



COASTAL TEXAS PROTECTION AND RESTORATION FEASIBILITY STUDY ENGINEERING

GLO CONTRACT No. 18-127-044



DESIGN CRITERIA

DRAFT SUBMITTAL

NOVEMBER 9, 2018

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FEASIBILITY STUDY ENGINEERING
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PREPARED FOR GLO

PREPARED BY TETRA TECH, INC.



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1 General

1.1 General Description

This Design Criteria Document includes features associated with Alternative A, as described in the Coastal Texas Protection and Restoration Feasibility Study, DIFR-EIS.

1.2 Clear Creek

The Clear Creek facility includes a sector gate located on the existing navigation channel, a pump station, floodwalls, and T-walls. The pump station capacity will be determined from an interior drainage analysis by Mott MacDonald. The facilities will be located west of the Route 146 expansion right-of-way.

There are existing environmental gates on the second channel. The gate structure does not meet the HSDRRS requirements and will be removed.

1.3 Dickinson Bayou

The Dickinson Bayou facility includes a sector gate located on the existing navigation channel, a pump station, floodwalls, and T-walls. The pump station capacity will be determined from an interior drainage analysis. The facilities will be located west of the Route 146 expansion right-of-way.

1.4 Galveston Island

The Galveston Island facilities include four (4) pump stations to support interior drainage. Three pump stations will connect to the new interior drainage channels. One pump station will pump drainage from Offatts Bayou to Galveston Bay.

1.5 Care of Navigation

During construction, the Contractor shall be required to maintain navigation channels open and operable for passage of vessels appropriate for the specific location. Short periods of closure will be allowed with proper notification and coordination with the Coast Guard and marine interests.

1.6 Flood Control Criteria

The gates and floodwall concepts will be developed based on the requirements of Hurricane and Storm Damage Risk Reduction Design Standards (HSDRRDS) (June 2012 version); ETL 1110- 2-584 *Design of Hydraulic Steel Structures* (June 2014); EM 1110-2-1614 *Design of Coastal Revetments, Seawalls, and Bulkheads* (December 1995); and EM 1110-2-2502 *Retaining and Flood Walls* (September 1989). It is assumed that the gates will be designed to provide two-way traffic capability.

Pump station capacities will be developed based on the results of the interior drainage analysis. Pump station concepts will be developed based on requirements of EM 1110-2-3102 *General Principles of Pumping Station Design and Layout* (February 2015), EM 1110-2-3104 *Structural and Architectural Design of Pumping Stations* (June 1989), and EM 1110-2-3105 *Mechanical and Electrical Design of Pumping Stations* (November 1999).

1.7 Relative Sea Level Rise

Relative Sea Level Rise (RSLR) is defined as the sum of eustatic sea level change and the land sinking (subsidence). RSLR used for design of the features is 2.7 feet in the year 2085.

Year	RSLR (feet)
2017	0
2035	0.2
2085	2.72 ⁵

Note. Relative sea level rise (RSLR) has been incorporated into water elevations contained in these criteria.

1.8 Flood Risk Reduction Criteria

1. Basic design requirements are adopted from HSDRRS is for 100 year storm risk reduction. Still water level (SWL) elevations are calculated at 1 percent exceedance 90 percent confidence for 2085, while waves loads are based on mean sea levels at 90 percent confidence.
2. The resilience requirements [are](#) determined from 500-year storm risk reduction based on 0.2 percent exceedance with 90 percent confidence, waves at 90 percent confidence based on mean water levels. Wave loads can be approximated as 1.2 times the 100-year wave loads in lieu of calculating 500-year wave loads per HSDRRS UPDATED 20 MAR 12.
3. Maximum overtopping rates will inform the height of the CSRM features; the CSRM feature elevations are to be approved by USACE and are provisionally estimated as Elevation +17.0 at Clear Creek and +18.0 at Dickinson Bayou. It is assumed that the final elevations of the structures will be approved by the USACE.

1.9 Interior Drainage

Pump station capacity requirements were based on the USACE approved design rainfall events and/or HSDRRS overtopping rate requirements.

1.10 Datum

The horizontal datum is NAD83.

The vertical datum is NAVD88 with elevations expressed in U.S. Survey feet.

NOTE: Google Earth™ vertical datum is WGS84 EGM96 Geoid.

1.11 Design Elevations

Bathymetry used is published NOAA data; original datum MLLW.

1.12 References

1.12.1 Design Standards

American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)

American Concrete Institute, Code Requirements for Environmental Engineering Concrete Structures (ACI 350)

American National Standards Institute (ANSI), Rotodynamic Pumps for Pump Piping (ANSI/HI 9.6.6-2016)

ASTM International, Standard Specification for Structural Steel for Bridges (ASTM A709-17)

1.12.2 USACE Publications

Hurricane and Storm Damage Risk Reduction System Design Guidelines (HSDRRS-DG), June 2012 –USACE New Orleans District

EM 1110-2-1614, Design of Coastal Revetments, Seawalls, and Bulkheads, USACE, December 1995

EM 1110-2-2105, Design of Hydraulic Steel Structures, USACE, March 1993

EM 1110-2-2502, Retaining and Flood Walls, USACE, September 1989

EM 1110-2-2906, Design of Pile Foundations, USACE, January 1991

EM 1110-2-3102, General Principles of Pumping Station Design and Layout, USACE, February 2015

EM 1110-2-3104, Structural and Architectural Design of Pumping Stations, USACE, June 1989

EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations, USACE, November 1999

ETL 1110-2-307, Flotation Stability Criteria for Concrete Hydraulic Structures, USACE, , August 1987

ETL 1110-2-584, Design of Hydraulic Steel Structures, USACE, June 2014

1.12.3 Other

Coastal Texas Protection and Restoration Feasibility Study, Draft Integrated Feasibility Report and Environmental Impact Statement (DIFR-EIS), USACE Galveston District and Texas General Land Office, October 2018.

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2 Hydraulics

2.1 General

The Galveston Bay fetch is large enough to have surges within the bay. The surge data provided varies between the sites. Also, there are significant differences in the results of the drainage analyses at the various locations.

2.2 Clear Creek Hydraulics

Hydrostatic loads from the 2085 surge events are used in the design and resiliency load cases. The 0.2 % still water level is used to establish the top of floodwall elevation, top of floodgate elevation and the pump station minimum operating floor elevation.

Rainfall events were used in the interior drainage analyses and were the basis of the required pumping capacity. This analysis also confirmed the required minimum (open) gate width for outflow during the design rainfall event. The 100-year +30% rainfall event was also the assumed case for Hurricane Reverse Head, 1% AEP, 90% non-exceedance, year 2085.

Table 2-1. Clear Creek Stage Elevation

Stage	Elevation (Feet NAVD88), Flood Side / Land Side
Surge Events	
Still water level, 1% AEP, 90% non-exceedance, year 2085	13.47 ¹ (13.5 ²) / -1.0 ⁴
Still water level, 0.2% AEP, year 2085 ^{1,2}	16.90 ¹ (16.9 ²) / -1.0 ⁴
Reverse Head (Assumed)	7.4 / 0.0
Rainfall Events	
25-year rainfall event - Pump Station Operating at – 45,661 cfs ^{2,3}	-1.0 to +1.0
100-year rainfall event - Pump Station Operating at 45,661 cfs ^{2,3} [Assumed case for Hurricane Reverse Head, 1% AEP, 90% non-exceedance, year 2085 ²]	7.4 / 0.0
500-year rainfall event - Pump Station Operating at – 45,661 cfs ^{2,3}	-1.0 / 12.2

NOTES:

1. Result developed on 26 July 2018
 2. Result developed on 17 August 2018.
 3. Result developed on 21 August 2018
 4. Assumed at 25-year rainfall event with pumping
- Flood Side = Galveston Bay Side, Downstream; Land Side = Interior Side, Upstream

Table 2-2. Clear Creek: Gates Open, Pumps On

Sector Gate Open, Pumps On ¹ , 45,661 cfs ²				
Rainfall Event	Land Side (Interior) WSE [feet NAVD88]	Flood Side (Downstream) WSE [feet NAVD88]	Velocity Through Sector Gate Monolith fps	Velocity Downstream of Sector Gate Monolith fps
10-year (+30%)	-0.1	-0.1		
25-year (+30%)	1.0	1.0		
50-year (+30%)	2.5P	2.4		
100-year (+30%)	4.2	4.0	4.63 ³	0.92 ³
500-year (+30%) ⁴	7.0	5.6		

NOTES:

1. Result developed on 17 August 2018
2. 45,661 = 44,500 + 1,161 (peak overtopping); result developed on 21 August 2018
3. Result developed on 22 August 2018
4. (+30%) is stated rainfall increase used for interior drainage analyses

Table 2-3. Clear Creek: Gates Open No Pumps

Sector Gate Open, No Pumps ¹				
Rainfall Event	Land Side (Interior) WSE [feet NAVD88]	Flood Side (Downstream) WSE [feet NAVD88]	Velocity Through Sector Gate Monolith fps	Velocity Downstream of Sector Gate Monolith fps
10-year (+30%)	5.3	4.8		
25-year (+30%)	6.4	5.4		
50-year (+30%)	7.6	5.9		
100-year (+30%)	8.7	6.1	13.22 ²	2.24 ²
500-year (+30%) ³	11.7	6.7		

NOTES:

1. Result developed on 17 August 2018
2. Result developed on 22 August 2018
3. (+30%) is stated rainfall increase used for interior drainage analyses

2.3 Dickinson Bayou

Hydrostatic loads from the 2085 surge events are used in the design and resiliency load cases. The 0.2 % still water level is used to establish the top of floodwall elevation, top of floodgate elevation and the pump station minimum operating floor elevation.

Rainfall events were used in the interior drainage analyses and were the basis of the required pumping capacity. This analysis also confirmed the required minimum (open) gate width for outflow during the design rainfall event. The 100-year +30% rainfall event was also the assumed case for Hurricane Reverse Head, 1% AEP, 90% non-exceedance, year 2085.

Table 2-4. Dickinson Bayou Stage Elevations

Stage	Elevation (Feet NAVD88), Flood Side / Land Side
Surge Events	
Still water level, 1% AEP, 90% non-exceedance, year 2085	12.77 ¹ (12.8 ²) / -1.0
Still water level, 0.2% AEP, year 2085	17.39 ¹ , (17.42) / -1.0
Reverse Head (Assumed)	-1.0 / 12.0
Rainfall Events	
25-year rainfall event - Pump Station Operating at – 19,125 cfs ^{2,3}	-1.0 to +1.0
100-year rainfall event - Pump Station Operating at 19,125 cfs ^{2,3} [Assumed case for Hurricane Reverse Head, 1% AEP, 90% non-exceedance, year 2085 ^{1,2}]	-1.0 / 12.0
500-year rainfall event - Pump Station Operating at – 19,125 cfs ^{2,3}	-1.0 / 18.4

Flood Side = Galveston Bay Side, Downstream; Land Side = Interior Side, Upstream

NOTES:

1. Result developed on 26 July 2018
2. Result developed on 15 August 2018. Assume 100-year rainfall event for land (interior) side, EL -1.0 for flood side
3. 19,125 = 18,700 + 425 (peak overtopping); result developed on 21 August 2018

Table 2-5. Dickinson Bayou: Gates Open, Pumps On

Sector Gate Open, Pumps On ¹ , 19,125 cfs ²				
Rainfall Event	Upstream (Interior) WSE [feet NAVD88]	Downstream (Exterior) WSE [feet NAVD88]	Velocity Through Sector Gate Monolith fps	Velocity Downstream of Sector Gate Monolith fps
10-year (+30%)	0.8	1.2		
25-year (+30%)	0.7	0.7		
50-year (+30%)	1.6	1.4		
100-year (+30%)	2.4	1.7	8.0 ³	1.42 ³
500-year (+30%) ⁴	8.0	2.4		

1. Results developed on 15 August 2018
2. 19,125 = 18,700 + 425 (peak overtopping); received 21 August 2018
3. Results developed on 22 August 2018
4. (+30%) is stated rainfall increase used for interior drainage analyses

Table 2-6. Dickinson Bayou: Gates Open No Pumps

Sector Gate Open, No Pumps ¹				
Rainfall Event	Upstream (Interior) WSE [feet NAVD88]	Downstream (Exterior) WSE [feet NAVD88]	Velocity Through Sector Gate Monolith fps	Velocity Downstream of Sector Gate Monolith fps
10-year (+30%)	3.7	1.3		
25-year (+30%)	6.1	1.6		
50-year (+30%)	8.0	1.9		
100-year (+30%)	9.7	2.1	13.0 ²	3.65 ²
500-year (+30%)	14.3	2.9		

1. Results developed on 15 August 2018
2. Results developed on 22 August 2018
3. (+30%) is stated rainfall increase used for interior drainage analyses

2.4 Galveston Island, Offatts Bayou and Pump Stations 1

Rainfall events were used in the interior drainage analyses and were the basis of the required pumping capacity.

