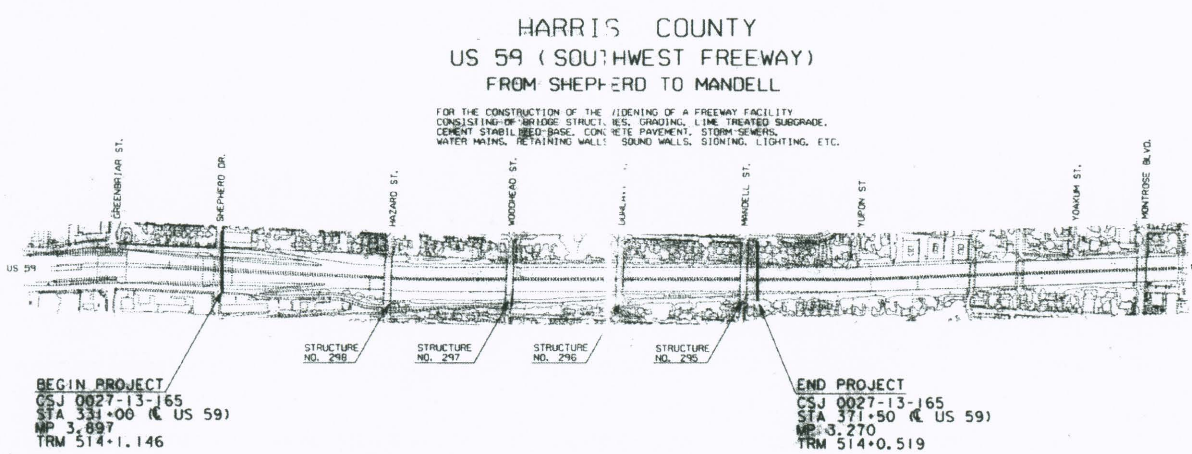


Figure 3.5: View of project area along US 59.

Figure 3.6: Aerial view of the project area before construction (image date: January 1995) (Google Inc., 2011).

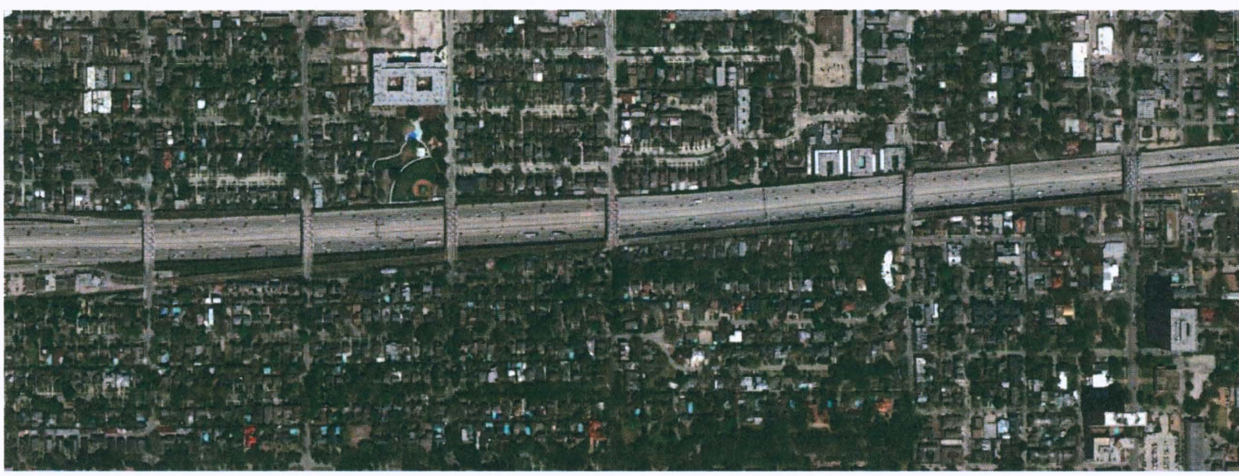


Figure 3.7: Aerial view of the project area after completion (image date: March 2011) (Google Inc., 2011).

3.3. IH 45 @ Greens Road (CSJ # 0110-06-102)

The retaining wall is located in north Houston, TX where Greens Road passes under the Interstate Highway 45 frontage road (Figure 3.8). A highway overpass existed prior to construction. Based on satellite imagery and correspondence with TxDOT, excavation was likely completed in mid-1997. Aerial images of the site before and after wall construction are shown in Figures 3.9 and 3.10. The wall is a hybrid structure consisting of a cantilever drilled shaft wall at shallow depths and a tieback wall at higher design heights. As of June 30, 2011, no obvious signs of distress have been observed.

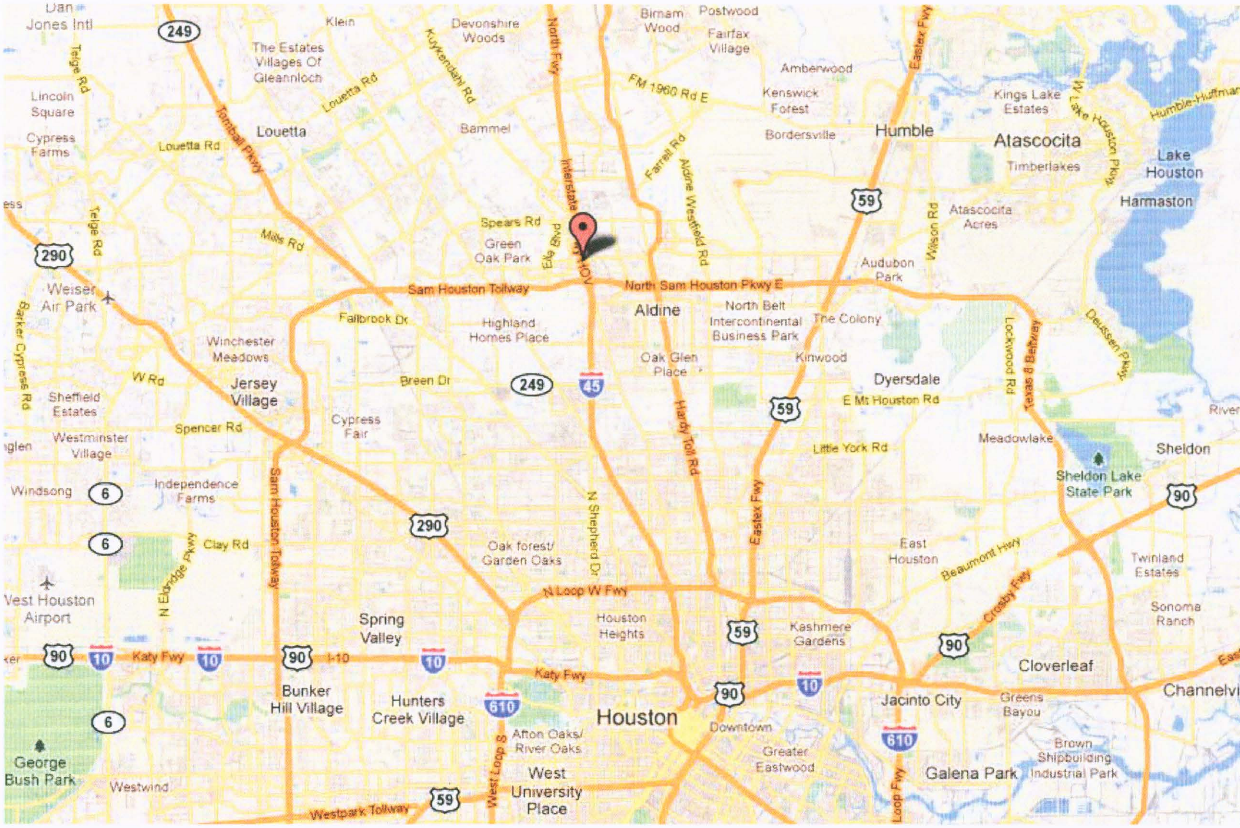


Figure 3.8: View of wall location within greater Houston (Google Inc., 2011).

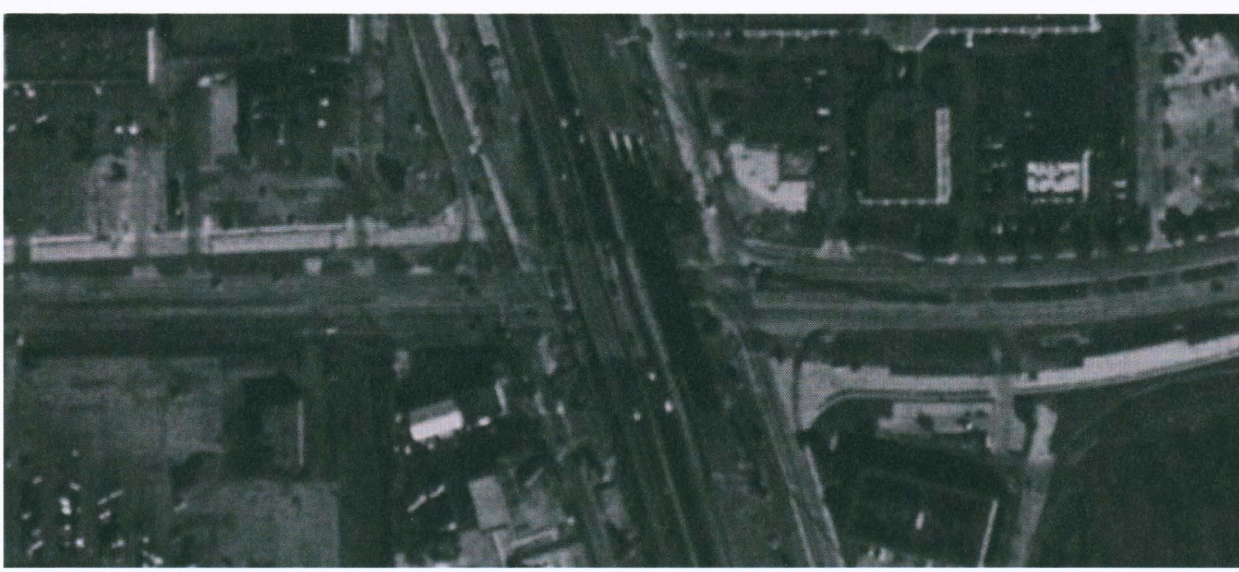


Figure 3.9: Aerial view of the project area before construction (image date: January 1995) (Google Inc., 2011).

Figure 3.10: Aerial view of the project area after completion (image date: March 2011) (Google Inc., 2011).

4. CLIMATE INFORMATION

In order to identify the potential for expansive soil movement, climatic cycles between wet and dry seasons need to be examined. Because the three candidate walls are located in Houston, TX, the climate data for all three should be sufficiently similar. Additionally, no construction records were available for the candidate walls. To estimate construction dates, TxDOT suggested that we go back one year from the final payment date from TxDOT to the contractor (Ozuna 2011). Because we lack precise information about when the shafts were installed and the excavations completed, conclusions drawn from site-specific climate data should be qualified.

Vipulanandan and Joseph (2011) examined moisture fluctuations in the active zone for the city of Houston from the years 2000 through 2007. While this information is not directly applicable to the candidate walls, it does indicate that Houston experienced a range of climate related soil moisture fluctuations that could potentially lead to expansive soil movement. During the month of January (lowest average temperature), the average moisture content in the upper 10 feet of soil was approximately 16 percent. During the month of July (highest average temperature), the average moisture content in the upper 10 feet was approximately 18 percent. Year-to-year fluctuations were much greater. The highest fluctuations occurred at depths from 0 to 5 feet. While temperature effects on soil moisture were seen immediately, the effects of rainfall did not appear until "the following months" (Vipulanandan and Joseph 2011). A graph of monthly rainfall and temperature in Houston from 2000-2011 is provided in Figure 4.1. A comparison of yearly precipitation with the historical average yearly precipitation is provided in Figure 4.2.

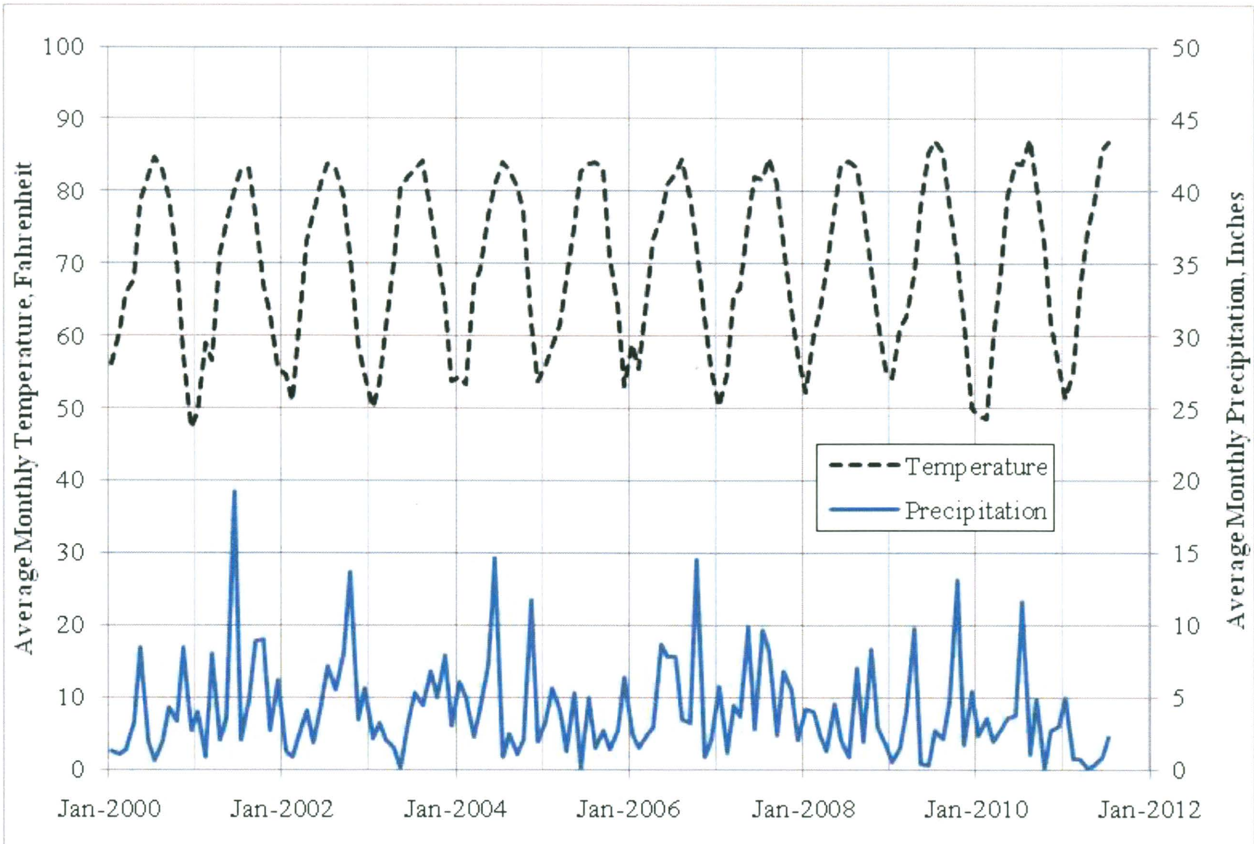


Figure 4.1: Average monthly precipitation and temperature for Houston, TX (2000-2011). Data from Weather Underground (2011).