Table 2-7. Galveston Island - Offatts Bayou - Pump Station No. 1 Stage Elevation

Stage	Elevation (Feet NAVD88), Flood Side / Land Side
Surge Events	
Still water level, 1% AEP, 90% non-exceedance, year 2085	11.9 ² / TBD
Still water level, 0.2% AEP, 90% non-exceedance, year 2085	14.7 ² / TBD
Rainfall Events	
Pump Station Operating at 25-year storm – 250 cfs	9.8 ² / -1.09 to +2.5 ¹

1. Results developed on 17 August 2018
2. Results developed on 15 October 2018

Flood Side = Galveston Bay Side, Land Side = Offatts Bayou Side

2.5 Galveston Island and Pump Station Nos. 2, 3, and 4

Hydrostatic loads from the 2085 surge events are used in the design and resiliency load cases. The 0.2 % still water level is used to establish the top of floodwall and minimum operating floor elevations of the pump stations.

Rainfall events were used in the interior drainage analyses and were the basis of the required pumping capacity and pump station intake water surface elevations.

Table 2-8. Galveston Island and Pump Station No. 2 Stage Elevation

Stage	Elevation (Feet NAVD88), Flood Side / Land Side
Surge Events	
Still water level, 1% AEP, 90% non-exceedance, year 2085	10.2 ² / TBD
Still water level, 0.2% AEP, 90% non-exceedance, year 2085	15.5 ² / TBD
Rainfall Events	
Pump Station Operating at 25-year +30% storm – 1500 cfs	8.0 ² /-10 to -6 ¹
<ol style="list-style-type: none"> 1. Results developed on 17 August 2018 2. Results developed on 15 October 2018 	

Flood Side = Galveston Bay Side

Table 2-9. Galveston Island and Pump Station No. 3 Stage Elevation

Stage	Elevation (Feet NAVD88), Flood Side / Land Side
Surge Events	
Still water level, 1% AEP, 90% non-exceedance, year 2085	14.6 ² / TBD
Still water level, 0.2% AEP, 90% non-exceedance, year 2085	18.2 ² / TBD
Rainfall Events	
Pump Station Operating at 25-year +30% storm – 4500 cfs	8.0 ² /-10 to -6.5 ¹
<ol style="list-style-type: none"> 1. Results developed on 17 August 2018 2. Results developed on 15 October 2018 	

Flood Side = Galveston Bay Side

Table 2-10. Galveston Island and Pump Station No. 4 Stage Elevation

Stage	Elevation (Feet NAVD88), Flood Side / Land Side
Surge Events	
Still water level, 1% AEP, 90% non-exceedance, year 2085	14.6 ² / TBD
Still water level, 0.2% AEP, 90% non-exceedance, year 2085	18.2 ² / TBD
Rainfall Events	
Pump Station Operating at 25-year +30% storm – 1500 cfs	8.5 ² /-10 to -0.5 ¹
<ol style="list-style-type: none"> 1. Results developed on 17 August 2018 2. Results developed on 15 October 2018 	

Flood Side = Galveston Bay Side

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3 Loads

3.1 Hydrostatic Loads

Hydrostatic loads refer to the vertical and horizontal loads induced by static water pressures, excluding uplift pressures. These loads include the weight of the water within the pump bay depending on the stages on the land (protected) side. Hydrostatic pressures may be applied from both the flood and land (protected) sides.

3.2 Uplift Loads

Uplift loads for which structures will be designed refer to the uplift conditions listed below:

1. Uplift Condition A (Impervious). This assumes the sheet pile cutoff wall is fully effective. The uplift pressure is constant across the base and is equal to the protected side pressure head of the cutoff.
2. Uplift Condition B (Pervious). This assumes the sheet pile cutoff wall is ineffective. The uplift pressure is assumed to vary linearly across the base between the flood side pressure and the protected side pressure head.

3.3 Wave Loads

The wave loads that follow are exclusive of hydrostatic loads.

Table 3-1. Storm Elevations

USACE Storm Number	529	453
Return Period (Years)	100	500

Elevation (Ft NAVD88)	F_{wave} [kip/ft]	F_{wave} [kip/ft]
17	0.3793	0.4821
16	0.4102	0.4810
15	0.4411	0.4764
14	0.4719	0.4718
13	0.485	0.4671
12	0.4804	0.4625
11	0.4757	0.4579
10	0.471	0.4533
9	0.4663	0.4487
8	0.4617	0.4441
7	0.457	0.4395
6	0.4523	0.4349
5	0.4476	0.4302
4	0.443	0.4256
3	0.4383	0.4210
2	0.4336	0.4164
1	0.429	0.4118
0	0.4243	0.4072
-1	0.4196	0.4026

Elevation (Ft NAVD88)	F _{wave} [kip/ft]	F _{wave} [kip/ft]
-2	0.4149	0.3980
-3	0.4103	0.3933
-4	0.4056	0.3887
-5	0.4009	0.3841
-6	0.3962	0.3795
-7	0.3916	0.3749
-8	0.3869	0.3703
-9	0.3822	0.3657
-10	0.3776	0.3611
-11	0.3729	0.3564
-12	0.3682	0.3518
-13	0.3635	0.3472
-14	0.3589	0.3426
-15	0.3542	0.3380
-16	0.3495	0.3334
-17	0.3448	0.3288
-18	0.3402	0.3242
-19	0.3355	0.3195
-20	0.3308	0.3149
-21	0.3261	0.3103
-22	0.3215	0.3057
-23	0.3168	0.3011
-24	0.3121	0.2965
-25	0.3075	0.2919
-26	0.3028	0.2873
-27	0.2981	0.2826
-28	0.2934	0.2780

In lieu of the wave load in the table above, the HSDRRS-DG states that the wave load for 0.2% AEP can be assumed as 1.2 * 1% AEP (100-year) wave load.

3.3.1 Wave Slamming

Wave slamming loads are not considered at this time.

3.3.2 Wave Overtopping

Wave overtopping loads are not considered at this time.

3.4 Dead Loads

The dead loads include the weight of the structure, water weight, mechanical equipment, and other components of the project.

Table 3-2. Material Unit Weights

Material	Unit Weight (lb/ft³)
Fresh Water	62.4
Salt Water	64.0
Normal Weight Concrete	150.0
Steel	490.0
Granular fill (saturated)	120.0

3.4.1 Pump Station Dead Loads

The pump station dead loads include weights of a fixed objects from the following structural components:

Superstructure - The superstructure includes the items above the operating floor level. The dead-load-generating components within this part of the pump station are the roof system, columns, crane beam supports, precast support beams, and day tank (including support).

Substructure - The loads from the substructure include dead load from the concrete slab and walls.

Foundation - The dead load for the foundation includes the 12-inch-thick contractor designed mud slab underneath the base slab. The foundation dead load incorporates the load from this mud slab.

3.5 Live Loads

Live loads not specified by HSDRRS-DG, USACE Engineering Manuals (EM) or Engineering Technical Letters (ETL) shall be in accordance with ASCE 7.

3.5.1 Construction Live Loads

Per the HSDRRS-DG, structures shall be designed for 200 psf equipment load with no uplift.

3.6 Soil Loads

Hydrostatic loads for which structure will be designed refer to the vertical and horizontal loads induced by static water pressures, excluding uplift pressures.

3.7 Silt Loads

Not considered at this time.

3.8 Marine Growth

Not considered at this time.

3.9 Wind Loads

The wind force used for design is based on the requirements of ASCE-7 or 50 psf (from HSDRRS-DG), whichever is greater. The wind force is applied to exposed portions of the structure.

For ASCE-7 wind calculations, a basic wind speed of 160 mph (Risk Category II-IV) was used along with an importance factor of 1.15 and an Exposure Category C. HSDRRS-DG is based on Allowable Stress Design (ASD). Therefore, using LRFD as provided in ASCE 7, the comparable HSDRRS-DG based factored wind load for strength design would equal 83.5 psf (equals 1.67×50 psf).

3.10 Debris Impact Loads

The gates are located in areas similar to the HSDRRS-DG locations outside of barge/boat impact zones. For the Unusual Load Case and in accordance with HSDRRS-DG, a minimum debris impact loading of 0.5 kips/foot shall be applied at the top of the wall, but not to exceed the (500 year) SWL.

3.11 Basic Load Cases

The basic load cases for design and for resiliency are described in HSDRRS-DG (USACE, 2012) Table 5.1.

Table 3-3. Basic Load Cases for Storm Protection Features (HSDRRS-DG Table 5.1)

Load Case [1]	Description	Overstress Allowed		Pile Load - Factors of Safety (for Q-Case)						
		Structure & Gates	Foundation Piles	Static Load Test		PDA Load Test		No Load Test		
				C	T	C	T	C	T	
I [2]	Construction	$\gamma H_f (DL + LLc)$	16.7%	16.7%	1.70	1.70	2.15	2.60	2.60	2.60
II [2]	Construction + Wind	$\gamma H_f (DL + W + LLc)$	33.3%	33.3%	1.50	1.50	1.90	2.25	2.25	2.25
III a,b	Still Water Level (SWL)	$\gamma H_f (DL + H_{SWL} + UBL)$	0	0	2.00	2.00	2.50	3.00	3.00	3.00
IV a,b	SWL + Wind	$\gamma H_f (DL + H_{SWL} + UBL + W)$	33.3%	33.3%	1.50	1.50	1.90	2.25	2.25	2.25
V a,b	SWL + Wave	$\gamma H_f (DL + H_{SWL} + UBL + W_v)$	33.3%	33.3%	1.50	1.50	1.90	2.25	2.25	2.25
VI a,b [3]	SWL + Wind + (Unusual) Vessel or Debris Impact	$\gamma H_f (DL + H_{SWL} + UBL + BI + W)$	50%	33.3%	1.50	1.50	1.90	2.25	2.25	2.25
VII a,b	Reverse Head (RH)	$\gamma H_f (DL + H_{RH} + UBL)$	0	0	2.00	2.00	2.50	3.00	3.00	3.00
VIII a,b	RH + Wind	$\gamma H_f (DL + H_{RH} + UBL + W)$	33.3%	33.3%	1.50	1.50	2.25	2.25	2.25	2.25
IX a,b	RH + Wave	$\gamma H_f (DL + H_{RH} + UBL + W_v)$	33.3%	33.3%	1.50	1.50	2.25	2.25	2.25	2.25
X a,b	RH + Wind + (Unusual) Vessel or Debris Impact	$\gamma H_f (DL + H_{RH} + UBL + W + BI)$	50%	33.3%	1.50	1.50	2.25	2.25	2.25	2.25
XI	RH + Wind + Wave FLOODGATE ONLY	$\gamma H_f (DL + H_{RH} + UBL + W + W_v)$	33.3%	33.3%	1.50	1.50	2.25	2.25	2.25	2.25

a = impervious load distribution, b = pervious load distribution

[1] Ref. HSDRRS-DG Table 5.2 (b) and Table 5.1 for a, impervious and b, pervious

[2] LLc, Construction load = 200 psf equipment soil surcharge

[3] BI, Barge Impact unusual vessel = 225 K; combined with 140 MPH Wind Load (Zone 1B). Debris Impact only, within protection zones behind impact barriers = 0.5 K/ft

Table 3-4 Design Resiliency Checks (HSDRRS-DG Table 5.2)