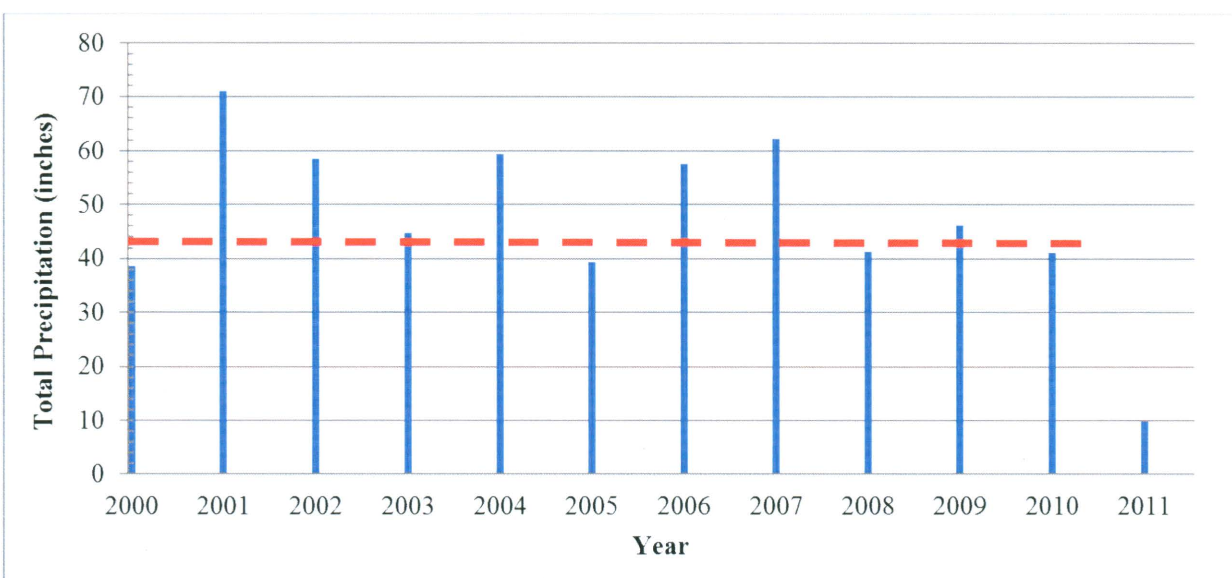


Figure 4.2: Yearly precipitation totals since 2000. The dashed red line denotes the yearly historic average. The 2011 total is of the end of July. The historic average from January through July is 19.2 inches of precipitation. Data from Weather Underground (2011).

5. DESIGN INFORMATION

This section presents the geotechnical and design information for the candidate walls. This information is based on design documents provided by TxDOT.

5.1. FM 1960 @ Kuykendahl

5.1.1. Geotechnical Information

Nine geotechnical borings were drilled near the project site between April 26, 2001 and May 7, 2001. Boring logs indicate the soil profile consists of very stiff clay to a depth of approximately 10 feet, which is underlain by approximately 10 feet of dense sand to a depth of about 20 feet. Below 20 feet, there are alternating layers of dense sand and stiff clay that show some variability across the project site (boring locations are not indicated in the available documents).

The very stiff clay in the upper 10 feet is of particular interest because it may be subjected to moisture changes causing shrinking and swelling. The Plasticity Index (PI) of this layer ranged from 11 to 39, but is typically in the mid 20s, indicating marginal swell potential (Department of Army 1983). Liquid Limits (LL) ranged from 24 to 54 percent. Moisture contents in the upper 20 feet ranged from 11 to 23 percent. A water table location was not reported in any of the boring logs. The average undrained shear strength reported in the boring logs ranges from approximately 2000 to 4000 psf, based on the results of pocket penetrometer and UU testing.

5.1.2. Design Information

The retaining wall at FM 1960 and Kuykendahl Road consists of a combination of cantilever drilled shaft walls and tieback walls (Figure 5.1). For this investigation, only the cantilever drilled shaft wall is considered.

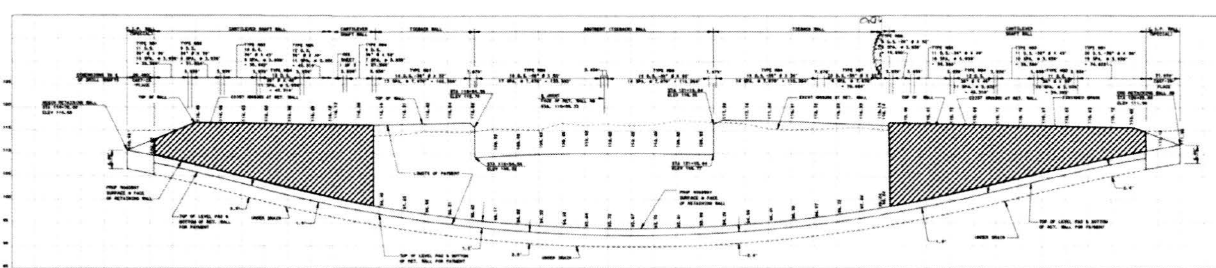


Figure 5.1: Composite sketch of northbound side of underpass. Shaded areas indicate locations of cantilever drilled shaft wall.

The drilled shaft wall consists of 36-inch diameter shafts that are 31 to 52 feet in length. Design heights range from 14 to 23 feet. As shaft length increases, more steel reinforcement is used (Figures 5.2 and 5.3). The center-to-center spacing of the shafts is approximately 47 inches (Figure 5.4). Wall facing consists of 8.5 inches of cast-in-place concrete and 5.5 inch precast panels (Figure 5.5).

NORTH BOUND RETAINING WALL			
TYPE (2)	WALL DESIGN HEIGHT "H"	VERTICAL REINF.	LENGTH OF DRILLED SHAFT
TYPE NB1	14	10 #9	36'
TYPE NB2	16	12 #9	39'
TYPE NB3	18	12 #11	43'
TYPE NB4	20	14 #11	46'
TYPE NB5	22	16 #11 (3)	49'
TYPE NB6	23	18 #11 (3)	52'

SOUTH BOUND RETAINING WALL			
TYPE (2)	WALL DESIGN HEIGHT "H"	VERTICAL REINF.	LENGTH OF DRILLED SHAFT
TYPE SB1	12	10 #9	31'
TYPE SB2	14	10 #9	36'
TYPE SB3	16	12 #9	39'
TYPE SB4	18	12 #11	43'
TYPE SB5	20	14 #11	46'
TYPE SB6	23	18 #11 (3)	52'

(2) SEE RETAINING WALL LAYOUT FOR LOCATION AND DRILLED SHAFT SPACING.
(3) BUNDLED BARS ~ 2 BARS PER BUNDLE.

Figure 5.2: Design heights, reinforcement type, and shaft length for cantilever drilled shaft wall.

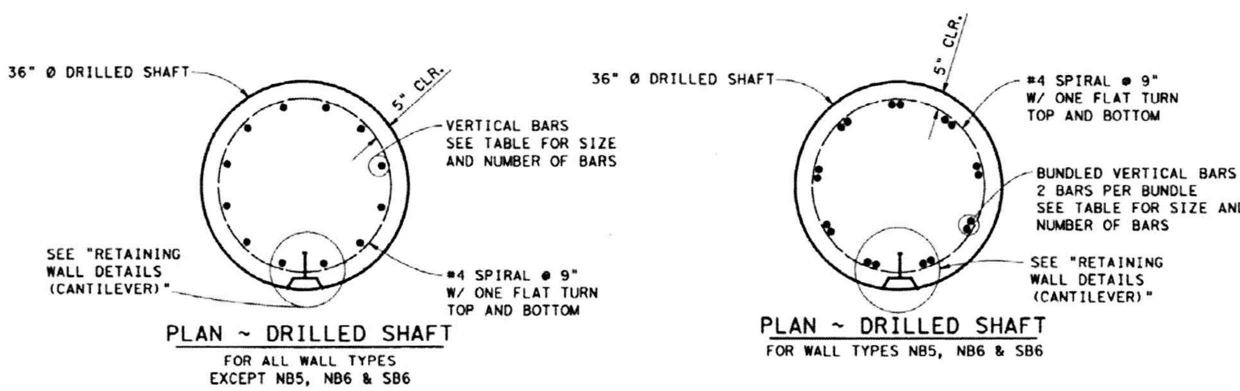


Figure 5.3: Reinforcement types for cantilever drilled shaft wall.

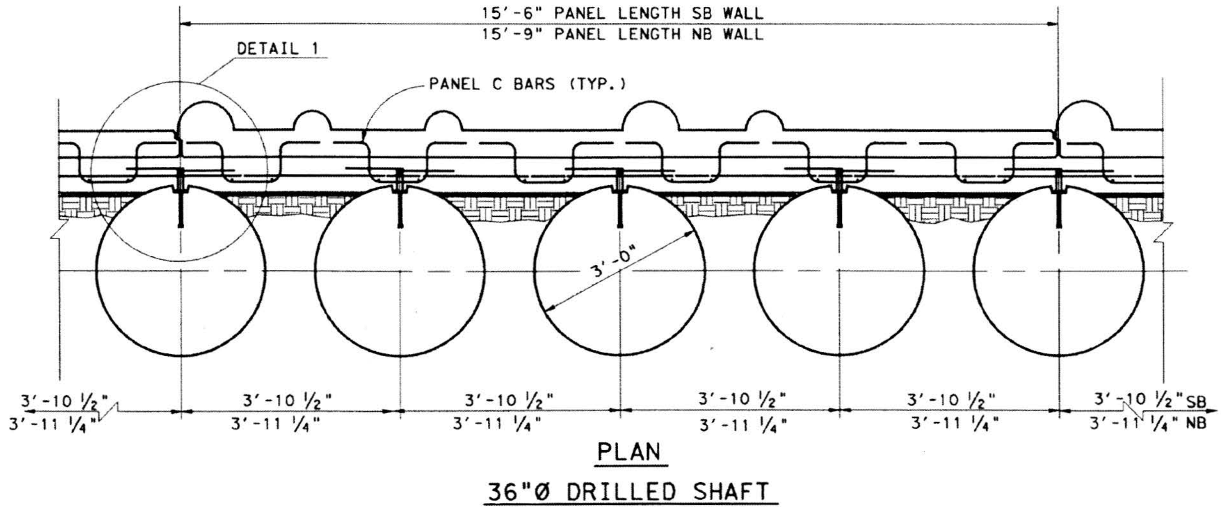


Figure 5.4: Shaft spacing.

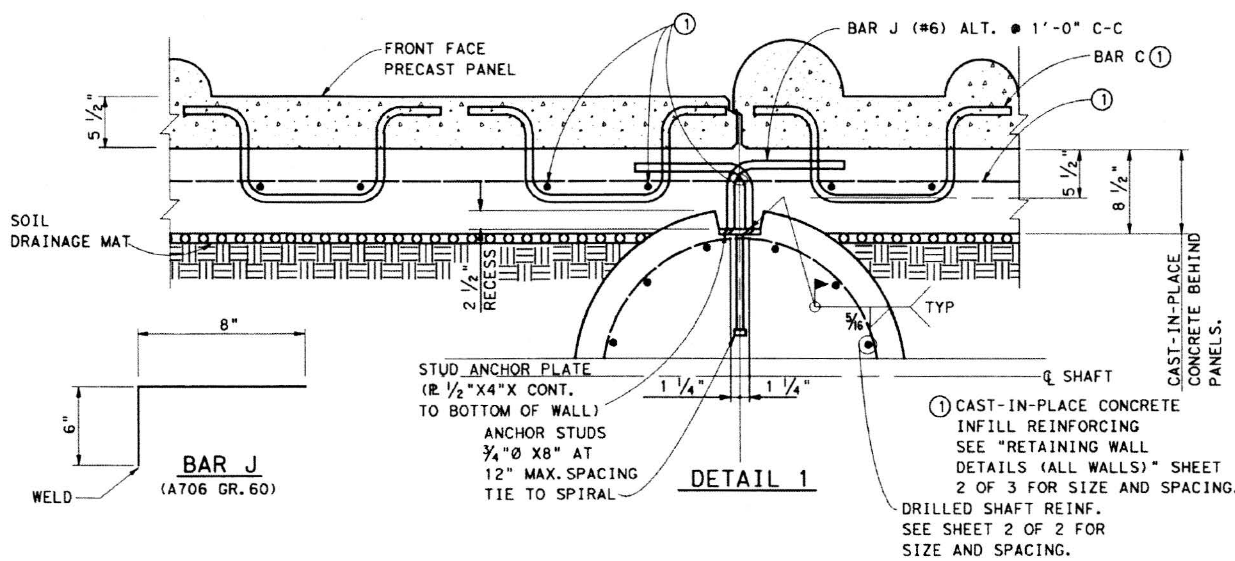


Figure 5.5: Detail of wall facing.

5.2. US 59 @ Hazard Street

5.2.1. Geotechnical Information

Across the project area, several geotechnical borings are present in the available documents. The soil profile consists primarily of stiff clay. At depths of 0 to 30 feet, the average plasticity index (PI) is approximately 40, indicating high swell potential (Department of Army 1983). Measured moisture contents in the upper 10 feet were generally between 15 and 30 percent. Based on the results of UU testing, undrained shear strengths ranged from approximately 2000 psf to 4500 psf over the depth of the wall.

5.2.2. Design Information

A secant wall consisting of alternating 48-inch and 18-inch shafts is the primary retaining structure for this project. In two locations, 48-inch shafts are used by themselves. At varying distances behind the wall, a sound wall is installed using 36-inch shafts on 5-foot center-to-center spacing (Figure 5.6). In some cases, these 36-inch shafts contribute to the strength of the main retaining structure. Details on internal shaft geometry are provided in Figure 5.7. Facing consists of precast concrete panels. The wall height across the project site is approximately 15 feet (Figure 5.8).

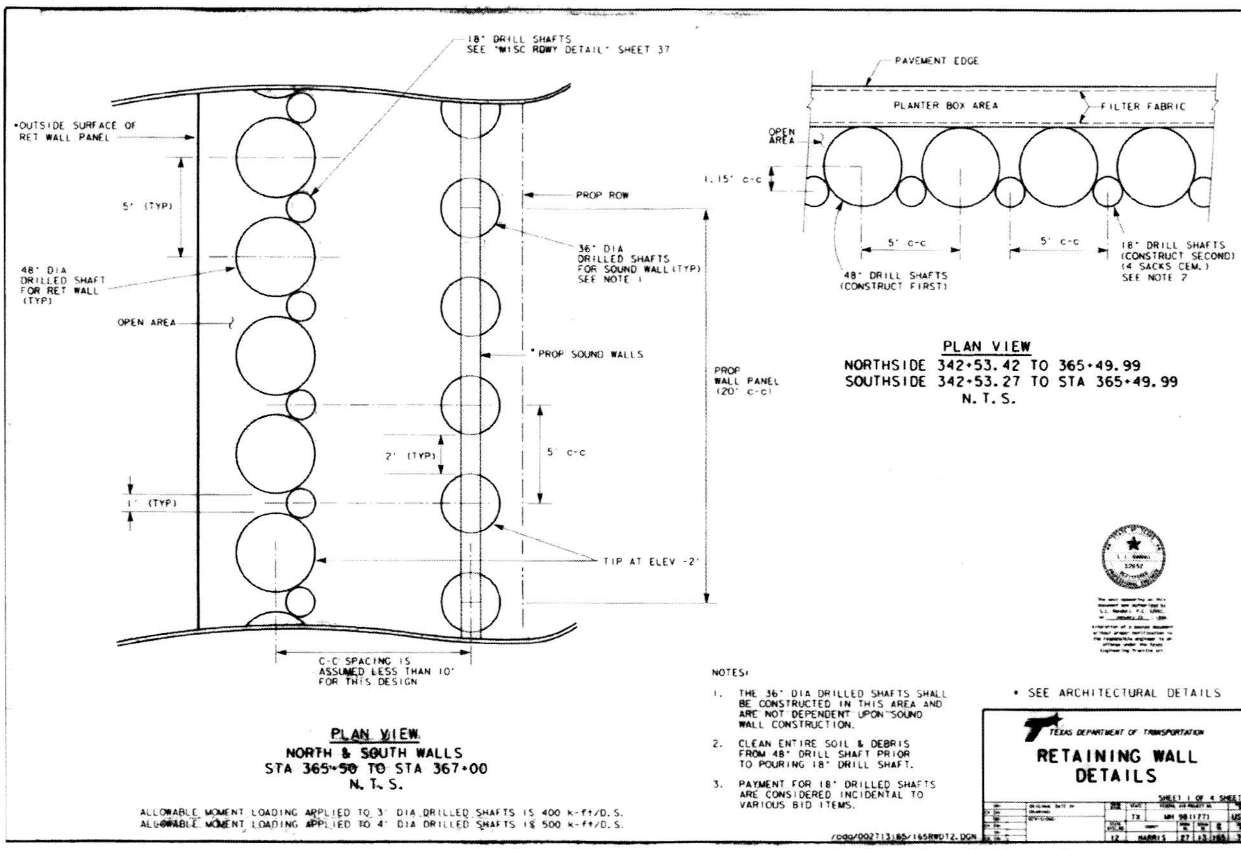
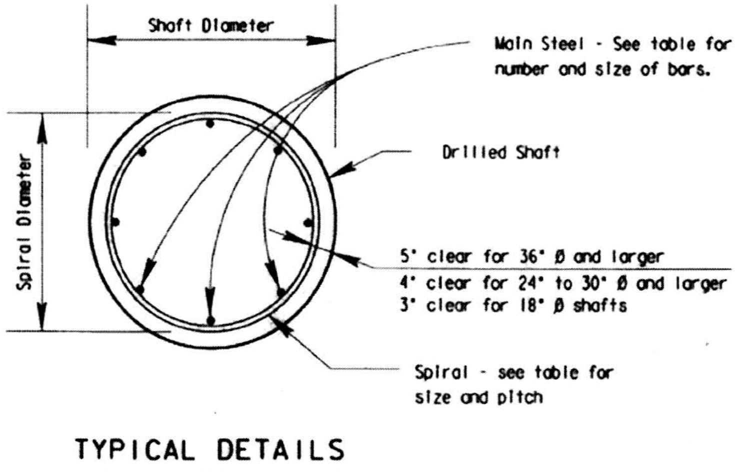
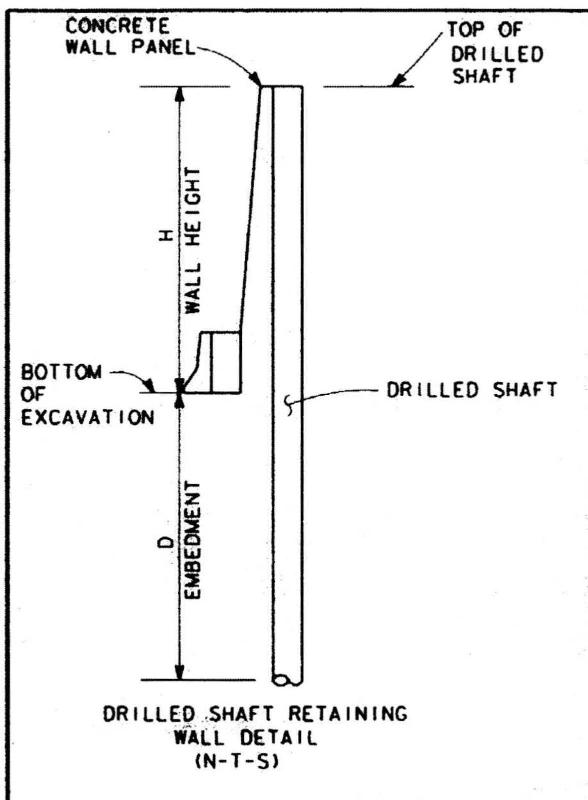


Figure 5.6: Details of drilled shafts for secant wall and sound wall foundation.



DRILLED SHAFT REINFORCING STEEL (unless noted otherwise)		
Drilled Shaft Diameter (Inches)	Main Steel	Spiral
18	6-#6	#3 at 6"
24	8-#7	#3 at 6"
30	8-#9	#3 at 6"
36	8-#10	#4 at 9"
42	12-#10	#4 at 9"
48	12-#11	#4 at 9"
54	18-#10	#4 at 9"
60	22-#10	#4 at 9"

Figure 5.7: Reinforcement and internal dimensions of drilled shafts.



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STA 343+38.75 TO 365+50</td></tr>
<tr><td colspan="6">WALL HEIGHT
H
(FT)</td><td colspan="6">EMBEDMENT
D
(FT)</td></tr>
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</table>

Figure 5.8: Typical cross section showing wall geometry.

5.3. IH 45 @ Greens Road

5.3.1. Geotechnical Information

Two boring logs are available for the project site, drilled on November 2, 1987 and November 4, 1987. The soil profile consists of stiff to very stiff clay. At depths of 0 to 30 feet, the average liquid limit (LL) is approximately 40 and the average plasticity index (PI) is approximately 23, indicating low swell potential (Department of Army 1983). Measured moisture contents in the upper 30 feet were generally between 20 and 30 percent. The results of several unconfined and UU tests, run at confining pressures up to 5000 psf, indicate shear strengths of between 1500 and 3000 psf over the depth of the wall.

5.3.2. Design Information

The retaining wall at Greens Road and IH 45 consists of a combination of cantilever drilled shaft walls and tieback walls (Figure 5.9). For this investigation, only the cantilever drilled shaft wall is considered.

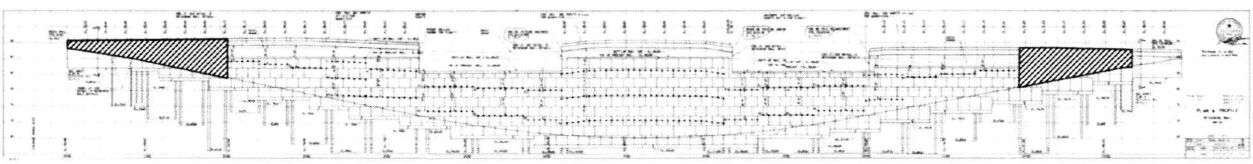

Figure 5.9: Composite sketch of project area. Shaded areas indicate locations of cantilever drilled shaft wall.

The cantilever drilled shaft wall consists of 18 to 36-inch diameter shafts with a spacing of 7 feet on center (Figure 5.10). Design heights range from approximately 3 to 10 feet, and shaft lengths range from approximately 15 to approximately 35 feet (Figure 5.11). Reinforcement details are provided in Figure 5.12. Wall facing consists of either cast-in-place concrete or precast concrete panels (Figure 5.13).

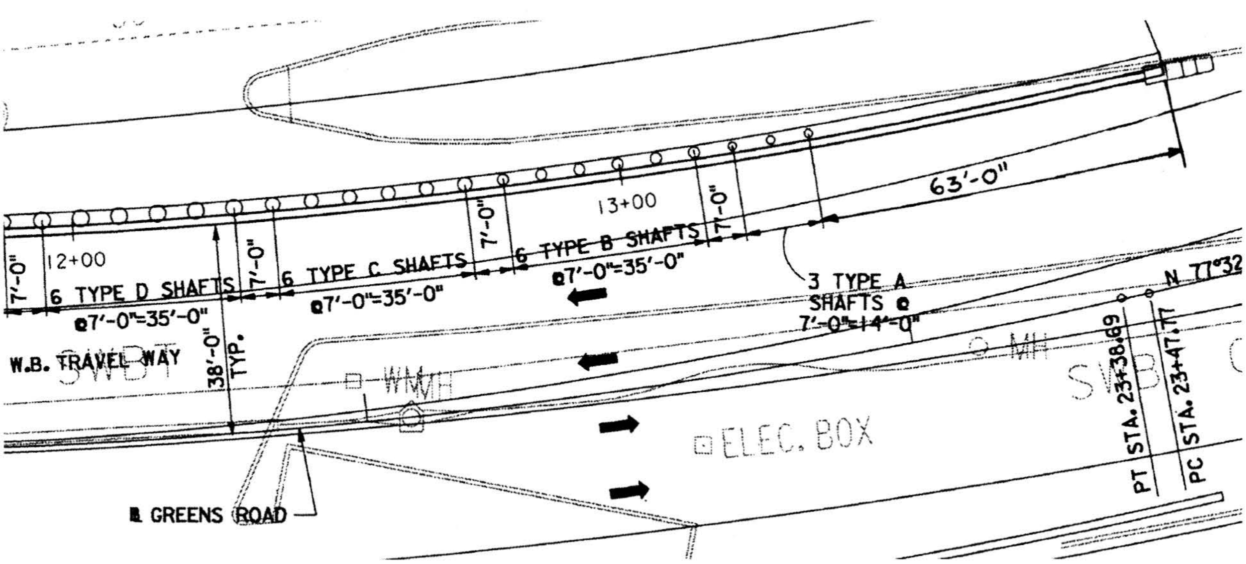

Figure 5.10: Plan view of typical drilled shaft layout.