Load Case	Description		Overstress Allowed		Pile Load - Factors of Safety (for Q-Case)					
			Structure & Gates	Foundation Piles [1]	Static Load Test		PDA Load Test		No Load Test	
					C	T	C	T	C	T
DRC I [2] a,b	Water to TOW, W/O Unbalanced Load	$\gamma H_f (DL + H_{TW})$	33.3%	33.3%	1.50	1.50	1.90	2.25	2.25	2.25
DRC II [2] a,b	Water To TOW, W/ Unbalanced Load	$\gamma H_f (DL + H_{TW} + UBL)$	50%	50%	1.33	1.33	1.67	2.00	2.00	2.00
DRC III a,b	SWL (500-yr) + WAVE	$\gamma H_f (DL + H_{SWL} + W_v + UBL)$	50%	50%	1.15	1.15	1.40	1.40	1.70	1.70
DRC IV a,b	SWL (500-yr) + WAVE W/ Unbalanced Load	$\gamma H_f (DL + H_{SWL} + UBL + W_v)$	67%	67%	1.15	1.15	1.40	1.40	1.70	1.70
DRC V Case I	(Zone 1A) 400 K Barge IMPACT + SWL(100-YR) + 160 MPH WIND	$\gamma H_f (DL + H_{SWL} + BI + W)$	**	**	1.15	1.15	1.40	1.40	1.70	1.70
DRC VI Case I	(Zone 1B) 450 K Barge IMPACT + SWL(500-YR) + 160 MPH WIND	$\gamma H_f (DL + H_{500SWL} + BI + W)$	**	**	1.15	1.15	1.40	1.40	1.70	1.70
DRC VII Case II	(Zone 1A) 200 K Barge Impact + SWL (100-YR) + WAVE(100 YR)	$\gamma H_f (DL + H_{SWL} + UBL + BI + W_v)$	**	**	1.15	1.15	1.40	1.40	1.70	1.70
DRC VII Case II	(Zone 1B) 225 K Barge IMPACT + SWL(500-YR) + 1.2*WAVE(100 YR)	$\gamma H_f (DL + H_{SWL} + UBL + BI + W_v)$	**	**	1.15	1.15	1.40	1.40	1.70	1.70
DRC I [2] a,b	Water to TOW, W/O Unbalanced Load	$\gamma H_f (DL + H_{TW})$	33.3%	33.3%	1.50	1.50	1.90	2.25	2.25	2.25
DRC II [2] a,b	Water To TOW, W/ Unbalanced Load	$\gamma H_f (DL + H_{TW} + UBL)$	50%	50%	1.33	1.33	1.67	2.00	2.00	2.00
DRC III a,b	SWL (500-yr) + WAVE	$\gamma H_f (DL + H_{SWL} + W_v + UBL)$	50%	50%	1.15	1.15	1.40	1.40	1.70	1.70

** Both concrete and steel designs shall utilize the LRFD methods of analyses. The strength reduction factor Φ shall comply with ACI and AISC codes. The hydraulic factor (Hf) shall equal 1.0. The applicable load factor combination is:

[1] Actual “unfactored” service load shall be used in any pile analysis programs

[2] TOW is considered the lower of 500-year SWL or TOW per HSDRRS-DG, pg. 5-45, Note 2, NOTES ON STILL WATER LEVELS (SWL)

Compute Wind Load, $F = .00256 (V^2)(I)(A)$, $I = 1.15$, $A =$ gross exposed area.

4 Geotechnical

Draft pile capacity curves were developed on 9 August 2018. Pile capacity curves and pile lateral capacities are included in Appendix B.

Factors of safety (FOS) for axial pile capacity are presented in Table 3.7 of the HSDRRS-DG. Design factors of safety for unusual and extreme load cases are presented in Table 5.2 (a) and (b) of the HSDRRS-DG. It is assumed that pile load testing program will be performed and that the FOS for a static load test will be used for design and to select pile tip elevations.

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5 Floodwalls and T-walls

Floodwalls and T-walls are included at both Clear Creek and Dickinson Bayou.

Floodwalls are similar to the Inner Harbor Navigation Canal (IHNC) floodwall, consisting primarily of vertical spun-cast piles, steel pipe batter piles, and a concrete cap. Floodwalls will be constructed in-the-wet.

T-walls will be composed of cast-in-place reinforced concrete with a base slab and a stem. The base slab will be founded on a pile foundation. T-walls will be constructed in the dry. A cutoff wall will be provided below the base slab to limit seepage. The sheet pile tip shall extend a minimum of 10-feet below the bottom of the base slab in accordance with HSDRRS-DG.

HSDRRS-DG, Chapter 3, requires seepage, global stability, heave and other pertinent geotechnical analyses are performed to ensure that the overall stability of the wall system is designed to meet USACE criteria.

Transitions and levee tie-ins shall be in accordance with Chapters 5 and 12 of HSDRRS-DG. The transitions include L-wall or T-wall to T-wall, L-wall or T-wall to I-wall, and L-wall or T-wall to uncapped sheet piling. Tie-in details are also provided for T-walls, L-walls and I-walls that terminate into a levee section.

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6 Gates

6.1 Sector Gates

Floodgates will be provided at Clear Creek and Dickinson Bayou.

Table 6-1. Clear Creek and Dickinson Sector Gates

Location	Structures	Width	Sill Elevation	Top Elevation	Guidewall Type
Clear Creek	Sector Gates	75	-12.0	17.0	Timber
Dickinson Bayou	Sector Gates	100*	-9.0	18.0	Timber

*The width of the sector gate at Dickinson Bayou was increased from the initial 60-foot specified width to 100-feet wide to increase the outflow for the interior drainage design.

6.2 Cofferdams

The sector gate monoliths will be constructed in the dry using an internally braced steel sheet pile cofferdam. To maintain a navigable channel, the recess sides of each monolith will each have a stand-alone cofferdam. The sill between them, which includes the gate seal plate, is assumed constructed within a separate cofferdam cell; this will be cut to the top of slab level to re-establish the navigation channel and allow construction of the cofferdam for the second recess monolith.

6.3 Sector Gate Monolith Concrete

The monolith base slab will be pile supported.

The recess walls of the sector gate monoliths are designed as a cantilever wall extending from the base slab.

The thrust block and machinery block are based on historical data for similar equipment.

6.4 Monolith Pile Criteria

6.4.1 Sector Gate Monolith Pile Foundations

Pile curves were provided for both Clear Creek and Dickinson based on limited existing geotechnical information. The design factors of safety meet the requirements of EM 1110-2-2906 and the HSDRRS-DG. Pile capacities are assumed to have pile load testing. All piles will be furnished with tension connections.

6.4.2 Cut-off Wall

A cut-off sheet pile wall will be provided to reduce possible seepage, uplift and scour. The cutoff wall is assumed to extend 40-feet below the bottom of the sector gate tremie slab.

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7 Pump Station

7.1 General

Assume 15 ft/s is the maximum discharge velocity (ANSI, 2016). Minimum Intake elevations and maximum discharge elevations vary as follows:

7.2 Materials

7.2.1 Structural Material Weights

Table 7-1. Material Unit Weights

Material	Unit Weight (lb/ft ³)
Fresh Water	52.4
Salt Water	64.0
Normal Weight Concrete	150.0
Steel	490.0
Granular Fill (Saturated)	120.0

7.2.2 Concrete

7.2.2.1 Cast-in-Place Concrete

The Pump stations and Safe House structures will utilize conventional "cast-in-place" construction techniques.

Concrete Compressive Strength $f'c = 4,000$ psi.

7.2.2.2 Precast Concrete

The Pump station siding will precast concrete panels similar to West Closure Complex (WCC).

Concrete Compressive Strength $f'c = 54,000$ psi.

7.2.3 Steel

7.2.3.1 Reinforcing Steel

The reinforcing steel is Grade 60. The minimum reinforcing ratios of both HSDRRS-DG and ACI 350 apply.

7.2.3.2 Structural Steel

Structural steel is ASTM A709 Steel.

7.3 Safe House

The safe house is designed as an independent structure to meet wind loads determined by HSDRRS-DG. The safe house is assumed similar to the control building/safe house at West Closure Complex (WCC), with an approximate footprint of 20-feet wide by 50-feet long. Safe houses are assumed at Clear Creek, Dickinson Bayou, and Galveston Island Pump Stations 2, 3 and 4. At Galveston Island Pump Station 1, it is assumed that a common safe house will be provided for the nearby gate at Offatts Bayou.

7.4 Clear Creek Pump Station

Design Discharge = 45,661 cfs.

The width of the pump station entrance is based on the maximum flow a maximum velocity of 2.5 ft/s across the trash rack.

7.4.1 Structures

The Clear Creek Pump Station foundation, substructure and superstructure are assumed similar to WCC with the following exceptions:

1. Clear Lake is shallower than the pump station invert elevation. The foundation will include a sloping entrance.
2. WCC is located on the Gulf Intercoastal Waterway (GIWW). The discharge from the pump station is directly into the GIWW. Highway 146 bridge piers are downstream of the pump station. To reduce the potential for scour, the pump discharge will be piped and discharged into Galveston Bay at a location east of the highway.

7.4.2 Mechanical

Mechanical features are assumed similar to WCC, except that the pump discharge will be modified as described above.

Assume 15 ft/s is the maximum discharge velocity.

7.5 Dickinson Bayou Pump Station

Design Discharge = 19,125 cfs.

The width of the pump station entrance is based on the maximum flow a maximum velocity of 2.5 ft/s across the trash rack.

7.5.1 Structures

The Dickinson Bayou Pump Station foundation, substructure and superstructure are assumed similar to WCC with the following exceptions:

1. Dickinson Bayou is more shallow than the pump station invert elevation. The foundation will include a sloping entrance.
2. WCC is located on the Gulf Intercoastal Waterway (GIWW). The discharge from the pump station is directly into the GIWW. Highway 146 bridge piers are downstream of the pump station. To reduce the potential for scour, the pump discharge will be piped and discharged into Galveston Bay at a location east of the highway.

7.5.2 Mechanical

Mechanical features are assumed similar to WCC, except that the pump discharge will be modified as described above.

Assume 15 ft/s is the maximum discharge velocity.

7.6 Galveston Island Pump Station No. 1

Design Discharge = 250cfs

The width of the pump station entrance is based on the maximum flow at a maximum velocity of 2.5 ft/s across the trash rack.

The Galveston Island Pump Station No. 1 will pump water from Offatts Bayou into Galveston Bay. foundation, substructure and superstructure are assumed for a vertical pump, discharging directly to Galveston Bay. Assume 15 ft/s is the maximum discharge velocity.

Galveston Island 1 is significantly smaller than the other pump stations, and therefore power to drive an electric motor is more reliable. A diesel generator backup is still required, complete with 5-day reserve.

7.7 Galveston Island Pump Station No. 2

Design Discharge = 1,500cfs

The width of the pump station entrance is based on the maximum flow a maximum velocity of 2.5 ft/s across the trash rack.

The Galveston Island Pump Station No. 2 will pump water from interior drainage channel system into Galveston Bay. At the pump station the channel is 20-feet wide by 10-feet high. A transition structure will connect the drainage channel and the pump station intake. The pump station foundation, substructure and superstructure are assumed for a vertical pump, discharging directly to Galveston Bay. Assume 15 ft/s is the maximum discharge velocity.

7.8 Galveston Island Pump Station No. 3

Design Discharge = 4,500cfs

The width of the pump station entrance is based on the maximum flow a maximum velocity of 2.5 ft/s across the trash rack.

The Galveston Island Pump Station No. 3 will pump water from interior drainage channel system into Galveston Bay. At the pump station the channel is 20-feet wide by 10-feet high. A transition structure will connect the drainage channel and the pump station intake. The pump station foundation, substructure and superstructure are assumed for a vertical pump, discharging directly to Galveston Bay. Assume 15 ft/s is the maximum discharge velocity.

7.9 Galveston Island Pump Station No. 4

Design Discharge = 1,500cfs

The width of the pump station entrance is based on the maximum flow a maximum velocity of 2.5 ft/s across the trash rack.

The Galveston Island Pump Station No. 4 will pump water from interior drainage channel system into Galveston Bay. At the pump station the channel is 20-feet wide by 10-feet high. A transition structure will connect the drainage channel and the pump station intake. The pump station foundation, substructure and superstructure are assumed for a vertical pump, discharging directly to Galveston Bay. Assume 15 ft/s is the maximum discharge velocity. Pump Station No. 4 is similar to Pump Station No. 2.

7.10 Fuel Storage

Five (5) days of fuel storage shall be provided at each site. Provide multiple smaller tanks over fewer larger tanks for both redundancy and ease of maintenance.

7.11 Electrical

Design will be based on USACE Publication EM 1110-2-3105, Mechanical and Electrical Design of Pumping Station, Dated November 1999, and the latest edition of the National Electric Code.

Electrical motors shall be 4160V for motors above 500 hp and 460V for motors 500 hp and below.

Pump stations will be controlled via a PLC-based control system.

CCTV system with HD color cameras and monitors will be used for monitoring purposes.

Permanent emergency generators will be provided as needed.