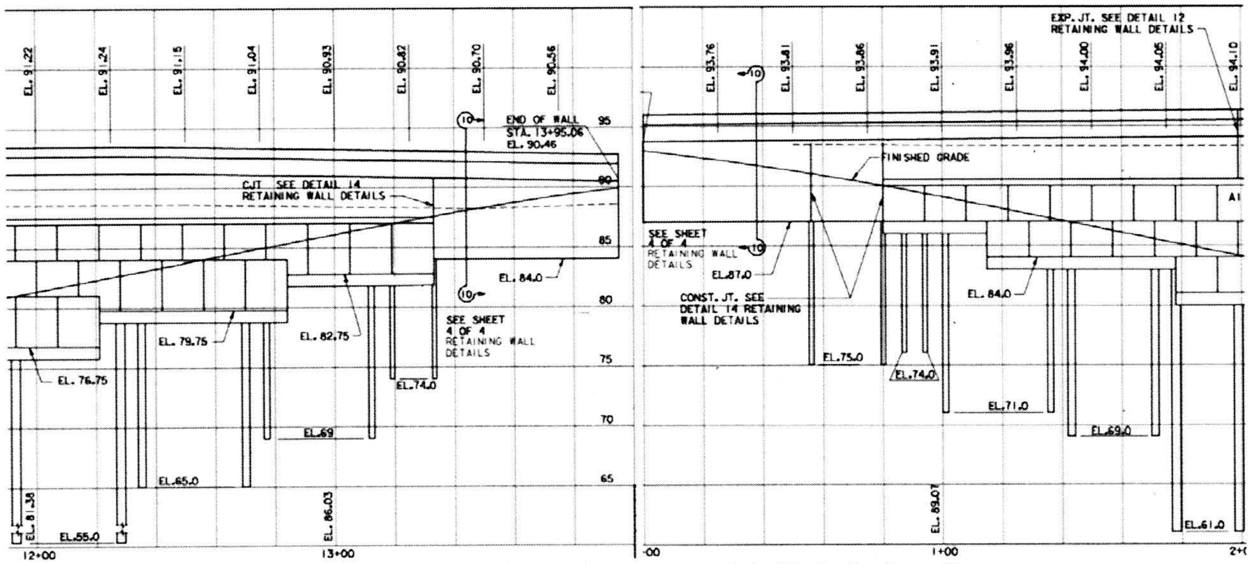

Figure 5.11: Profiles of two typical drilled shaft walls.

DRILLED SHAFT REINFORCEMENT TABLE					
TYPE	DIAMETER (INCHES)	REINFORCING STEEL		SPIRAL (SIZE OF PITCH)	REQUIRED SECTION MODULUS (IN.^{3}) *
		BARS A	BARS B		
A	18	4#7	1#6	#3 @ 18" PITCH	-
B	24	4#9	1#6	#3 @ 18" PITCH	-
C	30	6#7	1#6	#3 @ 18" PITCH	-
D	36	6#10	1#6	#3 @ 18" PITCH	-
E	30				30
F	30				40

* PER INDIVIDUAL CHANNEL

Figure 5.12: Drilled shaft reinforcement table.

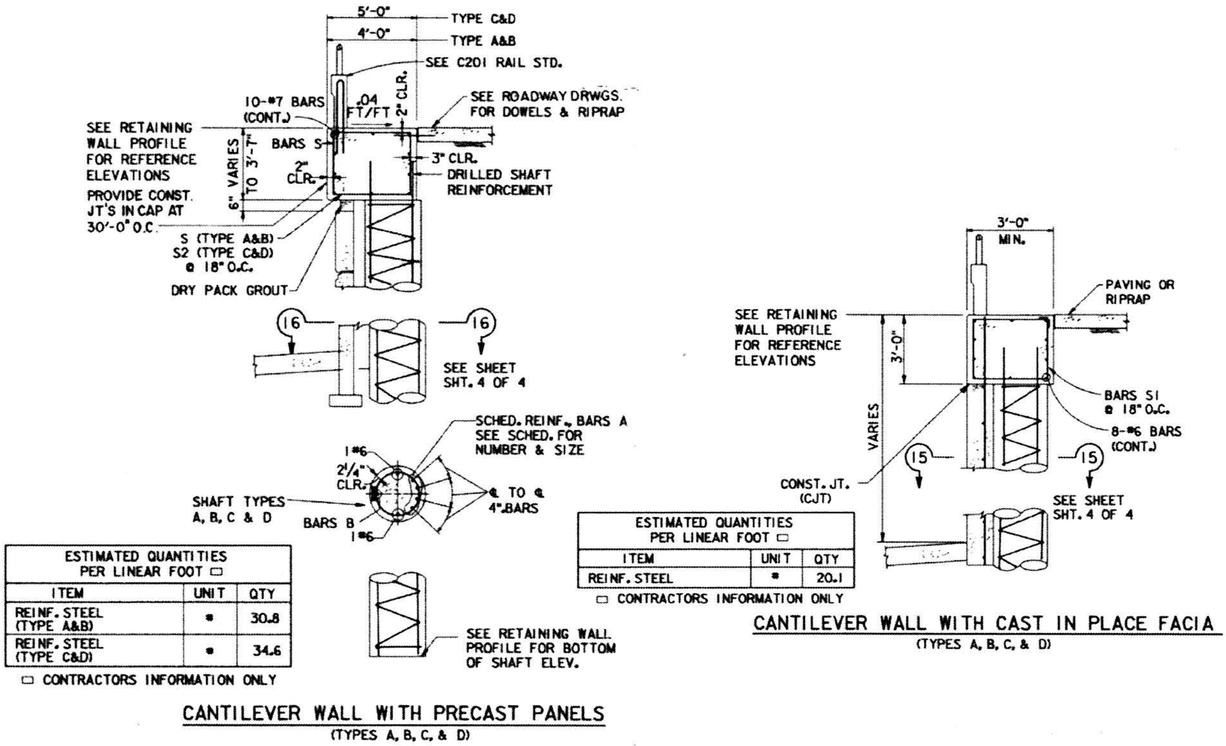

Figure 5.13: Details of precast and cast in place facing.

6. PERFORMANCE ASSESSMENT

On June 30, 2011, an assessment was conducted at the wall site. This was limited to what could be safely conducted on foot without disrupting traffic flow. As a result, most insights into wall performance are qualitative.

6.1. FM 1960 @ Kuykendahl

From a distance, the wall appears to be in excellent condition (Figure 6.1). No obvious signs of distress were observed when walking along the top and base of the wall. A four-foot carpenter's level showed the panels to be vertical (Figure 6.2). This is consistent with the overall condition of the wall.

At all cantilever drilled shaft wall locations, the retained soil is covered with pavement for at least three traffic lanes (Figure 6.3). This could limit the potential for large moisture fluctuations near the wall. There are a few grass medians in the area with widths of approximately 4 feet. The nearest location for larger scale moisture infiltration is at the southeast corner of the intersection, at least 40 feet from the nearest shafts (Figure 6.4). A closer inspection of this unpaved area shows that some potential for water ponding exists, but offers no clear evidence that it has occurred near the wall (Figure 6.5). An inspection of the wall showed no indication of differential movements in the shafts nearest to the unpaved area.

Figure 6.1: View of north and southbound walls from FM 1960 bridge (facing southeast).

Figure 6.2: Wall facing was observed to be vertical and in good condition throughout the project area.

Figure 6.3: Paved area behind the wall. In all locations, pavement extends at least 30 feet behind the wall.

Figure 6.4: View of nearest location for moisture infiltration, at southeast corner of intersection.

Figure 6.5: Unpaved area near southeast corner of intersection.

6.2. US 59 @ Hazard Street

A large scale view of the project area is presented in Figure 6.6. Because the facing was installed at an angle and is currently covered with dense vegetation, very little information about the shafts can be obtained from road level (Figure 6.7).

Figure 6.6: Large-scale view of project area (facing east). Sound wall is located above the main retaining structure.

Figure 6.7: Dense vegetation and batter angle on concrete facing panels. Sound wall is located above main retaining structure.

An inspection of the area behind the north wall near Hazard Street showed approximately 40 feet of exposed soil between the drilled shaft wall and the sound wall (Figure 6.8). Along the length of the project site, the distance between the drilled shaft wall and the sound wall can range from 0 to approximately 50 feet. At several locations, gaps were observed between the retained soil and the shafts (Figure 6.9). Some of these gaps were up to 8 feet deep and 1 foot back from the wall. The potential for water to drain into these gaps during heavy rainfall seems high.

The soil behind the wall is well vegetated and water flow is directed to large grates leading to an underdrain system. Of the walls we studied, this wall appears to have the largest potential for moisture change. Despite this, every measurement that could be made indicated that the wall is vertical and no major red flags were observed (apart from the gaps between the soil and the wall).

Figure 6.8: Approximately 40 feet of exposed soil behind the wall near Hazard Street (facing west).

Figure 6.9: Several gaps were observed between the shafts and the soil behind the wall.

6.3. IH 45 @ Greens Road

From a distance, the wall appears to be in good condition (Figure 6.10). No obvious signs of structural distress were observed when walking along the top and base of the wall, but some superficial facing damage was observed. Most of this damage appears to be age related and not caused by any structural distress on the wall. Some imperfections in the white concrete facing appear to be caused by seams in the concrete formwork (Figure 6.11). Some cracking and differential settlement is present in the sidewalks near the wall, but no signs of corresponding wall distress were observed at these locations (Figure 6.12). Some concrete cracking was seen at the connection between two facing elements (Figure 6.13).

Figure 6.10: View of current wall conditions, facing west.

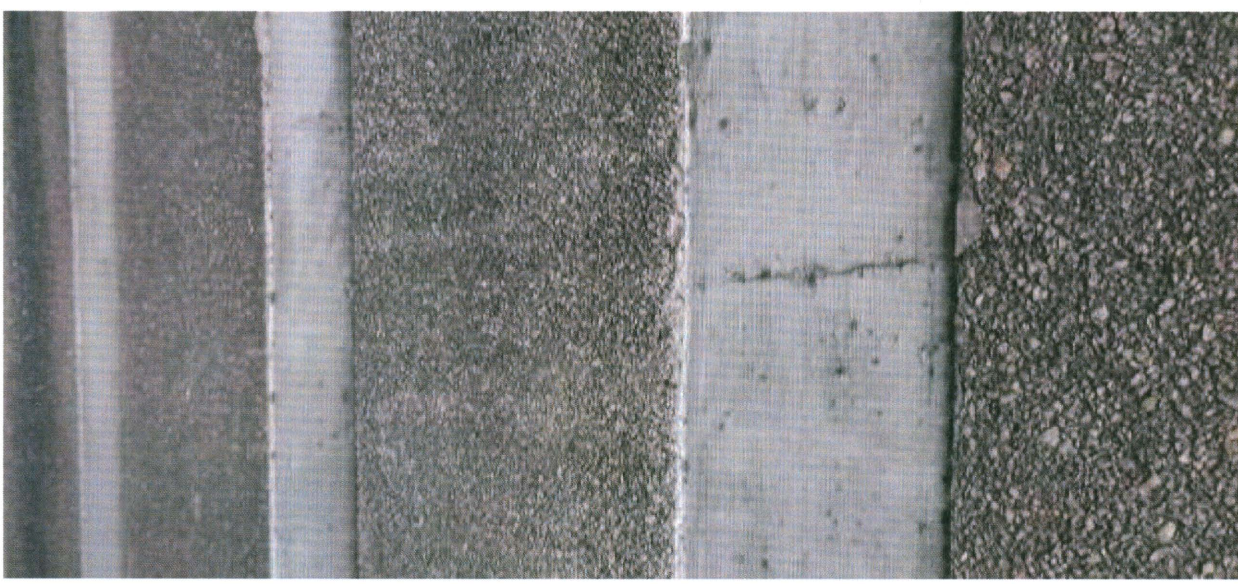

Figure 6.11: Example of imperfection in white concrete facing, possibly caused by a seam in the plywood formwork.

Figure 6.12: View of sidewalk damage along the base of the wall.

Figure 6.13: Some cracking was seen at the connection between two facing panels on the southwestern portion of the wall.

At least 30 feet of pavement covers the soil behind the wall in all directions (Figure 6.14). This could limit the potential for large moisture fluctuations near the wall. There are a few grass medians in the area, and runoff is directed into a system of storm drains. The extent of pavement near the wall is shown in Figures 6.15 and 6.16.

Figure 6.14: View of pavement coverage along southwestern side of intersection (facing west).

Figure 6.15: Aerial view of western portion of wall (Google Inc., 2011).

Figure 6.16: Aerial view of eastern portion of wall (Google Inc., 2011).

7. L-PILE ANALYSES

An L-Pile analysis was performed for each wall in order to predict the effects of different loads applied to the walls. These analyses were done on the portions of the walls that were just cantilever drilled shafts. For clay layers within five feet below the excavation line, the undrained shear strength (Su) was reduced by 50% to account for the reduction in strength from the reduction in vertical stress due to the excavation of soil above. This reduction in strength is consistent with the TxDOT Manual. Loading scenarios considered were equivalent fluid pressures of 40, 60, and 80 psf/ft, each with a surcharge load directly behind the wall equivalent to two feet of 120-pcf soil.

7.1. FM 1960 @ Kuykendahl

Seven different shaft layouts were used for this analysis. Soil, shaft, and loading properties used in this analysis are presented in Tables 7.1 and 7.2. Results show that the shaft layout that deflected the most relative to the height of the wall was the wall layout with shaft lengths of 52 feet and a height of 23 feet. At an equivalent fluid pressure of 40 psf/ft with two feet of soil surcharge, the top-of-wall deflection was 1.22 percent of the wall height (3.36 inches). At an equivalent fluid pressure of 80 psf/ft with two feet of surcharge, the wall deflected 2.96 percent (8.17 inches) of the wall height. The shortest shafts analyzed (L=31 ft, H=12 ft), deflected 0.16 percent of the wall height at an equivalent fluid pressure of 40 psf/ft. Results from the L-Pile analysis are summarized in Figures 7.1–7.7.

Table 7.1: Input Soil Properties for FM 1960 @ Kuykendahl

Layers	Soil Model	Top Depth	Bot Depth	γ , pcf	φ', deg	c, psf
1	Stiff Clay	0	10	120	-	3600
2	Sand	10	25	110	35	0
3	Stiff Clay	25	35	120	-	3600
4	Sand	35	75	110	35	0

Table 7.2: Shaft and loading properties used in L-Pile analysis for FM 1960 and Kuykendahl.