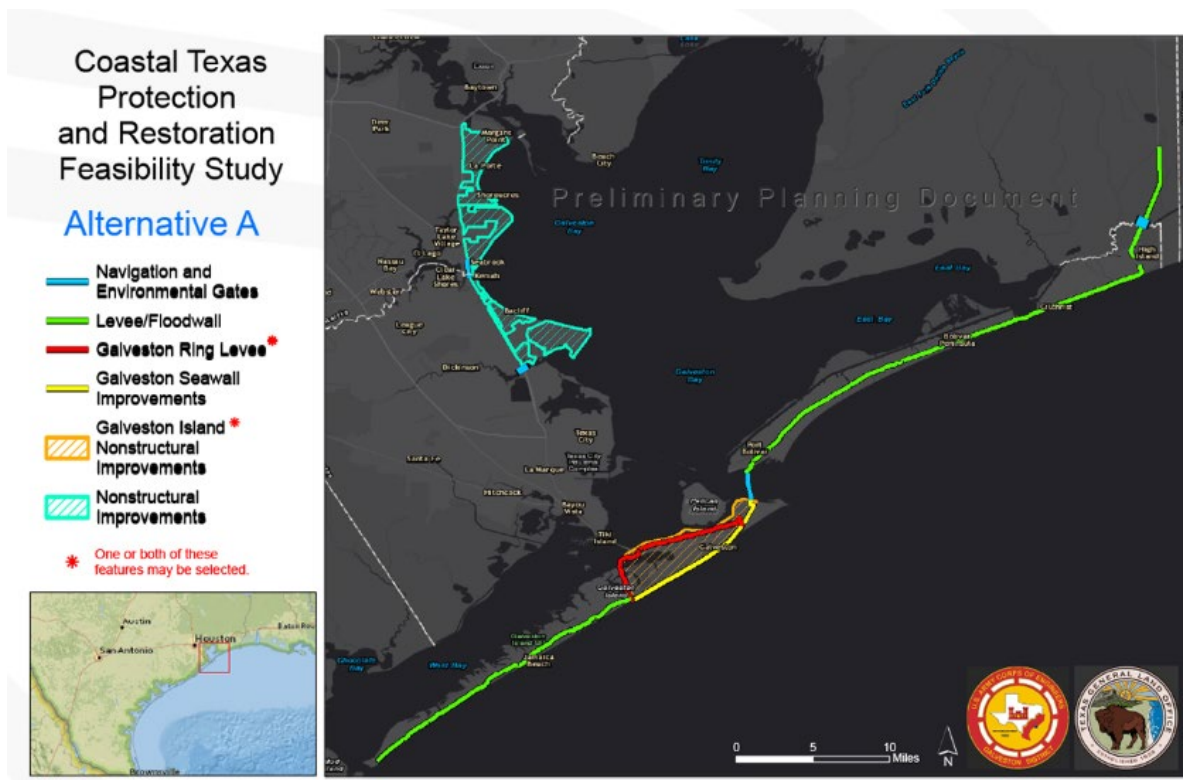
All equipment will be UL approved

All lighting will be LED based.

Appendix A - Alternative A

Alternative A: Coastal Storm Surge Barrier

This alternative was developed to address storm surge flooding at the Gulf interface and also to include the highest number of structures and critical facilities within the project area. This would provide risk reduction to the critical Gulf Intracoastal Water Way (GIWW), by maintaining the existing geomorphic features along Bolivar Peninsula and Galveston Island. A strategy included preventing storm surge from entering the Galveston Bay with a barrier system across Bolivar Peninsula, a closure at the pass at Bolivar Roads, improvements to the Galveston Seawall and a barrier along the west end of Galveston Island. To address wind-driven surges in the bay, which could impact both the back side of Galveston Island and the upper reaches of the bay, nonstructural measures, ring levees and closures on key waterways are also being investigated. Although the Ecosystem Restoration (ER) and CSRSM alternatives will be evaluated for separate benefits, the different Alternatives provide some nexuses between the features. By linking into the beach and dune restoration features along Bolivar Peninsula and Galveston Island, the ER features should also increase the resiliency of the CSRSM feature.

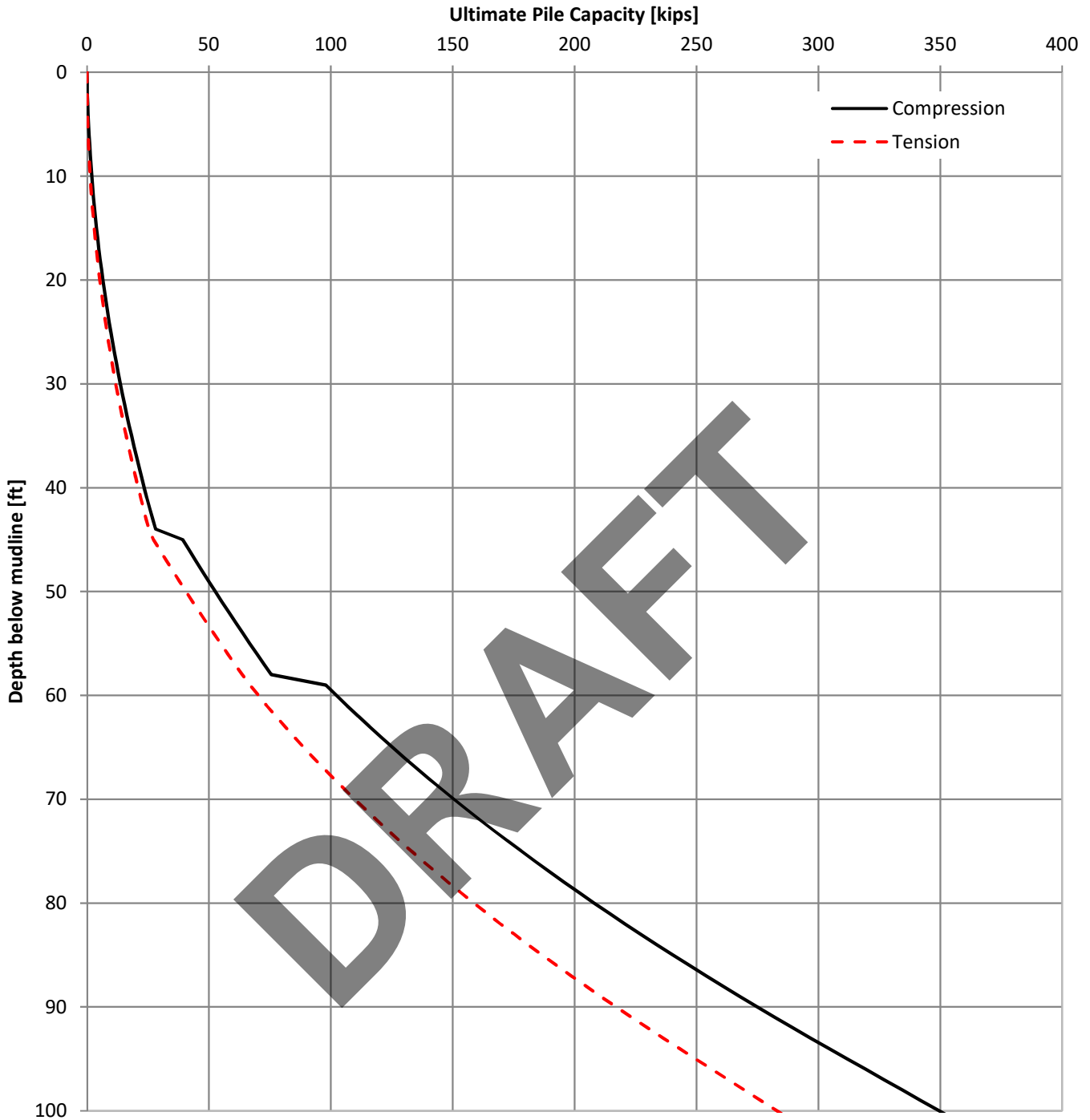


For more information please go to <http://coastalstudy.texas.gov/alternatives/index.html>

Appendix B - Geotechnical Information

B1 – Vertical Pile Capacity

B2 – Draft Lateral Piles Memo



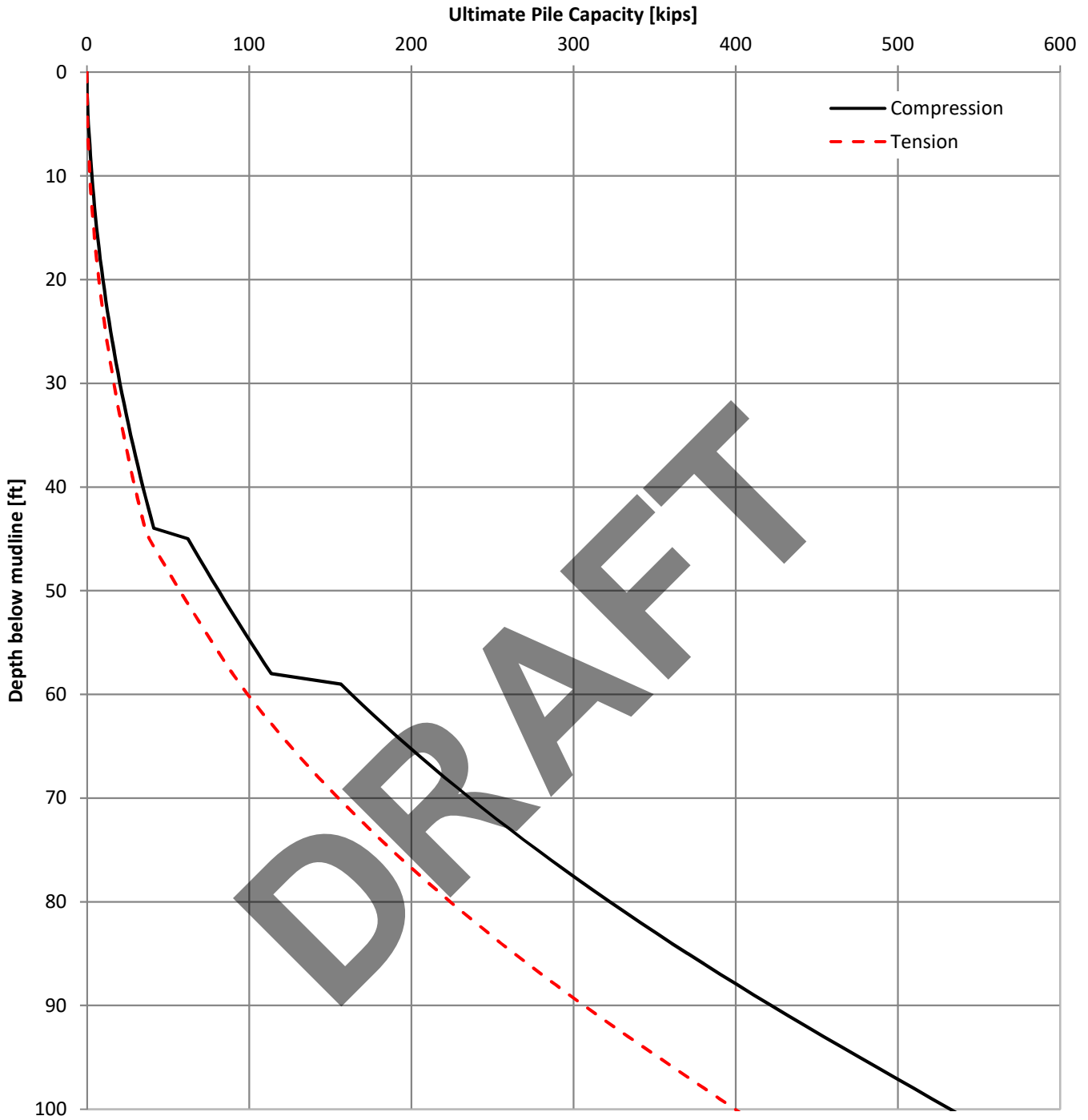
PILE DIAMETER: 12.75-IN

PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
2. PILE CAPACITIES CALCULATED PER API RP 2A.
3. REFERENCE GEOTECH REPORT: MCBRIDE-RATCLIFF AND ASSOCIATES, DECEMBER 6, 1982, AND MARCH 14, 1985.
4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.

	SUBJECT ULTIMATE PILE CAPACITY 18-IN STEEL PIPE PILE, CLEAR CREEK SITE		
	PROJECT COASTAL TEXAS	DATE 8/3/2018	REVISION Draft
	CLIENT THE TEXAS GENERAL LAND OFFICE	BY CB/MJW	PLATE
		CHECK Draft	XX



PILE DIAMETER: 18-IN

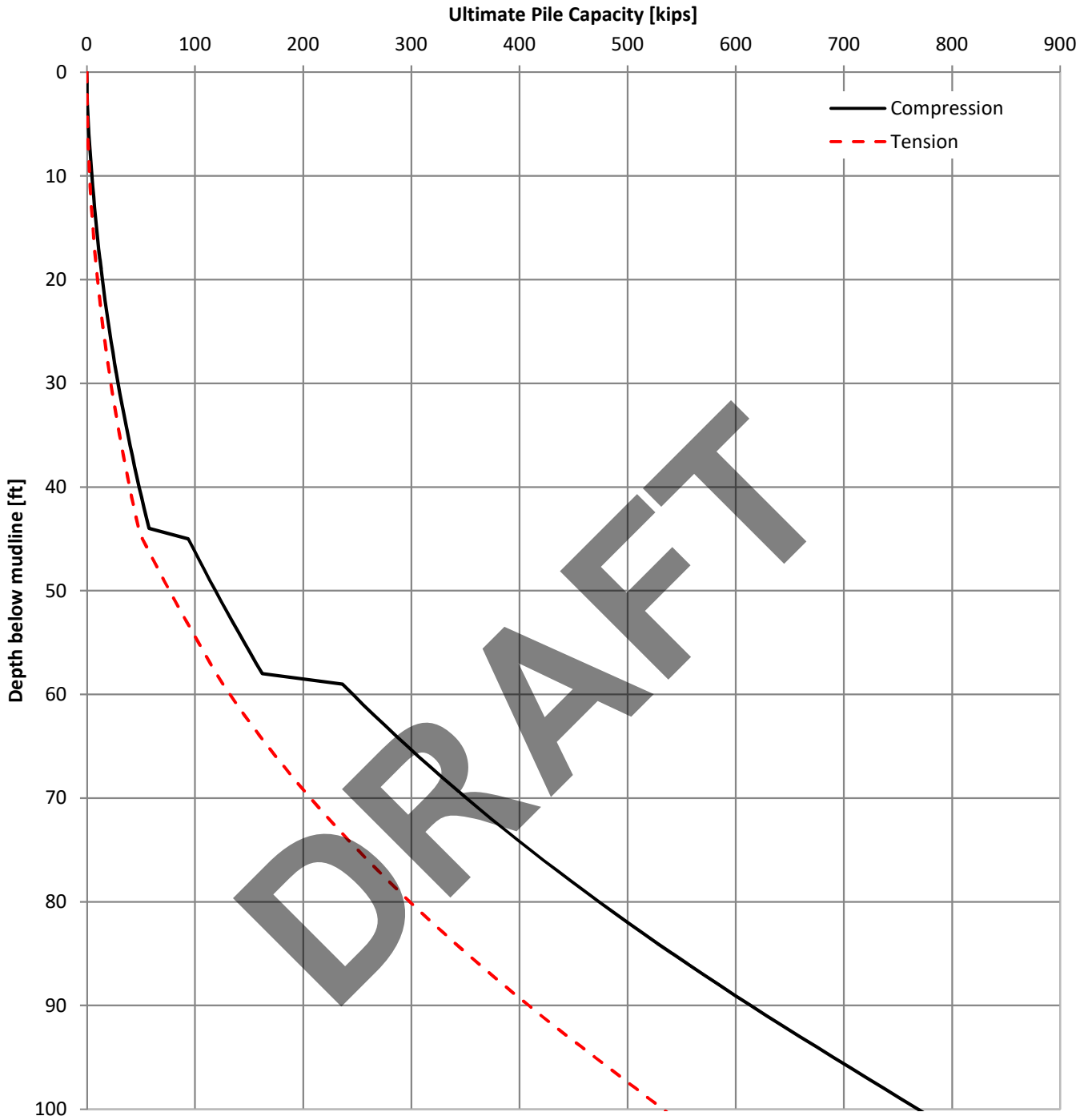
PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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		PLATE	XX



PILE DIAMETER: 24-IN

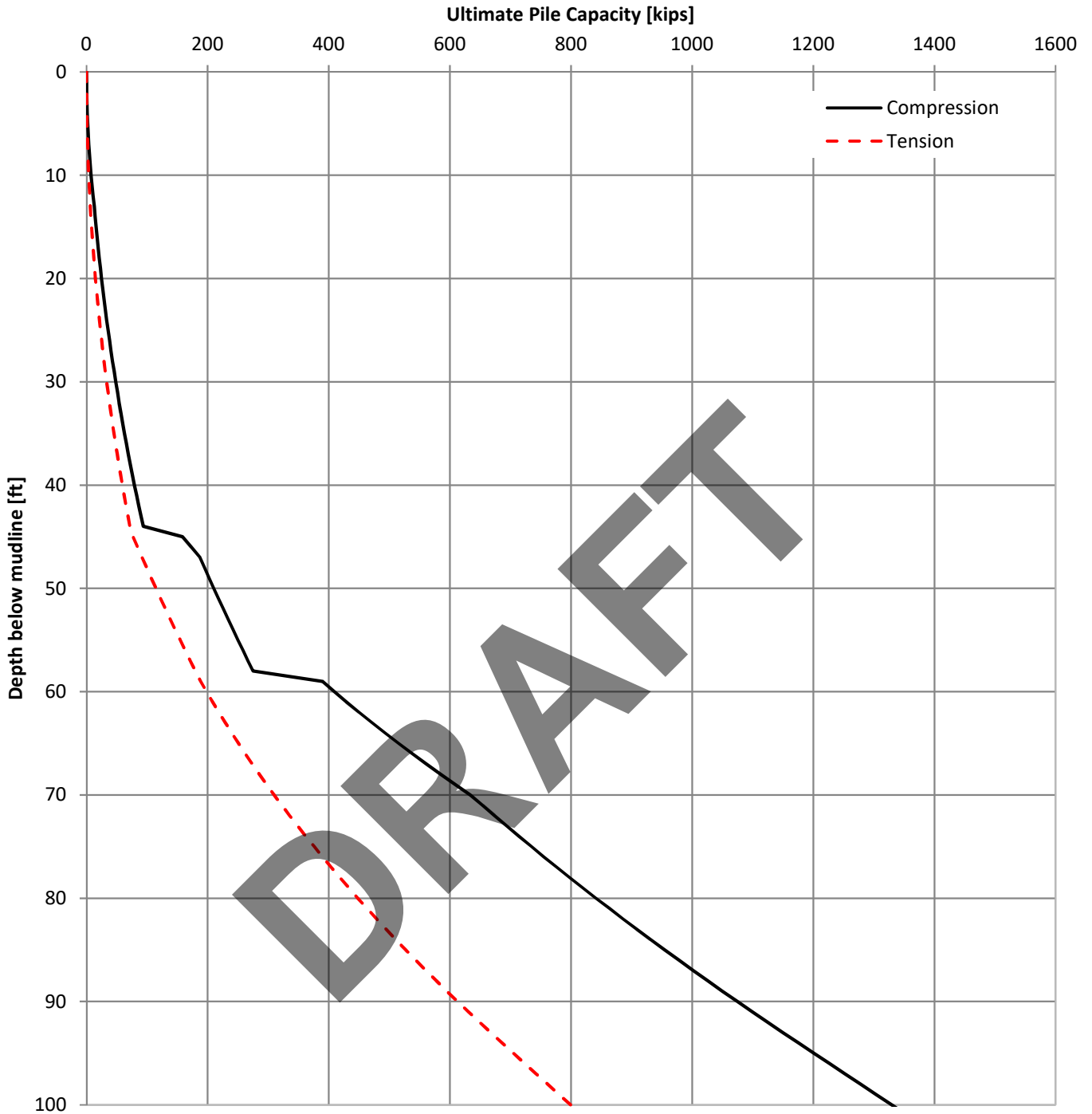
PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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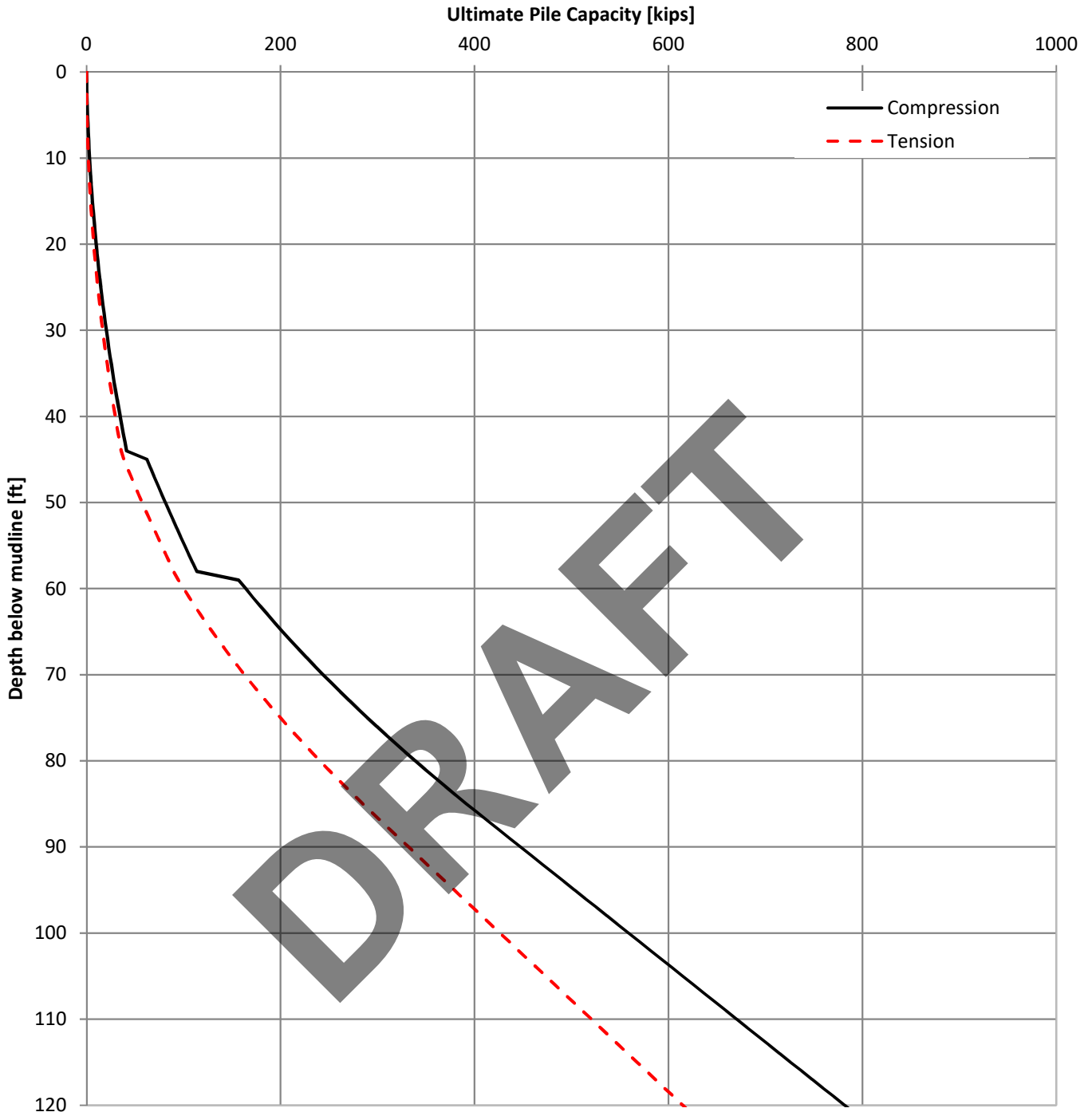


PILE DIAMETER: 36.00-IN PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
2. PILE CAPACITIES CALCULATED PER API RP 2A.
3. REFERENCE GEOTECH REPORT: MCBRIDE-RATCLIFF AND ASSOCIATES, DECEMBER 6, 1982, AND MARCH 14, 1985.
4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.