	Shaft				Reinforcement						Concrete		Loading	
Case	Total Length, ft	Wall Height, ft	Diam, in	C-C Spacing (ft)	Size, #	# of bars	# per group	concrete cover, in	σy , psi	E, psi	σc , psi	Max Agg. Size, in	Load, psf/ft	Surcharge, ft
1	**52**	**23**	**36**	**3.916667**	**11**	**18**	**2**	**5**	**60000**	**29000000**	**4000**	**0.75**	**40**	**2**
2	52	23	36	3.916667	11	18	2	5	60000	29000000	4000	0.75	60	2
3	52	23	36	3.916667	11	18	2	5	60000	29000000	4000	0.75	80	2
4	**49**	**22**	**36**	**3.916667**	**11**	**16**	**2**	**5**	**60000**	**29000000**	**4000**	**0.75**	**40**	**2**
5	49	22	36	3.916667	11	16	2	5	60000	29000000	4000	0.75	60	2
6	49	22	36	3.916667	11	16	2	5	60000	29000000	4000	0.75	80	2
7	**46**	**20**	**36**	**3.916667**	**11**	**14**	**1**	**5**	**60000**	**29000000**	**4000**	**0.75**	**40**	**2**
8	46	20	36	3.916667	11	14	1	5	60000	29000000	4000	0.75	60	2
9	46	20	36	3.916667	11	14	1	5	60000	29000000	4000	0.75	80	2
10	**43**	**18**	**36**	**3.916667**	**11**	**12**	**1**	**5**	**60000**	**29000000**	**4000**	**0.75**	**40**	**2**
11	43	18	36	3.916667	11	12	1	5	60000	29000000	4000	0.75	60	2
12	43	18	36	3.916667	11	12	1	5	60000	29000000	4000	0.75	80	2
13	**39**	**16**	**36**	**3.916667**	**9**	**12**	**1**	**5**	**60000**	**29000000**	**4000**	**0.75**	**40**	**2**
14	39	16	36	3.916667	9	12	1	5	60000	29000000	4000	0.75	60	2
15	39	16	36	3.916667	9	12	1	5	60000	29000000	4000	0.75	80	2
16	**36**	**14**	**36**	**3.916667**	**9**	**10**	**1**	**5**	**60000**	**29000000**	**4000**	**0.75**	**40**	**2**
17	36	14	36	3.916667	9	10	1	5	60000	29000000	4000	0.75	60	2
18	36	14	36	3.916667	9	10	1	5	60000	29000000	4000	0.75	80	2
19	**31**	**12**	**36**	**3.916667**	**9**	**10**	**1**	**5**	**60000**	**29000000**	**4000**	**0.75**	**40**	**2**
20	31	12	36	3.916667	9	10	1	5	60000	29000000	4000	0.75	60	2
21	31	12	36	3.916667	9	10	1	5	60000	29000000	4000	0.75	80	2

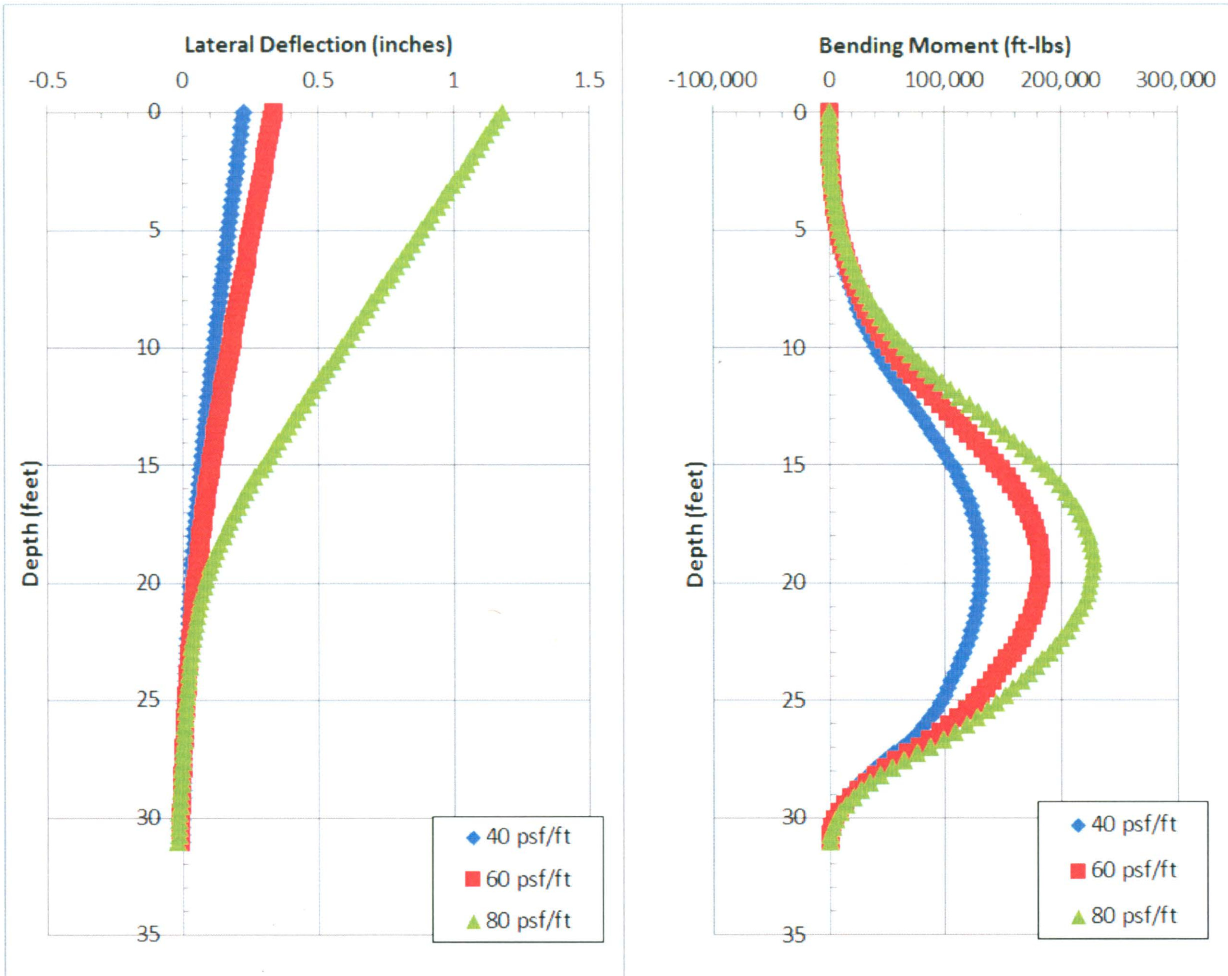

Figure 7.1: Results of L-Pile analysis for FM 1960 @ Kuykendahl; shaft length = 31 feet; wall height = 12 feet.

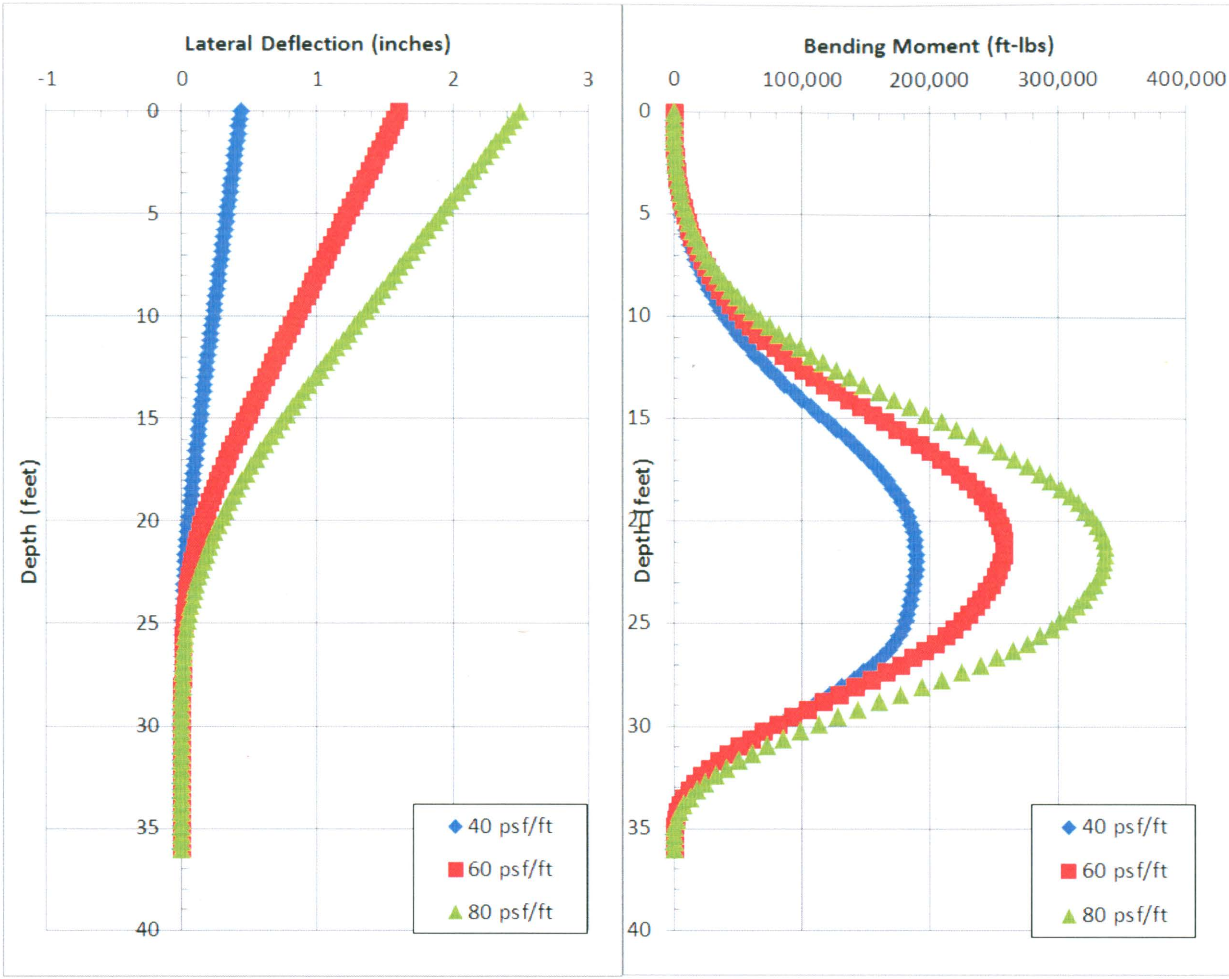

Figure 7.2: Results of L-Pile analysis for FM 1960 @ Kuykendahl; shaft length = 36 feet; wall height = 14 feet.

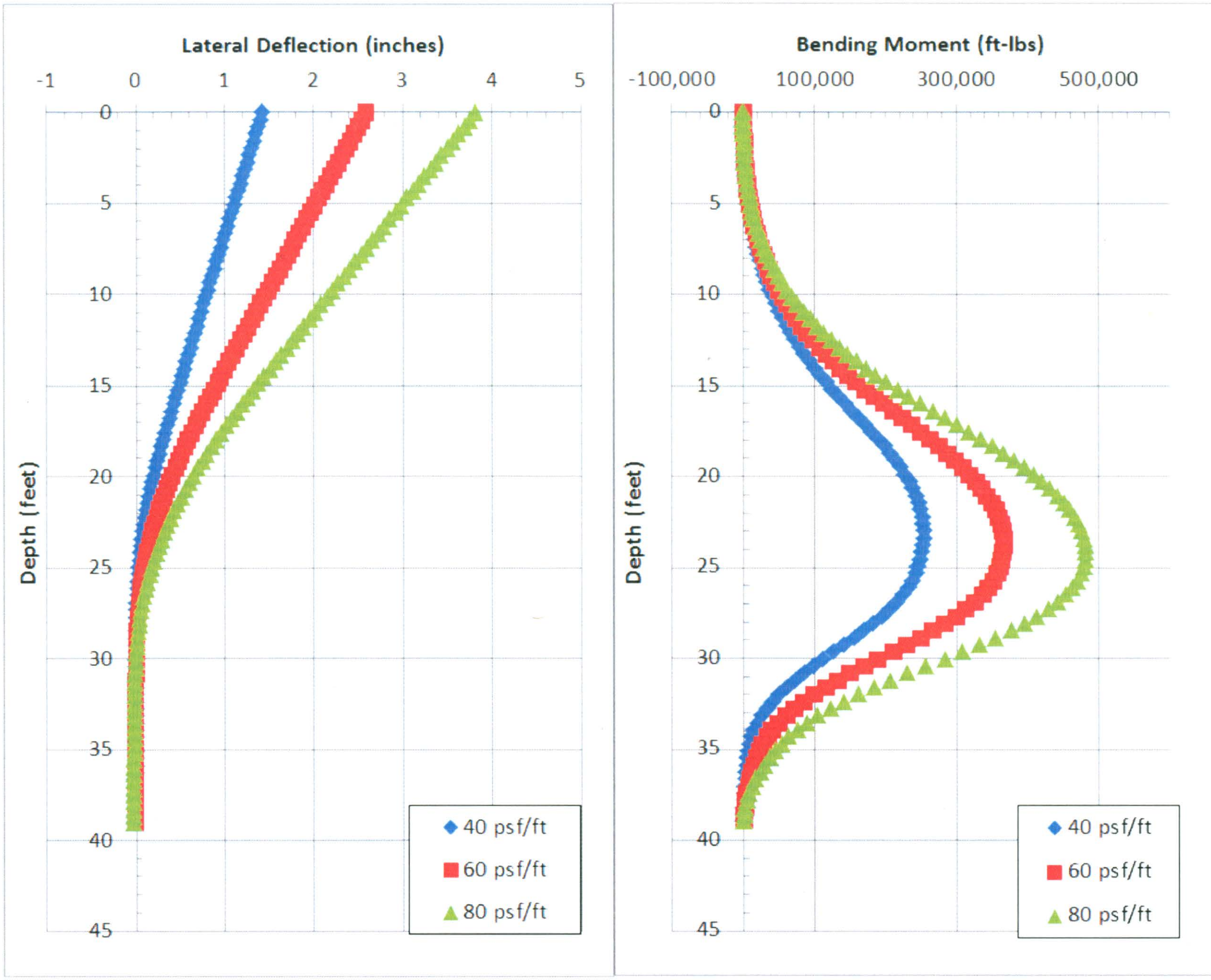

Figure 7.3: Results of L-Pile analysis for FM 1960 @ Kuykendahl; shaft length = 39 feet; wall height = 16 feet.