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			CHECK	Draft		



PILE DIAMETER: 18-IN

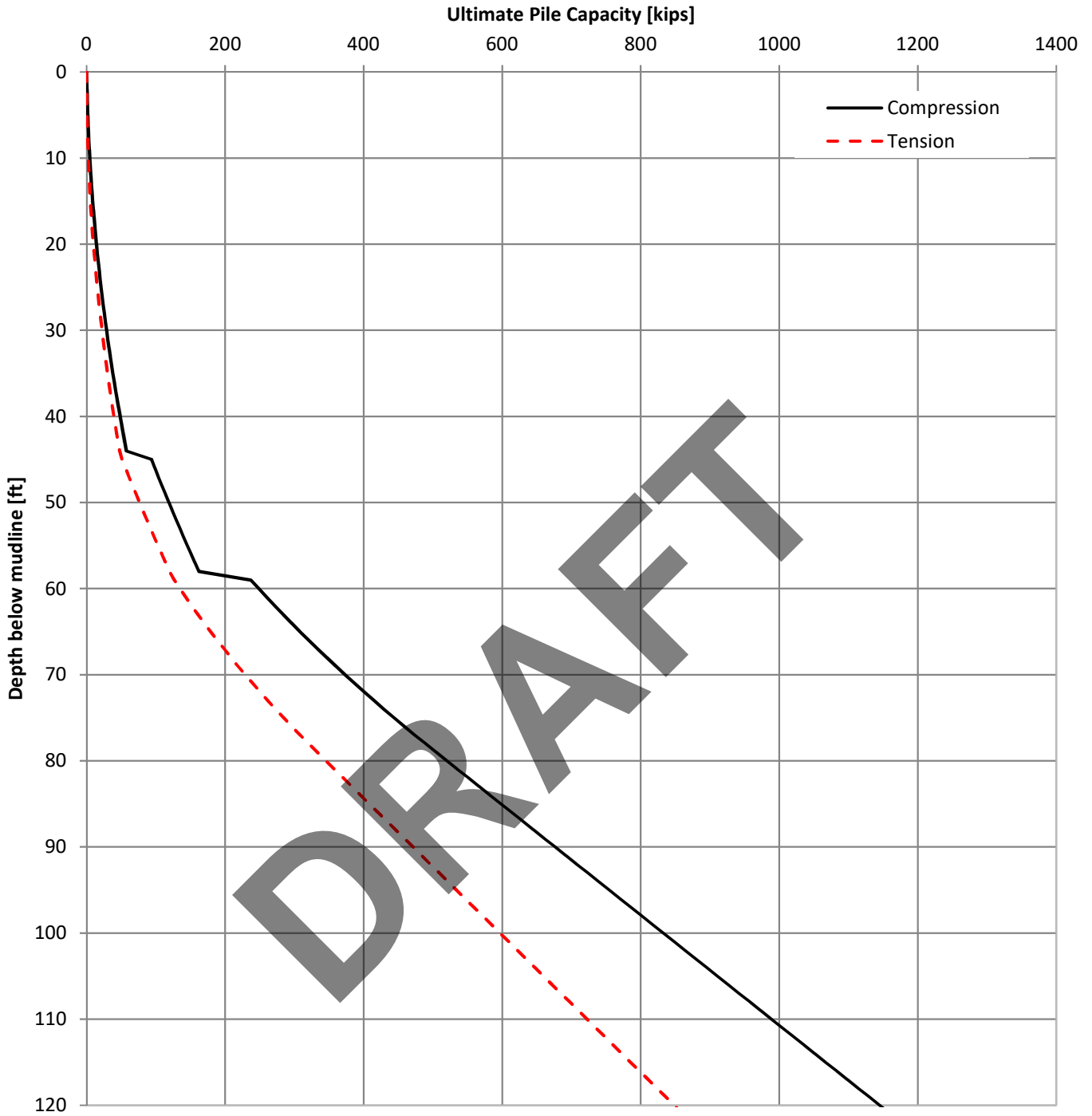
PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
2. PILE CAPACITIES CALCULATED PER API RP 2A.
3. REFERENCE GEOTECH REPORT: MCBRIDE-RATCLIFF AND ASSOCIATES, DECEMBER 6, 1982, AND MARCH 14, 1985.
4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.



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PILE DIAMETER: 24-IN

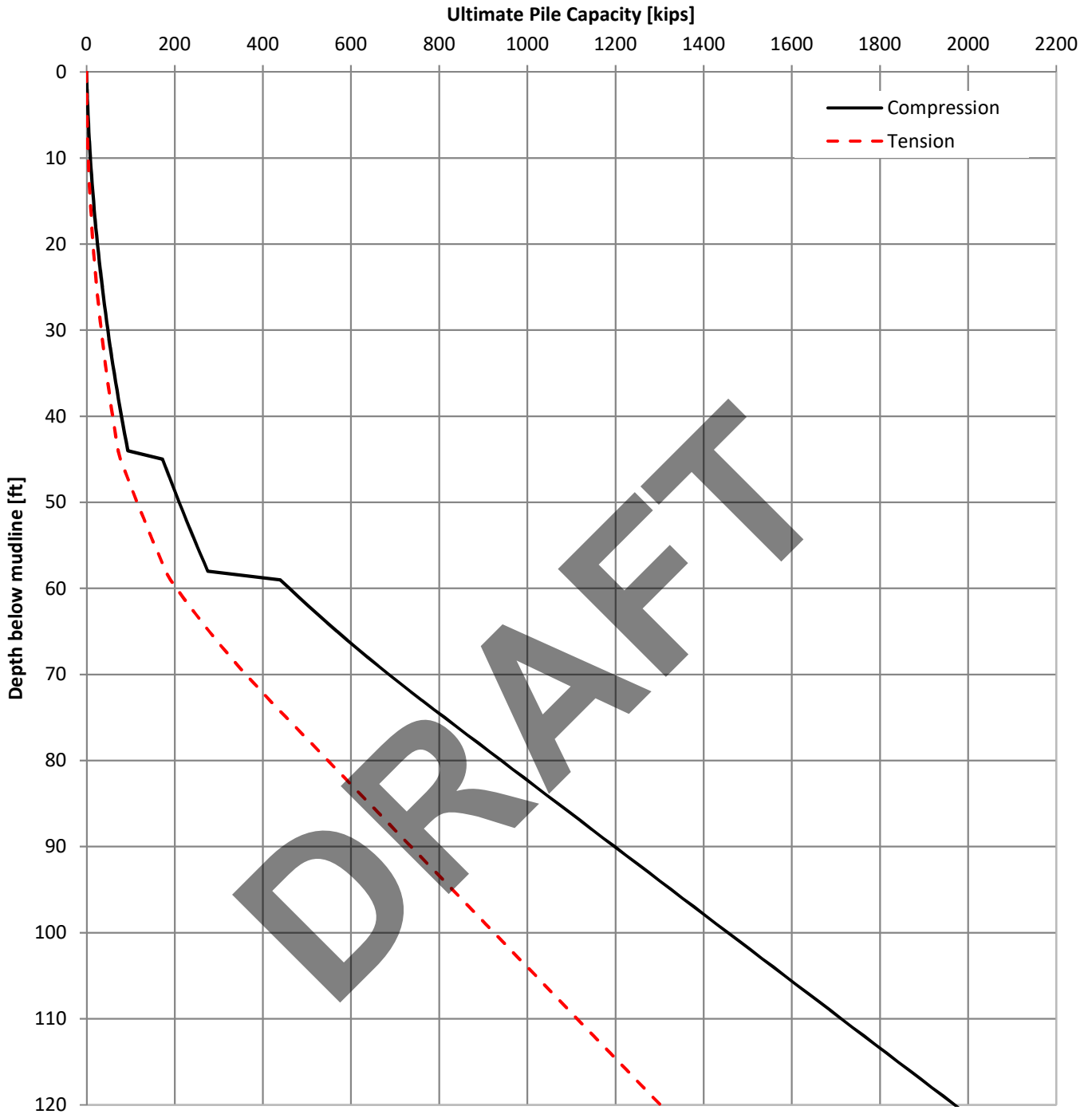
PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
2. PILE CAPACITIES CALCULATED PER API RP 2A.
3. REFERENCE GEOTECH REPORT: MCBRIDE-RATCLIFF AND ASSOCIATES, DECEMBER 6, 1982, AND MARCH 14, 1985.
4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.



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		PLATE	XX



PILE DIAMETER: 36-IN

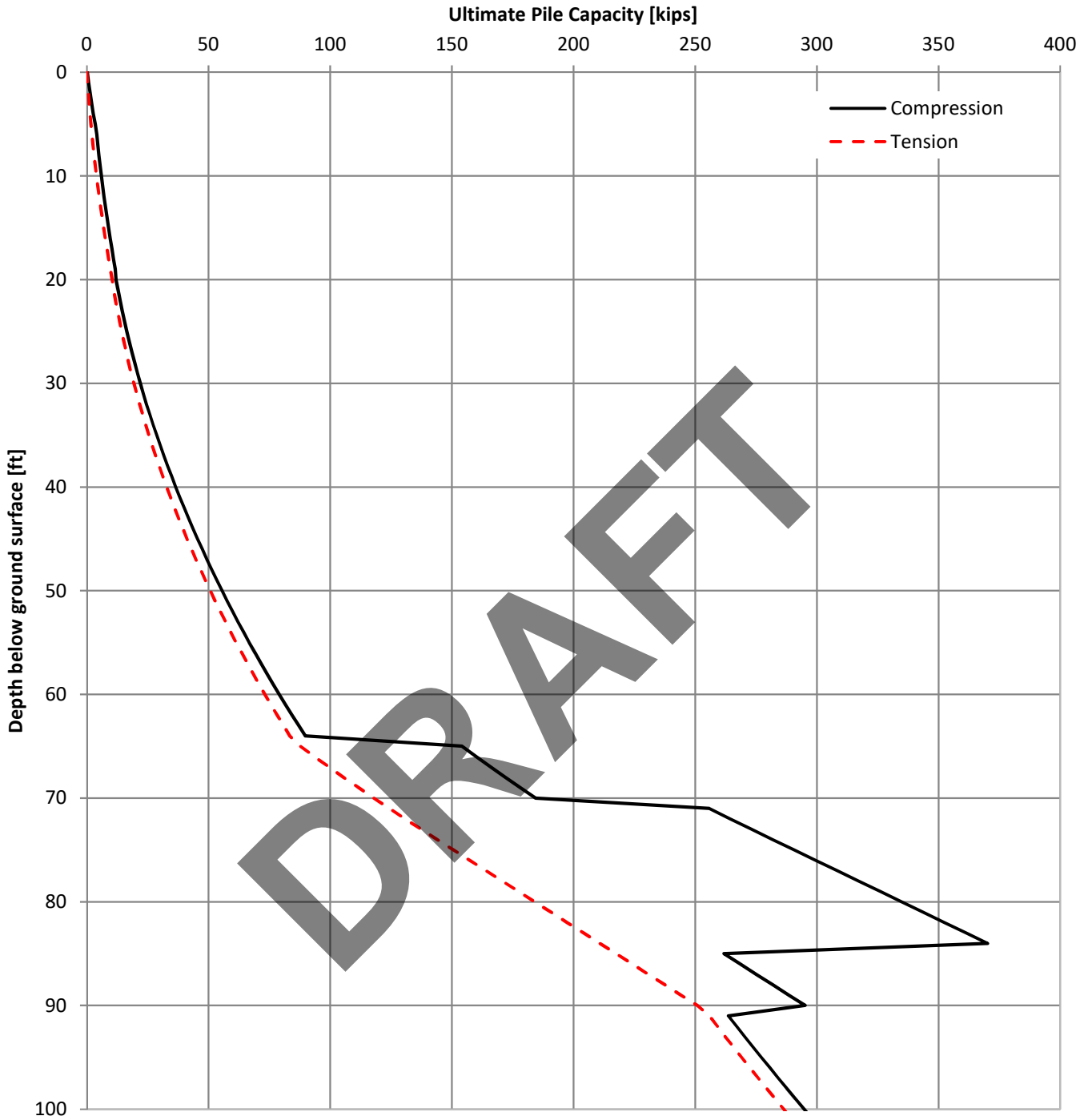
PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
2. PILE CAPACITIES CALCULATED PER API RP 2A.
3. REFERENCE GEOTECH REPORT: MCBRIDE-RATCLIFF AND ASSOCIATES, DECEMBER 6, 1982, AND MARCH 14, 1985.
4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.
5. DRIVABILITY OF LARGE DIAMETER SOLID SECTION CIRCULAR PILES SHOULD BE CONSIDERED. PILES LONGER THAN 80 FEET MAY BE INFEASIBLE.



SUBJECT	ULTIMATE PILE CAPACITY 36-IN CONCRETE PILE, CLEAR CREEK SITE		
PROJECT	COASTAL TEXAS	DATE	8/3/2018
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			XX

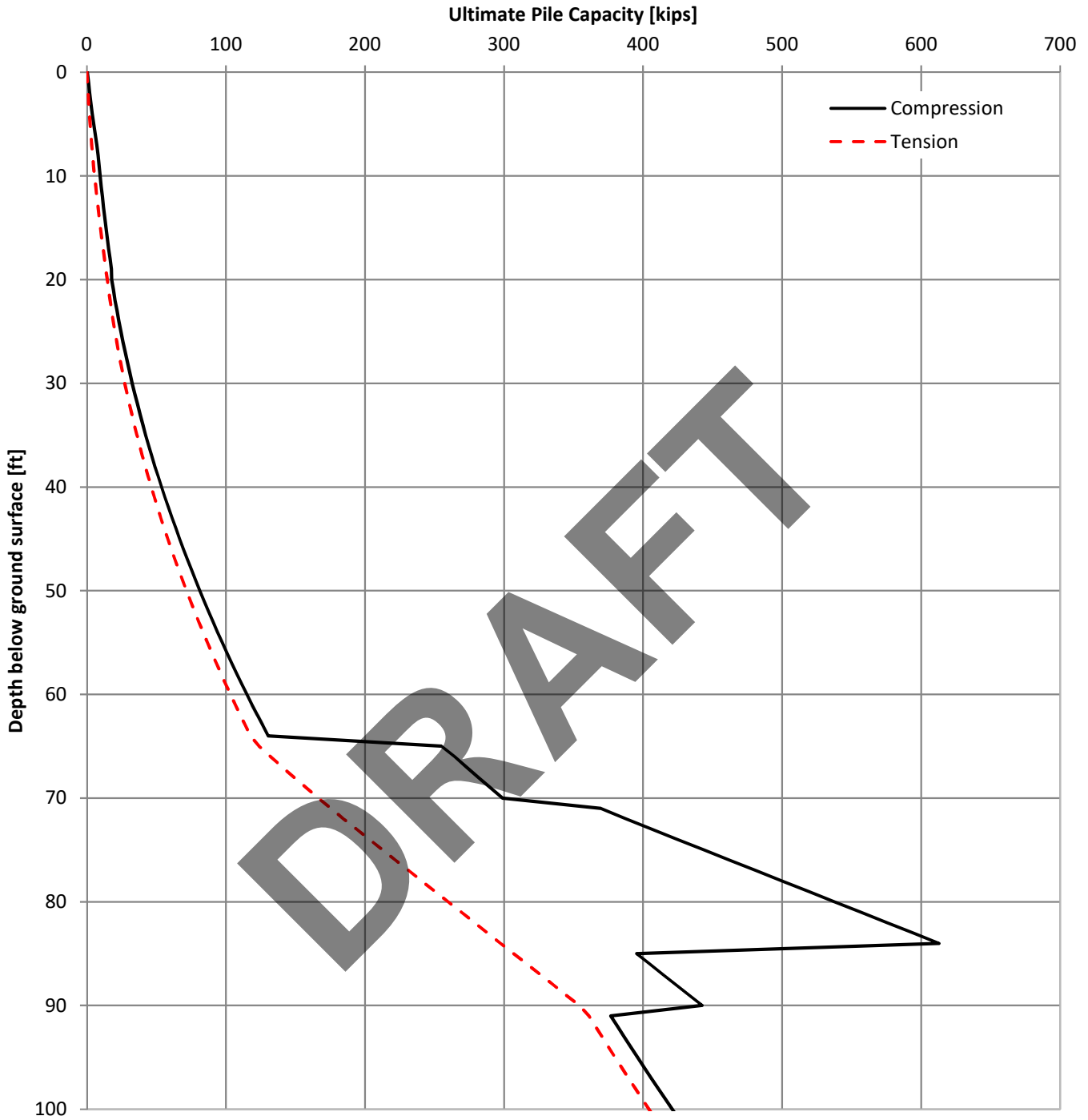


PILE DIAMETER: 12.75-IN PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
2. PILE CAPACITIES CALCULATED PER API RP 2A.
3. REFERENCE GEOTECH REPORT: GEOTECH ENGINEERING, INC. BOREHOLES COMPLETED FEB 2000. SH146 DICKINSON BAYOU BRIDGE.
4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.
5. GROUND SURFACE IDENTIFIED APPROXIMATELY ELEVATION 0.
6. CAPACITY INTENTIONALLY LIMITED IN UPPER AND LOWER 5 FEET OF DENSE SAND LAYER.

	SUBJECT ULTIMATE PILE CAPACITY 12.75-IN STEEL PIPE PILE, DICKINSON SITE					
	PROJECT	COASTAL TEXAS	DATE	8/15/2018	REVISION	Draft
	CLIENT	THE TEXAS GENERAL LAND OFFICE	BY	MJW	PLATE	XX
			CHECK	Draft		



PILE DIAMETER: 18-IN

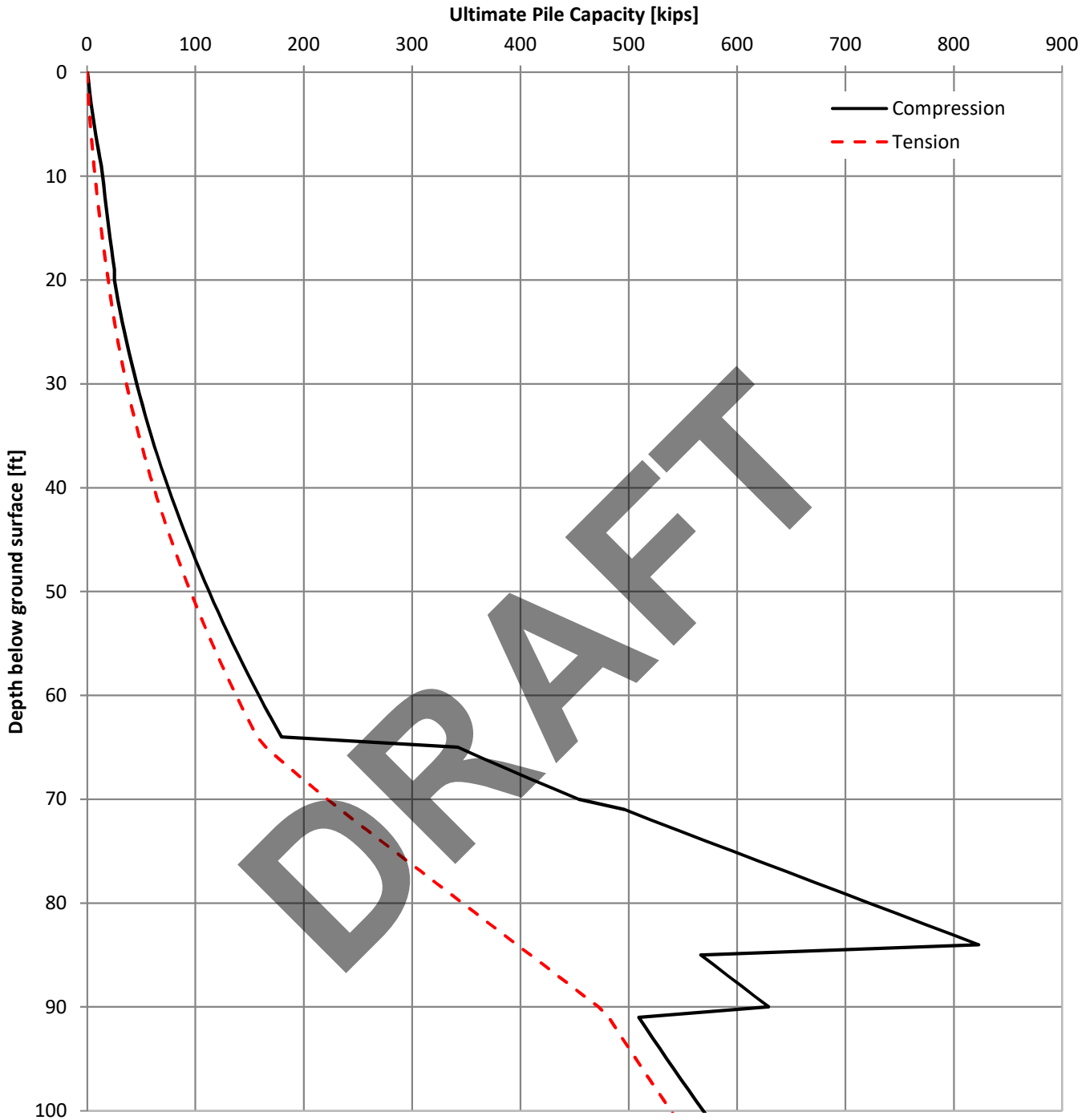
PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
2. PILE CAPACITIES CALCULATED PER API RP 2A.
3. REFERENCE GEOTECH REPORT: GEOTEST ENGINEERING, INC. BOREHOLES COMPLETED FEB 2000. SH146 DICKINSON BAYOU BRIDGE.
4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.
5. GROUND SURFACE IDENTIFIED APPROXIMATELY ELEVATION 0.
6. CAPACITY INTENTIONALLY LIMITED IN UPPER AND LOWER 5 FEET OF DENSE SAND LAYER.



SUBJECT ULTIMATE PILE CAPACITY 18-IN STEEL PIPE PILE, CLEAR CREEK SITE			
PROJECT	COASTAL TEXAS	DATE	8/15/2018
CLIENT	THE TEXAS GENERAL LAND OFFICE	BY	MJW
		CHECK	Draft
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PILE DIAMETER: 24-IN

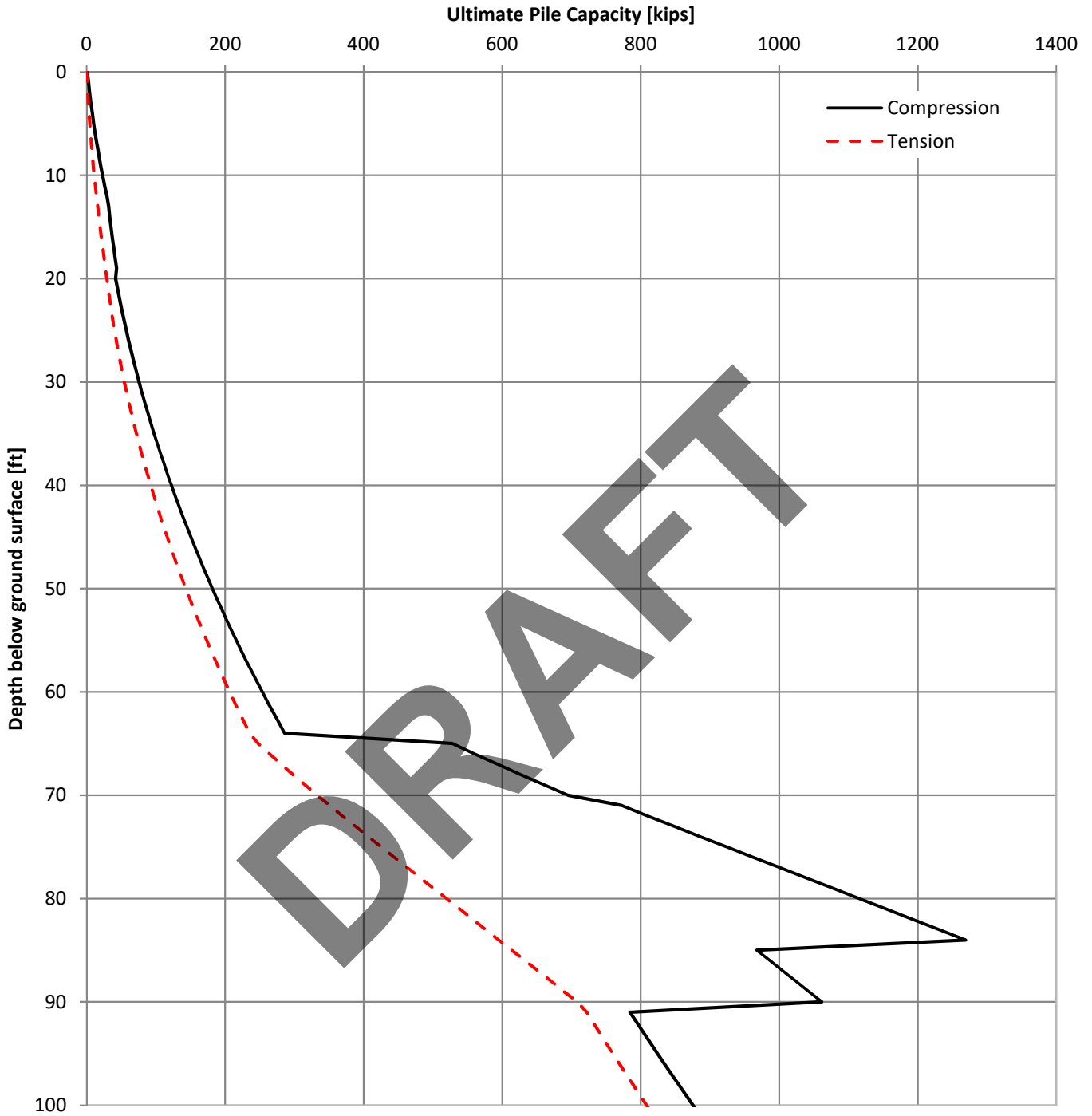
PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
2. PILE CAPACITIES CALCULATED PER API RP 2A.
3. REFERENCE GEOTECH REPORT: GEOTEST ENGINEERING, INC. BOREHOLES COMPLETED FEB 2000. SH146 DICKINSON BAYOU BRIDGE.
4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.
5. GROUND SURFACE IDENTIFIED APPROXIMATELY ELEVATION 0.
6. CAPACITY INTENTIONALLY LIMITED IN UPPER AND LOWER 5 FEET OF DENSE SAND LAYER.