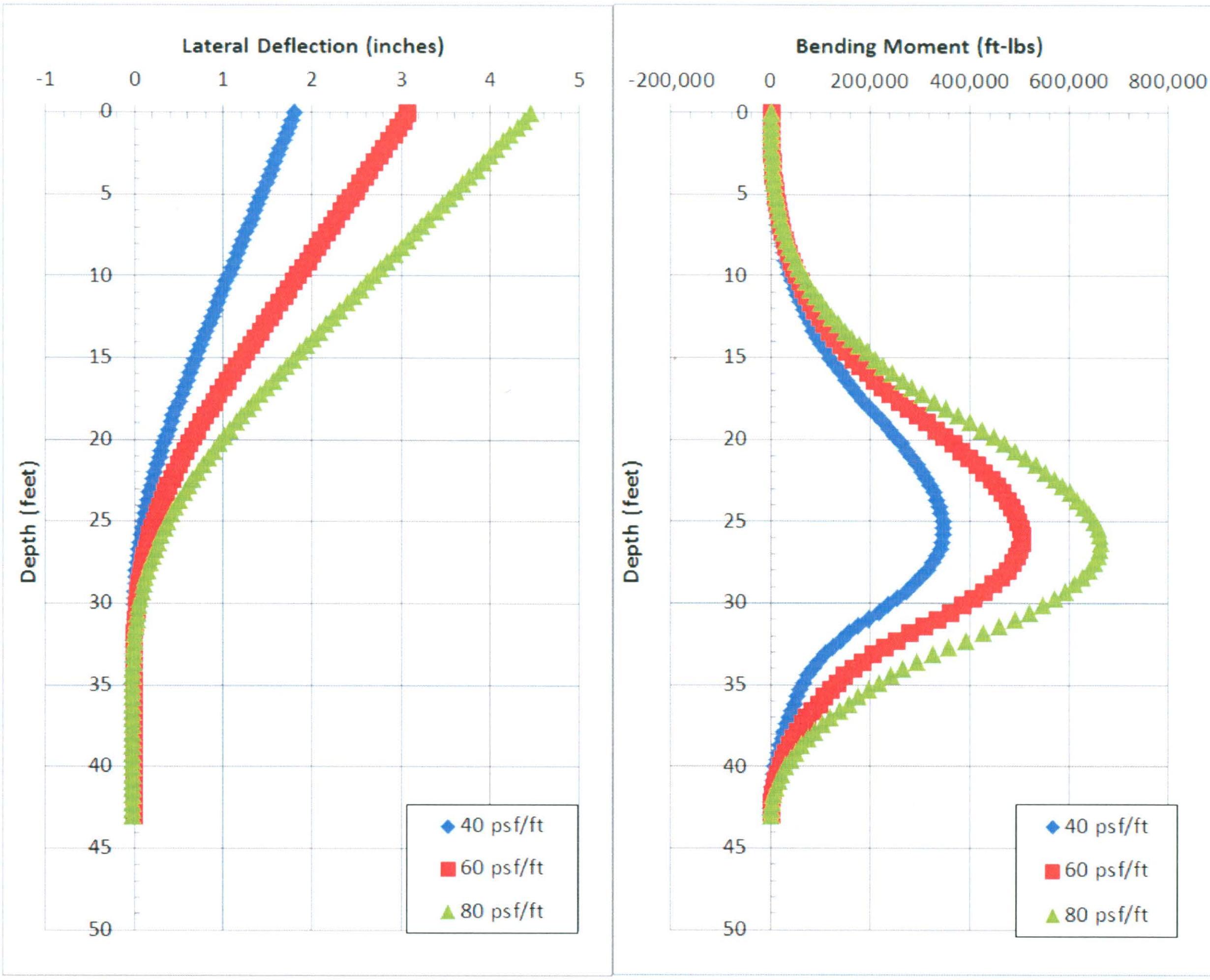

Figure 7.4: Results of L-Pile analysis for FM 1960 @ Kuykendahl; shaft length = 43 feet; wall height = 18 feet.

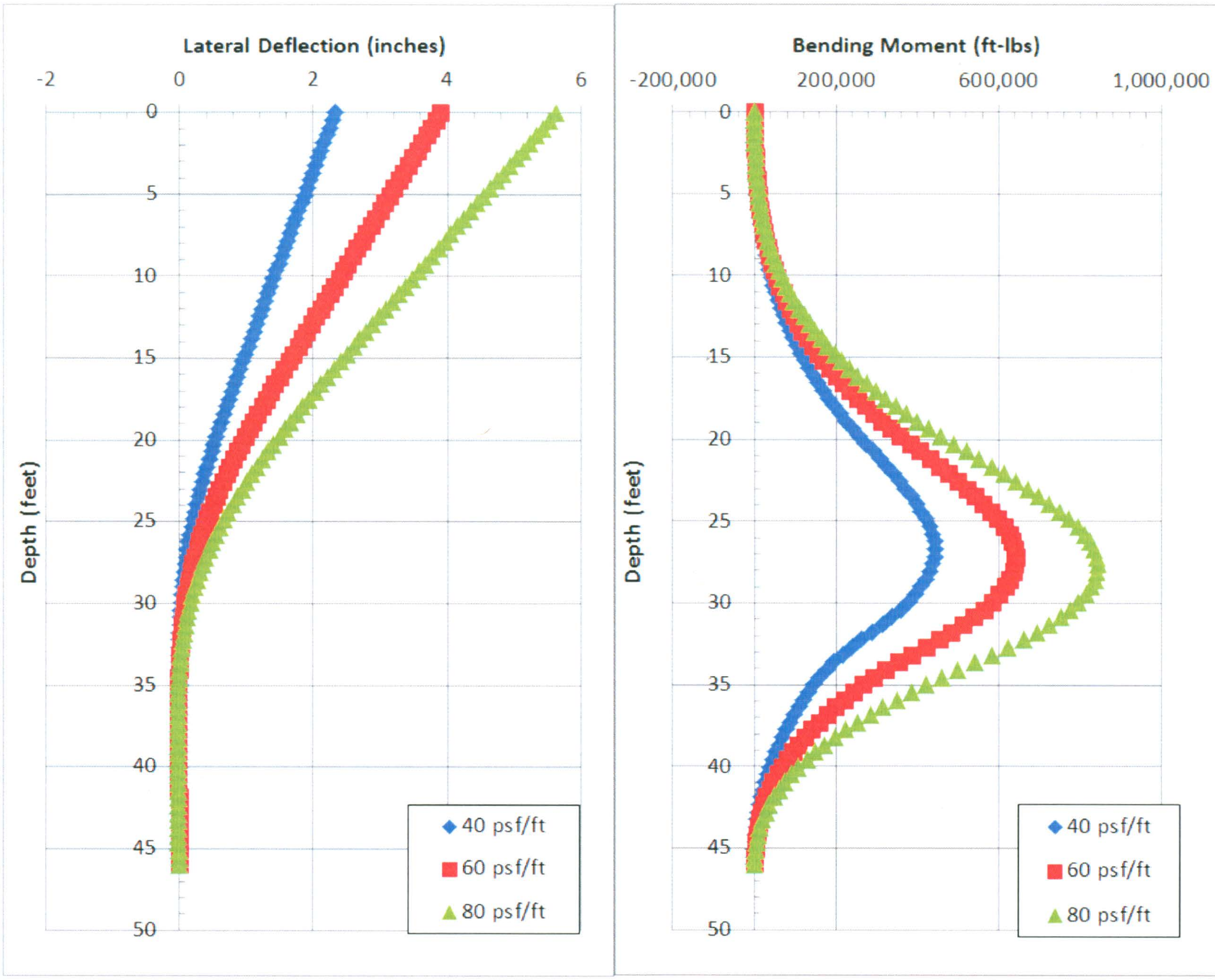

Figure 7.5: Results of L-Pile analysis for FM 1960 @ Kuykendahl; shaft length = 46 feet; wall height = 20 feet.

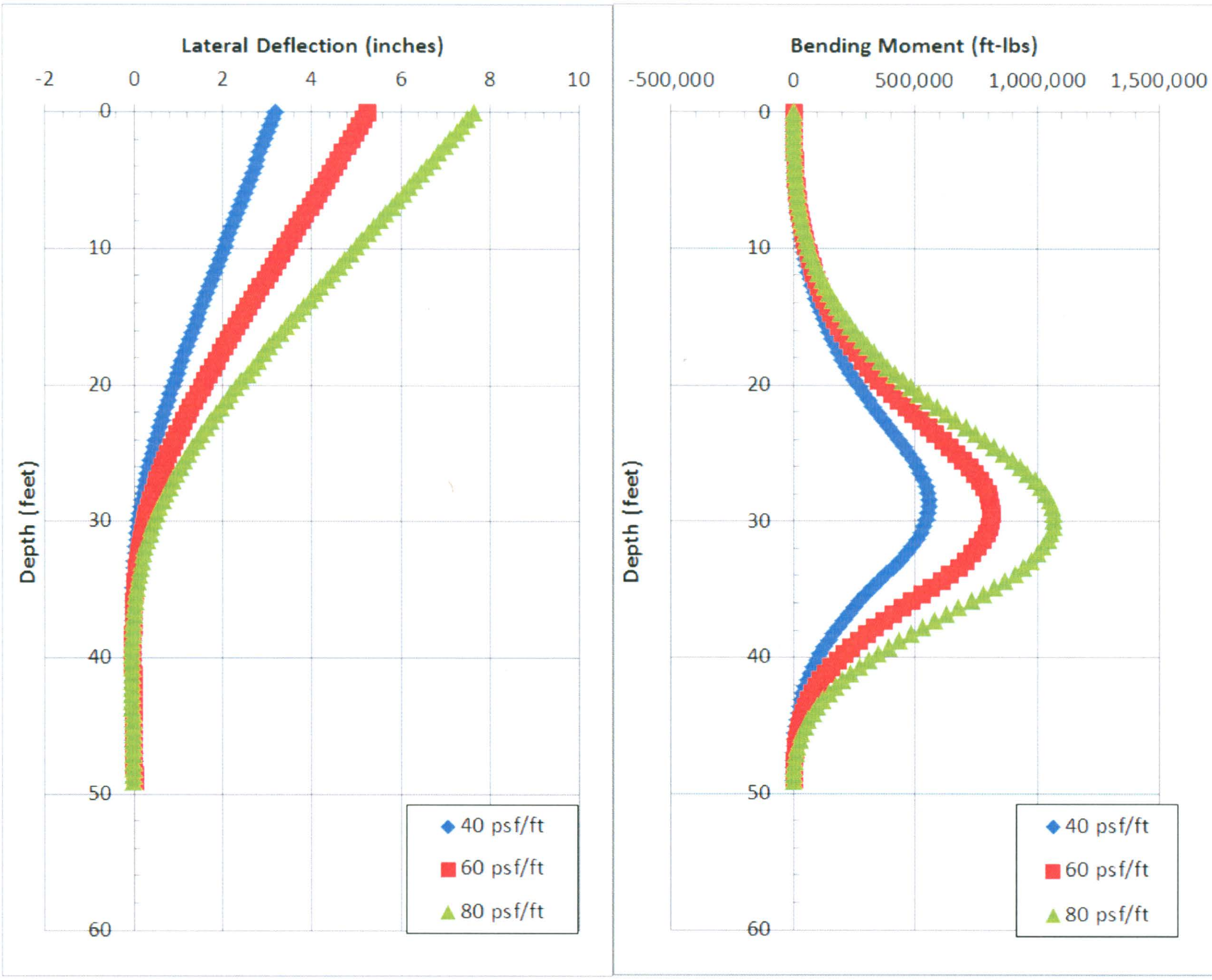

Figure 7.6: Results of L-Pile analysis for FM 1960 @ Kuykendahl; shaft length = 49 feet; wall height = 22 feet.

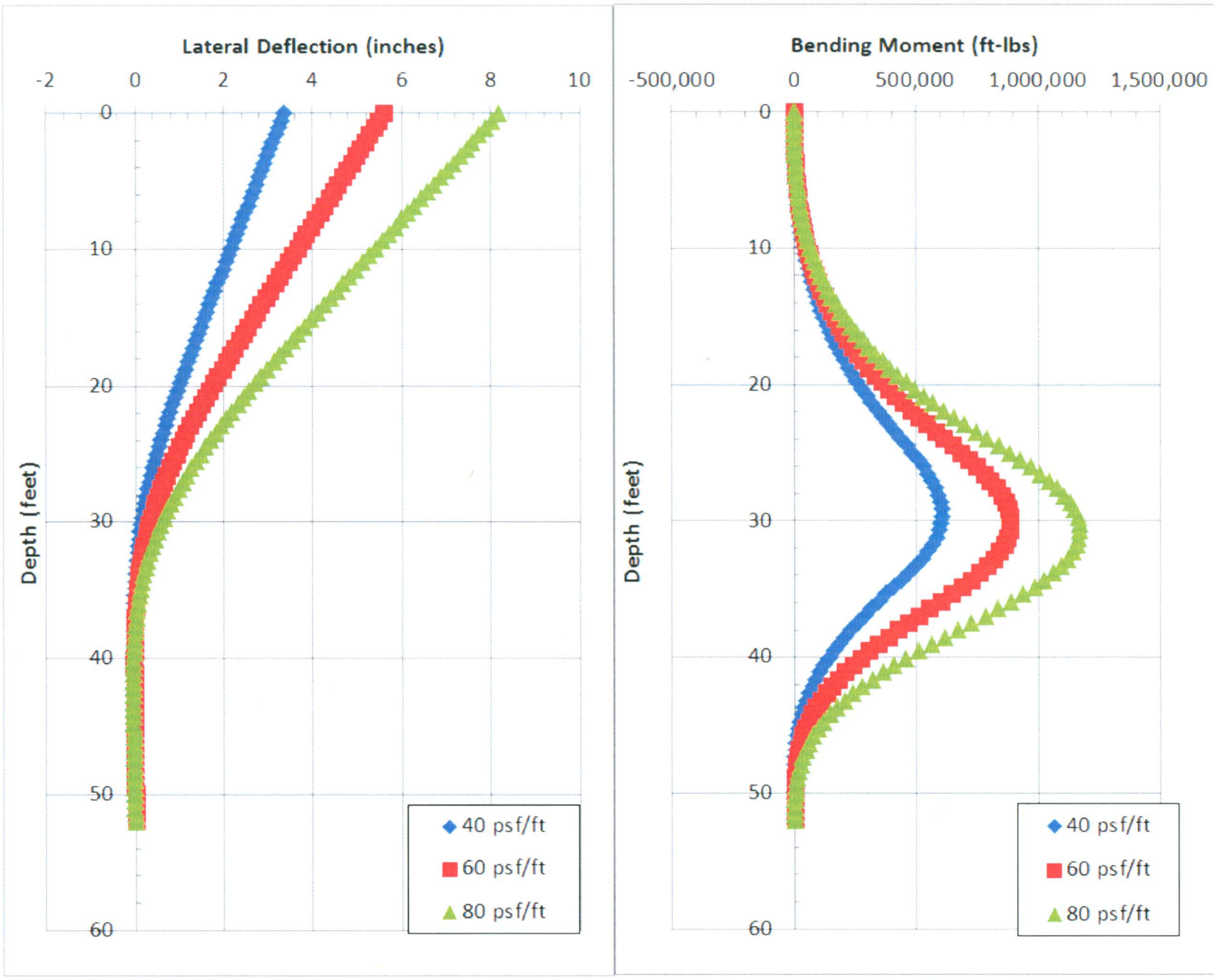

Figure 7.7: Results of L-Pile analysis for FM 1960 @ Kuykendahl; shaft length = 52 feet; wall height = 23 feet.

7.2. US 59 @ Hazard Street

Because of the large size of the shafts and potential for moisture changes on the project site, the wall layout was analyzed using one additional loading scenario consisting of an equivalent fluid pressure of 100 psf/ft with two feet of surcharge. Soil, shaft, and loading properties used in this analysis are presented in Tables 7.3 and 7.4. Results show that the wall will not deflect more than one percent of the wall height for any of the loading conditions. Even at an equivalent fluid pressure of 100 psf/ft with two feet of surcharge, the wall deflects just 0.3 percent of the wall height (0.55 inches). Results from the L-Pile analysis are summarized in Figure 7.8.

Table 7.3: Soil properties used in L-Pile analysis for US 59 and Hazard St.

Layers	Soil Model	Top Depth	Bot Depth	γ , pcf	c, psf
1	Clay	0	7	125	4400
2	Clay	7	11.5	120	2500
3	Clay	11.5	16	130	2150
4	Clay	16	21.5	135	3500
5	Clay	21.5	26	130	2800
6	Clay	26	75	125	2700

Table 7.4: Shaft and loading properties used in L-Pile analysis for US 59 and Hazard St.

Case	Shaft				Reinforcement						Concrete		Loading	
	Total Length, ft	Wall Height, ft	Diam, in	C-C Spacing (ft)	Size, #	# of bars	# per group	concrete cover, in	σy , psi	E, psi	σc , psi	Max Agg. Size, in	Load, psf/ft	Surcharge, ft
1	47.5	15	48	5	11	12	1	5	60000	29000000	4000	0.75	40	2
2	47.5	15	48	5	11	12	1	5	60000	29000000	4000	0.75	60	2
3	47.5	15	48	5	11	12	1	5	60000	29000000	4000	0.75	80	2
4	47.5	15	48	5	11	12	1	5	60000	29000000	4000	0.75	100	2

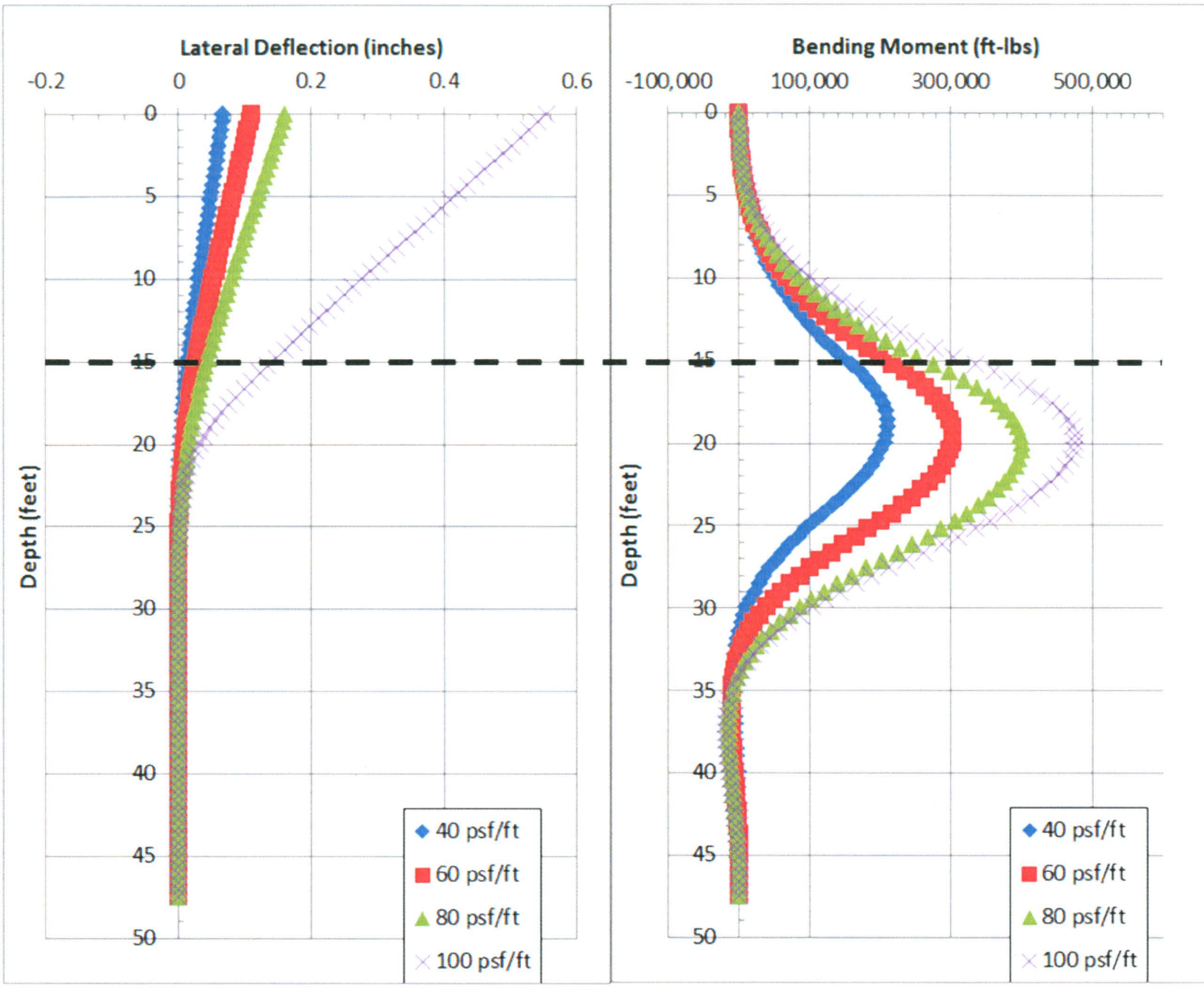

Figure 7.8: Results of L-Pile analysis for US 59 and Hazard Street; shaft length = 47.5 feet, wall height = 15 feet. Dashed line indicates excavation depth.

7.3. IH 45 @ Greens Road

Four different shaft layouts were used for this analysis. Soil, shaft, and loading properties are presented in Tables 7.5 and 7.6. Results show that for the 40 psf/ft loading scenario, the top-of-wall deflections are less than one percent of the wall height for each of the four different layouts. For wall heights of 5, 7, and 9 feet (total shaft lengths of 18, 19, and 25 feet, respectively), the top-of-wall deflections did not exceed one percent of the wall height until the 80 psf/ft loading scenario. For a wall height of 11 feet (total shaft length of 34 feet), the maximum top-of-wall deflection was 0.9 percent of the wall height for the 80 psf/ft loading scenario.

It should be noted that determining the shaft lengths and wall heights were estimated based on the design files provided by TxDOT. This estimation was due to the lack of clear documentation in the design files. The analysis of the wall with a height of 7 feet and a total shaft length of 19 feet shows that embedment depth is not sufficient to reach fixity for all the loading scenarios. It is likely that either the as-built wall height was smaller than reported, or the as-built shaft length was larger than reported, but without supporting documents that clearly indicate the layout of the shafts, this cannot be determined with certainty. Results from the L-Pile analysis are summarized in Figures 7.9–7.12.

Table 7.5: Soil properties used in L-Pile analysis for IH 45 @ Greens Rd

				Location 1		Location 2		Aveage	
Layers	Soil Model	Top Depth	Bot Depth	γ, pcf	c, psf	γ, pcf	c, psf	γ, pcf	c, psf
1	Stiff Clay	0	5	125	1510	125	1650	125	1580
2	Stiff Clay	5	8	125	850	125	1850	125	1350
3	Stiff Clay	8	14	125	2000	125	2750	125	2375
4	Clay	14	24	130	1600	125	1080	127.5	1340
5	Clay	24	42	130	2860	125	2860	127.5	2860

Table 7.6: Shaft and loading properties used in L-Pile analysis for IH 45 @ Greens Rd

	Shaft				Reinforcement						Concrete		Loading	
Case	Total Length, ft	Wall Height, ft	Diam, in	C-C Spacing (ft)	Size, #	# of bars	# per group	concrete cover, in	σ_y, psi	E, psi	σ_c, psi	Max Agg. Size, in	Load, psf/ft	Surcharge, ft
1	**18**	**5**	**18**	**8**	**7**	**7**	**1**	**2.25**	**60000**	**29000000**	**4000**	**0.75**	**40**	**2**
2	18	5	18	8	7	7	1	2.25	60000	29000000	4000	0.75	60	2
3	18	5	18	8	7	7	1	2.25	60000	29000000	4000	0.75	80	2
4	**19**	**7**	**24**	**7**	**9**	**7**	**1**	**2.25**	**60000**	**29000000**	**4000**	**0.75**	**40**	**2**
5	19	7	24	7	9	7	1	2.25	60000	29000000	4000	0.75	60	2
6	19	7	24	7	9	7	1	2.25	60000	29000000	4000	0.75	80	2
7	**25**	**9**	**30**	**7**	**7**	**9**	**1**	**2.25**	**60000**	**29000000**	**4000**	**0.75**	**40**	**2**
8	25	9	30	7	7	9	1	2.25	60000	29000000	4000	0.75	60	2
9	25	9	30	7	7	9	1	2.25	60000	29000000	4000	0.75	80	2
10	**34**	**11**	**36**	**7**	**10**	**9**	**1**	**2.25**	**60000**	**29000000**	**4000**	**0.75**	**40**	**2**
11	34	11	36	7	10	9	1	2.25	60000	29000000	4000	0.75	60	2
12	34	11	36	7	10	9	1	2.25	60000	29000000	4000	0.75	80	2

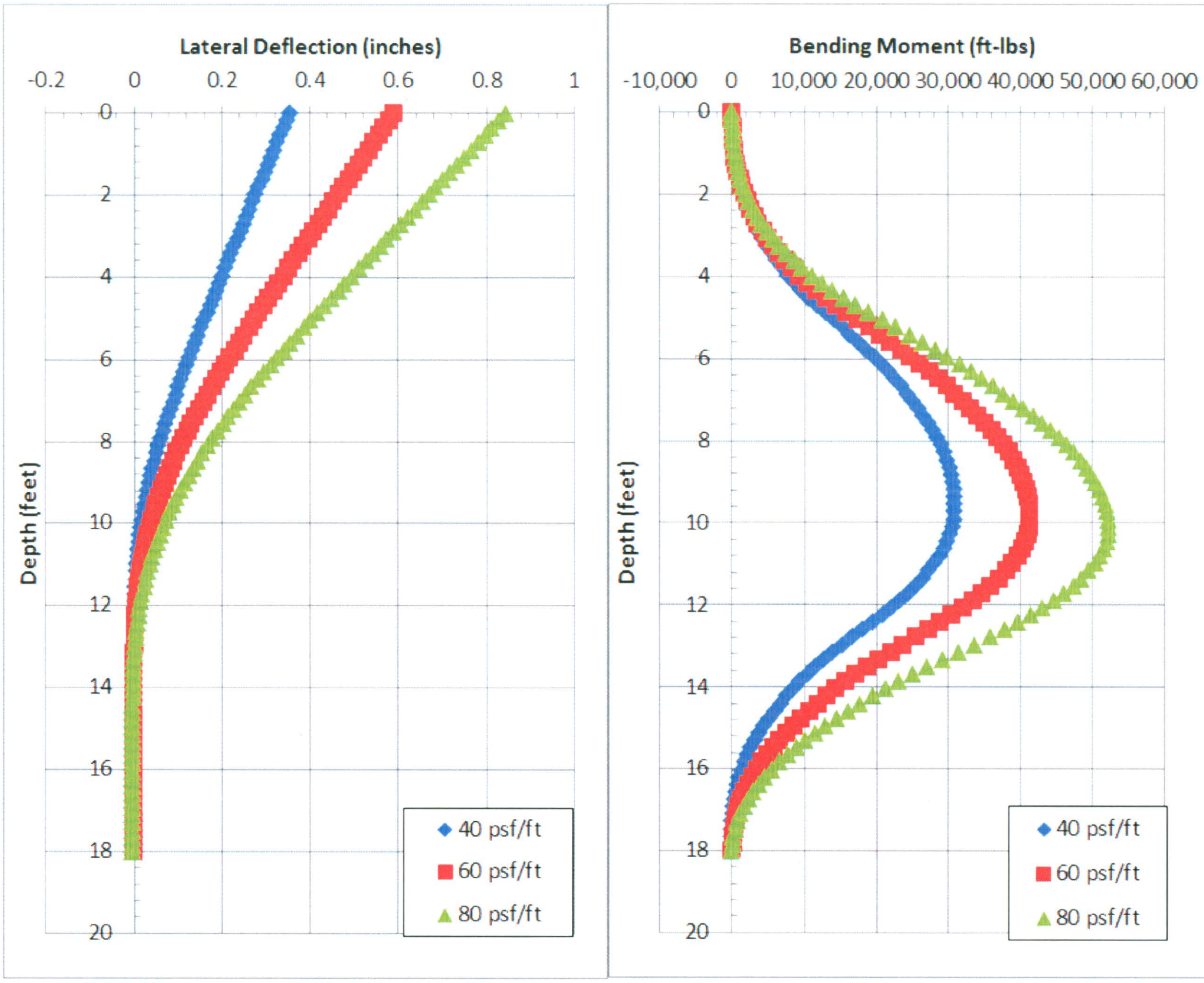

Figure 7.9: Results of L-Pile analysis for Greens Rd @ IH 45; shaft length = 18 feet, wall height = 5 feet.

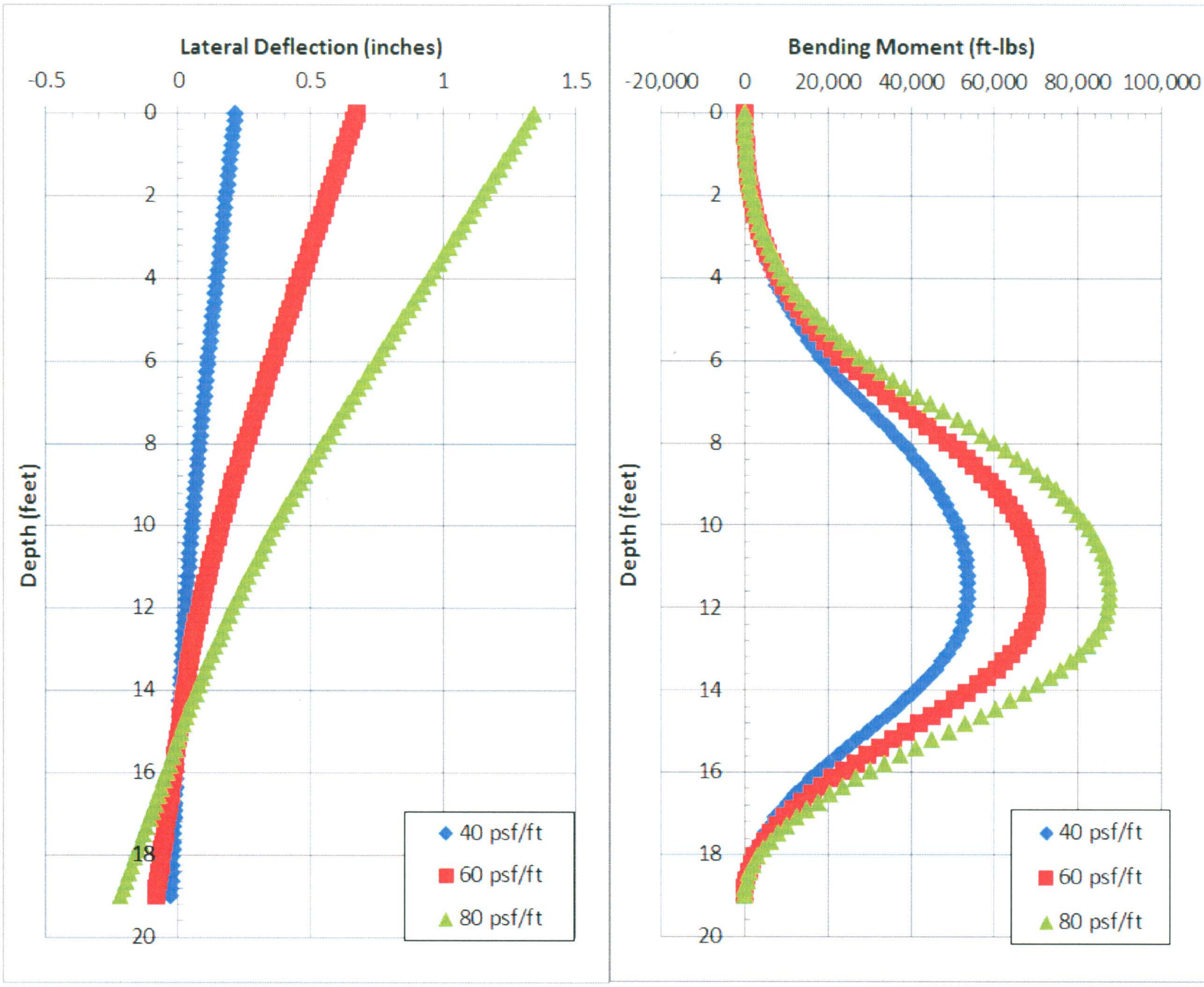

Figure 7.10: Results of L-Pile analysis for Greens Rd @ IH 45; shaft length = 19 feet, wall height = 7 feet.

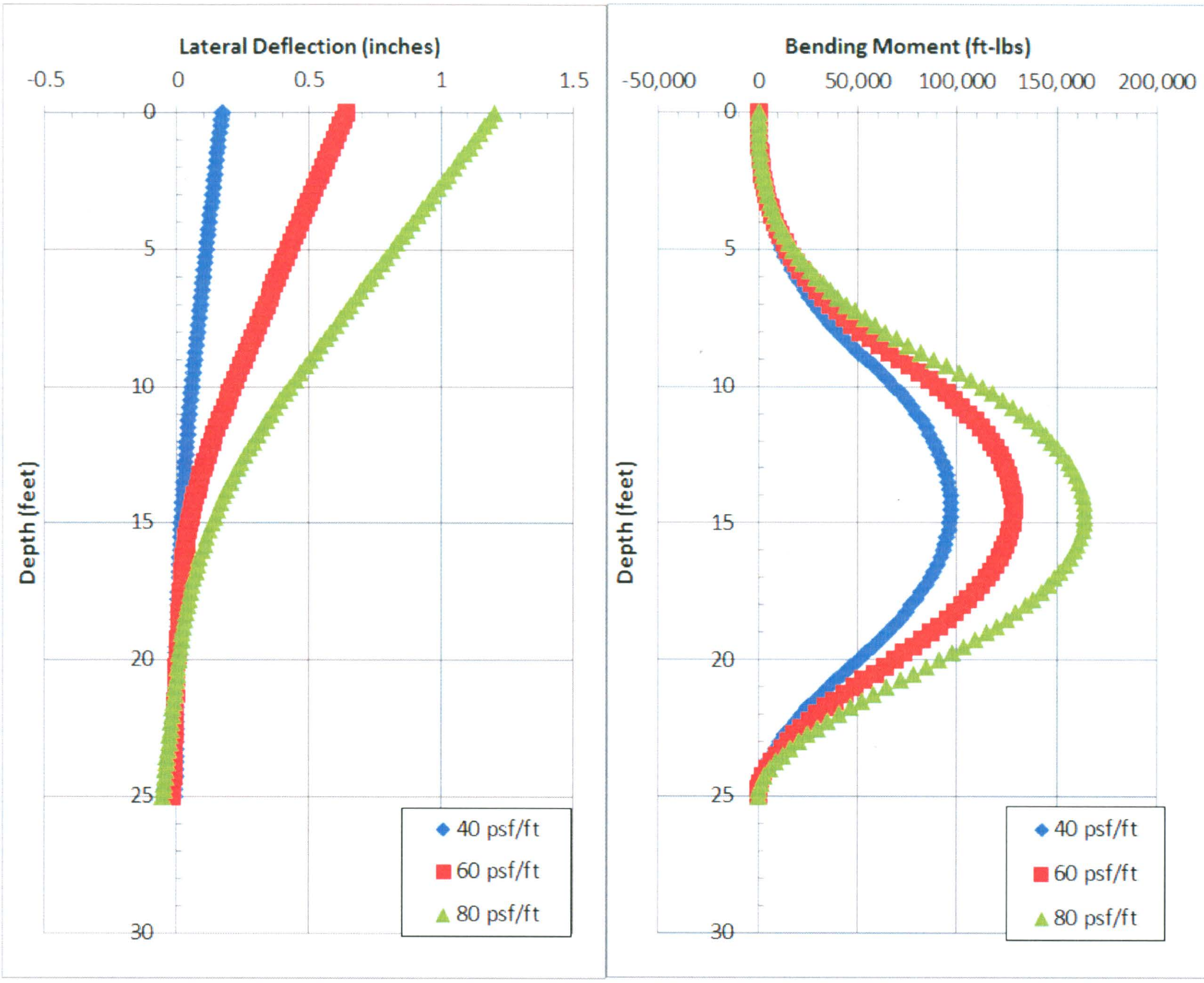

Figure 7.11: Results of L-Pile analysis for Greens Rd @ IH 45; shaft length = 25 feet, wall height = 9 feet.

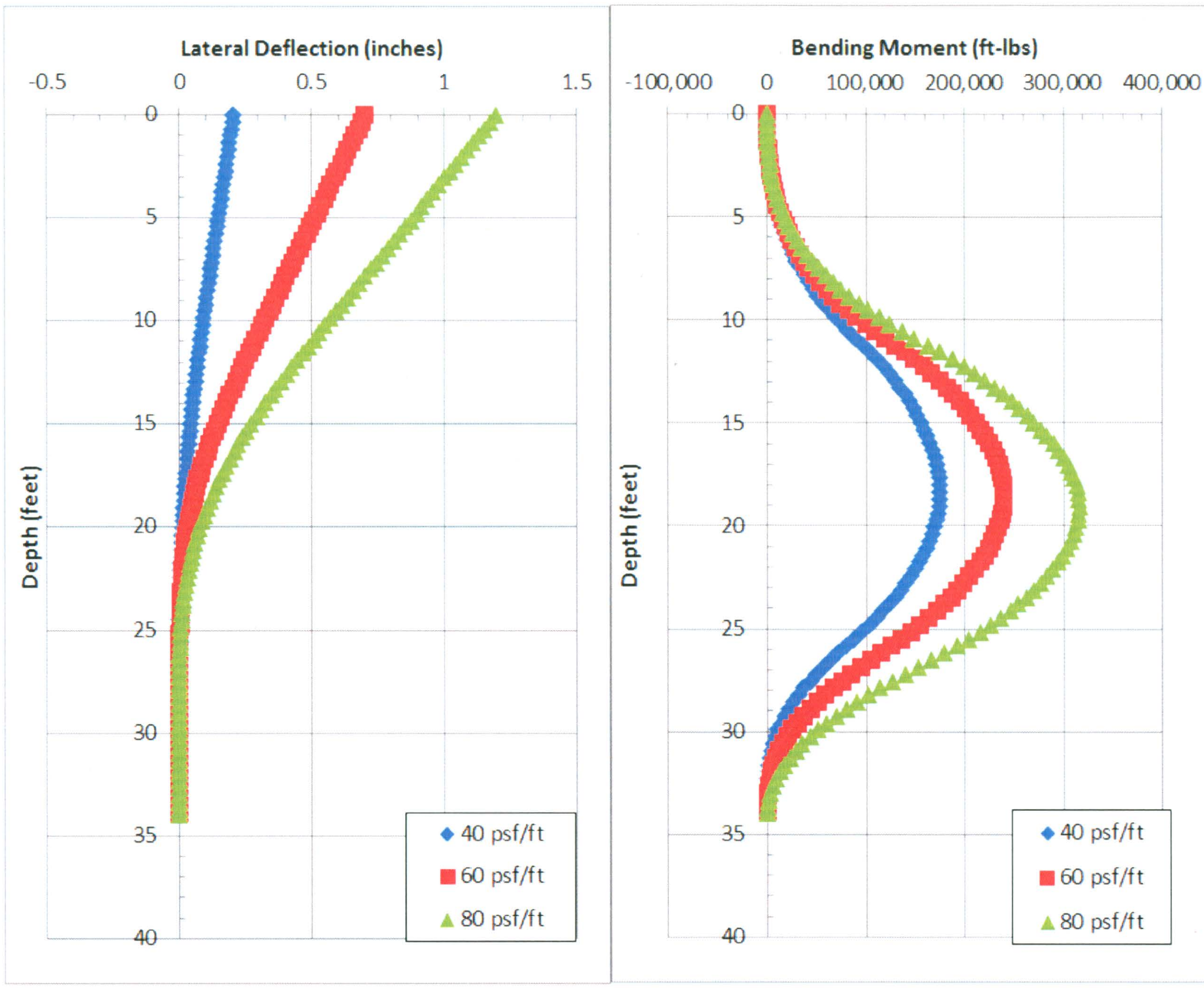

Figure 7.12: Results of L-Pile analysis for Greens Rd @ IH 45; shaft length = 25 feet, wall height = 11 feet

8. CONCLUSIONS

The walls we have assessed in this study are generally representative of typical drilled shaft walls in Texas. The three walls have been in service for 14, 9, and 2 years, respectively, and have cantilevered heights ranging from 5 to 23 feet. A field inspection of each wall revealed no obvious signs of significant distress. Based on L-Pile analyses of these walls, earth pressures greater than a linear increase of 80 psf/ft would likely be required to produce significant distress that could be readily observed.

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