SUBJECT ULTIMATE PILE CAPACITY 24-IN STEEL PIPE PILE, DICKINSON SITE			
PROJECT	COASTAL TEXAS	DATE	8/15/2018
CLIENT	THE TEXAS GENERAL LAND OFFICE	BY	MJW
		CHECK	Draft
		REVISION	Draft
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PILE DIAMETER: 36.00-IN

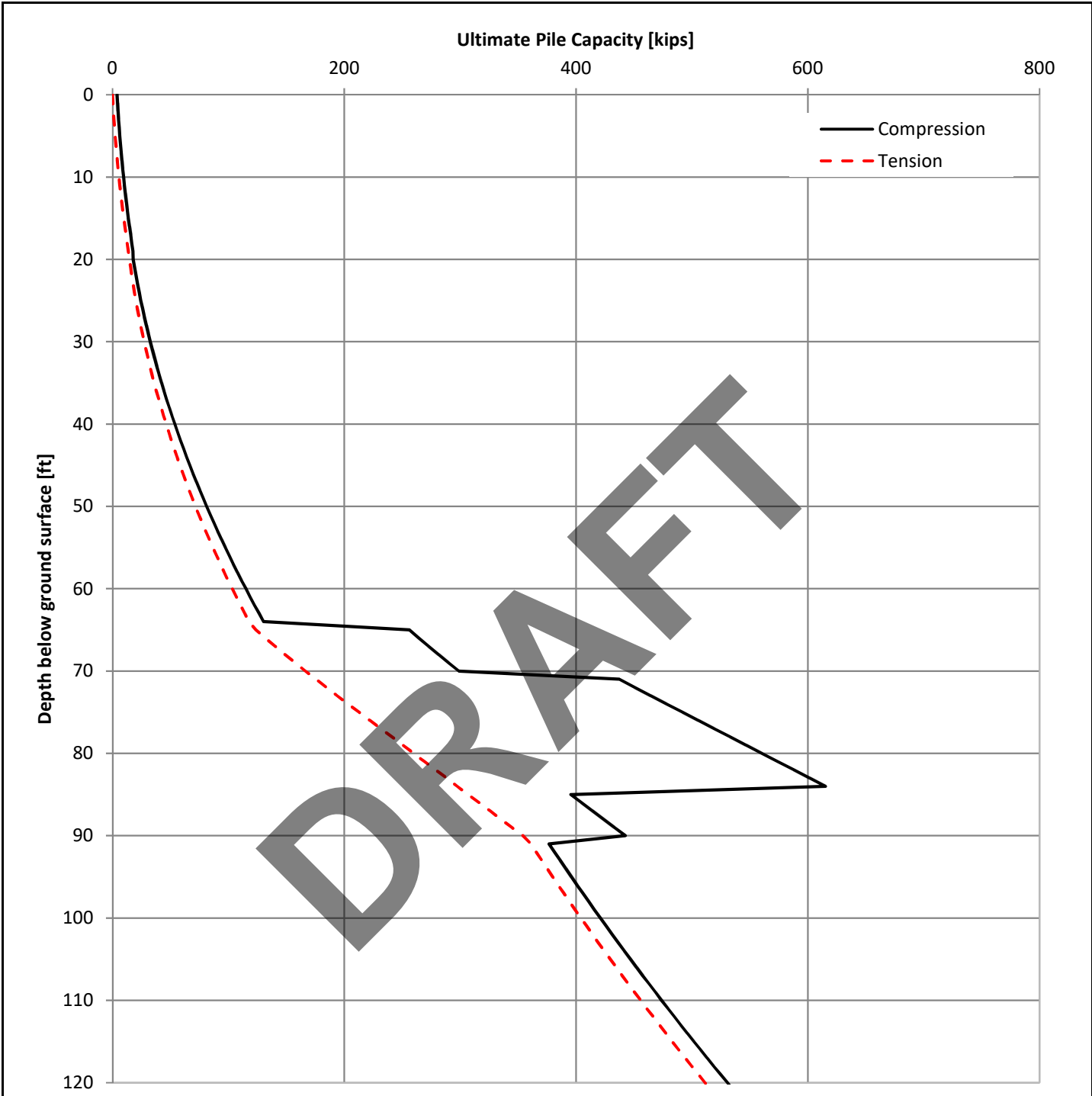
PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
2. PILE CAPACITIES CALCULATED PER API RP 2A.
3. REFERENCE GEOTECH REPORT: GEOTEST ENGINEERING, INC. BOREHOLES COMPLETED FEB 2000. SH146 DICKINSON BAYOU BRIDGE.
4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.
5. GROUND SURFACE IDENTIFIED APPROXIMATELY ELEVATION 0.
6. CAPACITY INTENTIONALLY LIMITED IN UPPER AND LOWER 5 FEET OF DENSE SAND LAYER.



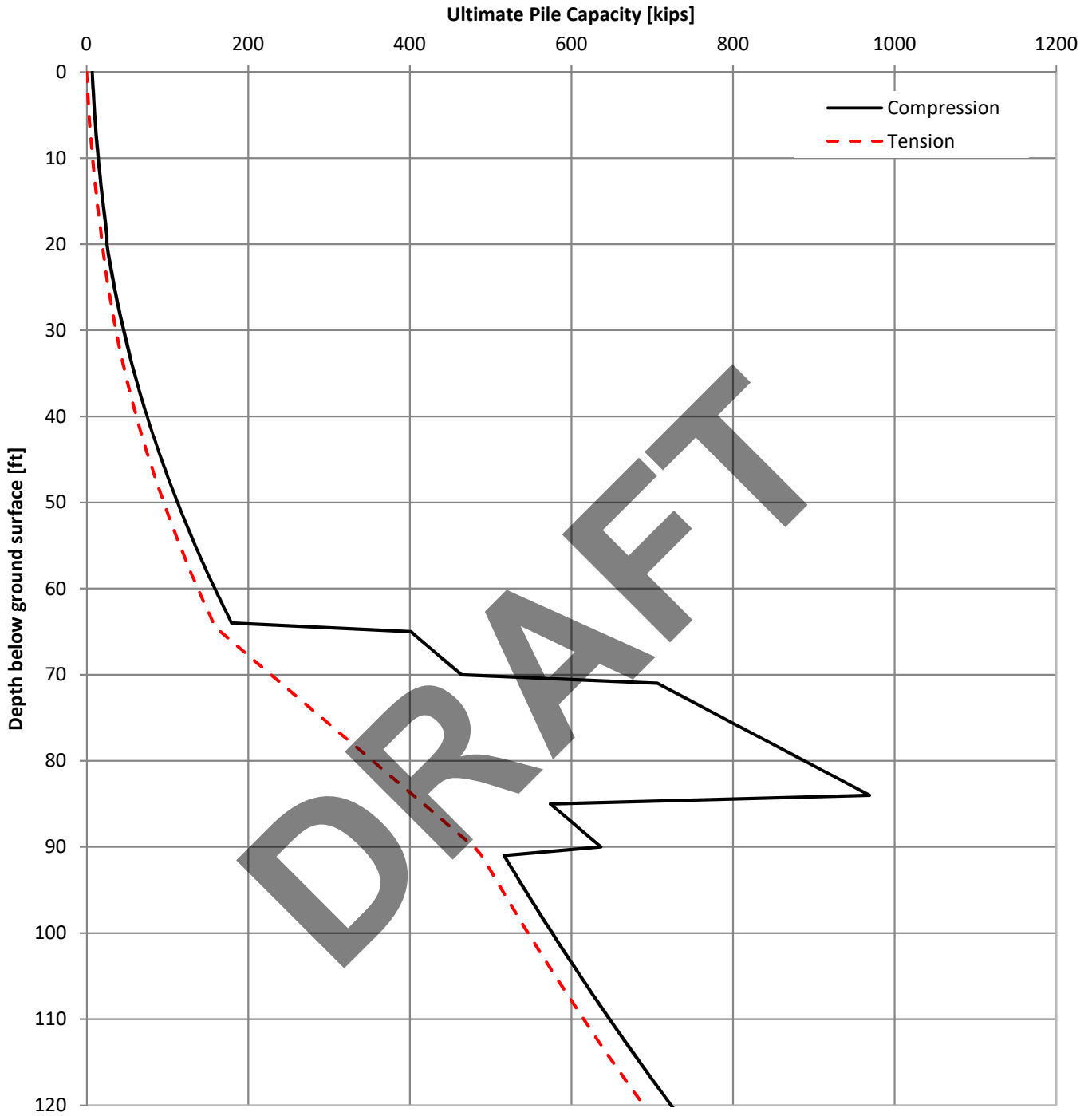
SUBJECT ULTIMATE PILE CAPACITY 36-IN STEEL PIPE PILE, DICKINSON SITE			
PROJECT	COASTAL TEXAS	DATE	8/15/2018
CLIENT	THE TEXAS GENERAL LAND OFFICE	BY	MJW
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PILE DIAMETER: 18-IN PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

- NOTES:**
1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
 2. PILE CAPACITIES CALCULATED PER API RP 2A.
 3. REFERENCE GEOTECH REPORT: GEOTECH ENGINEERING, INC. BOREHOLES COMPLETED FEB 2000. SH146 DICKINSON BAYOU BRIDGE.
 4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.
 5. GROUND SURFACE IDENTIFIED APPROXIMATELY ELEVATION 0.
 6. CAPACITY INTENTIONALLY LIMITED IN UPPER AND LOWER 5 FEET OF DENSE SAND LAYER.

	SUBJECT ULTIMATE PILE CAPACITY 18-IN CONCRETE PILE, DICKINSON SITE		
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PILE DIAMETER: 24-IN

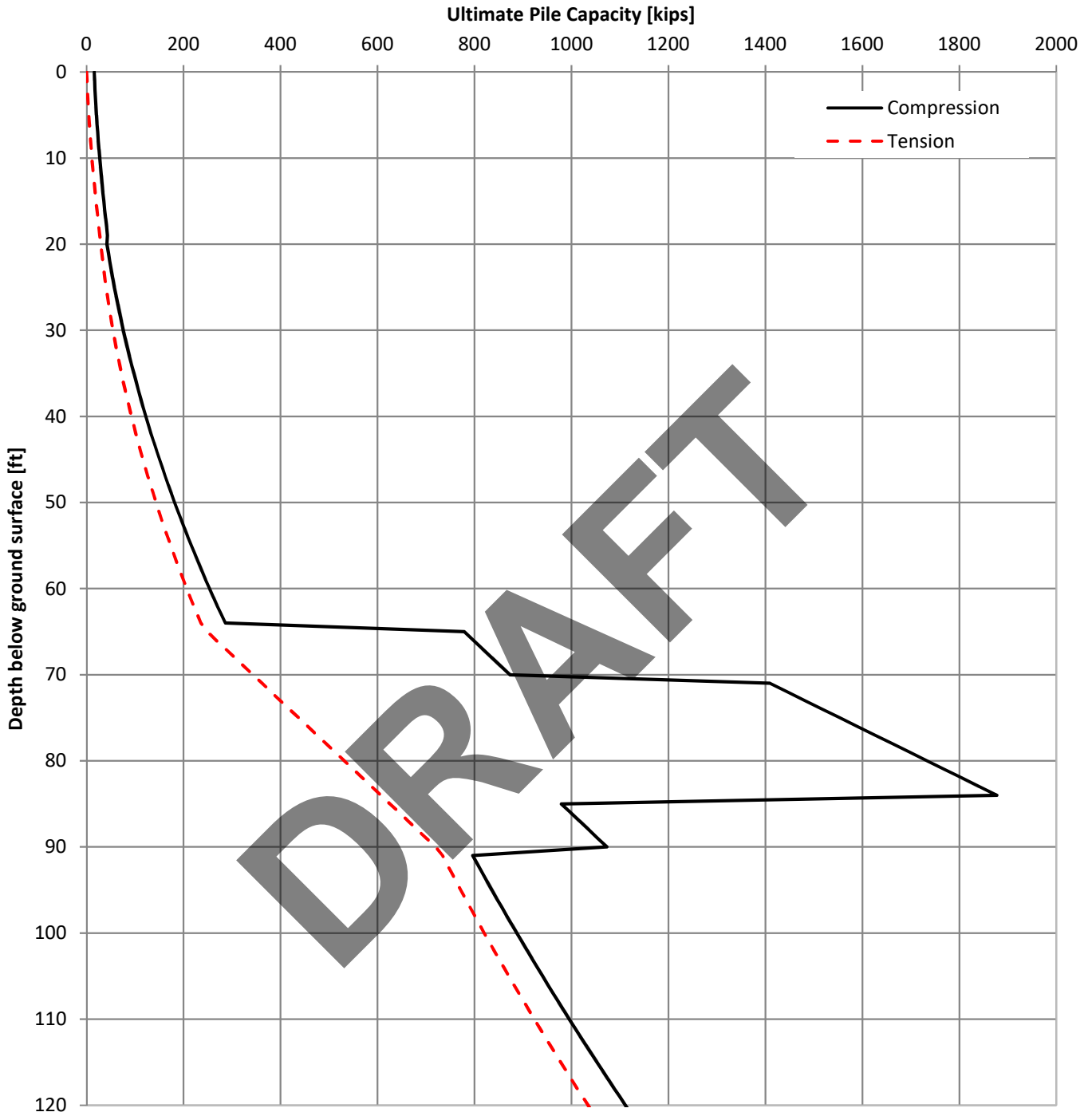
PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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SUBJECT ULTIMATE PILE CAPACITY 24-IN CONCRETE PILE, DICKINSON SITE			
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PILE DIAMETER: 36-IN

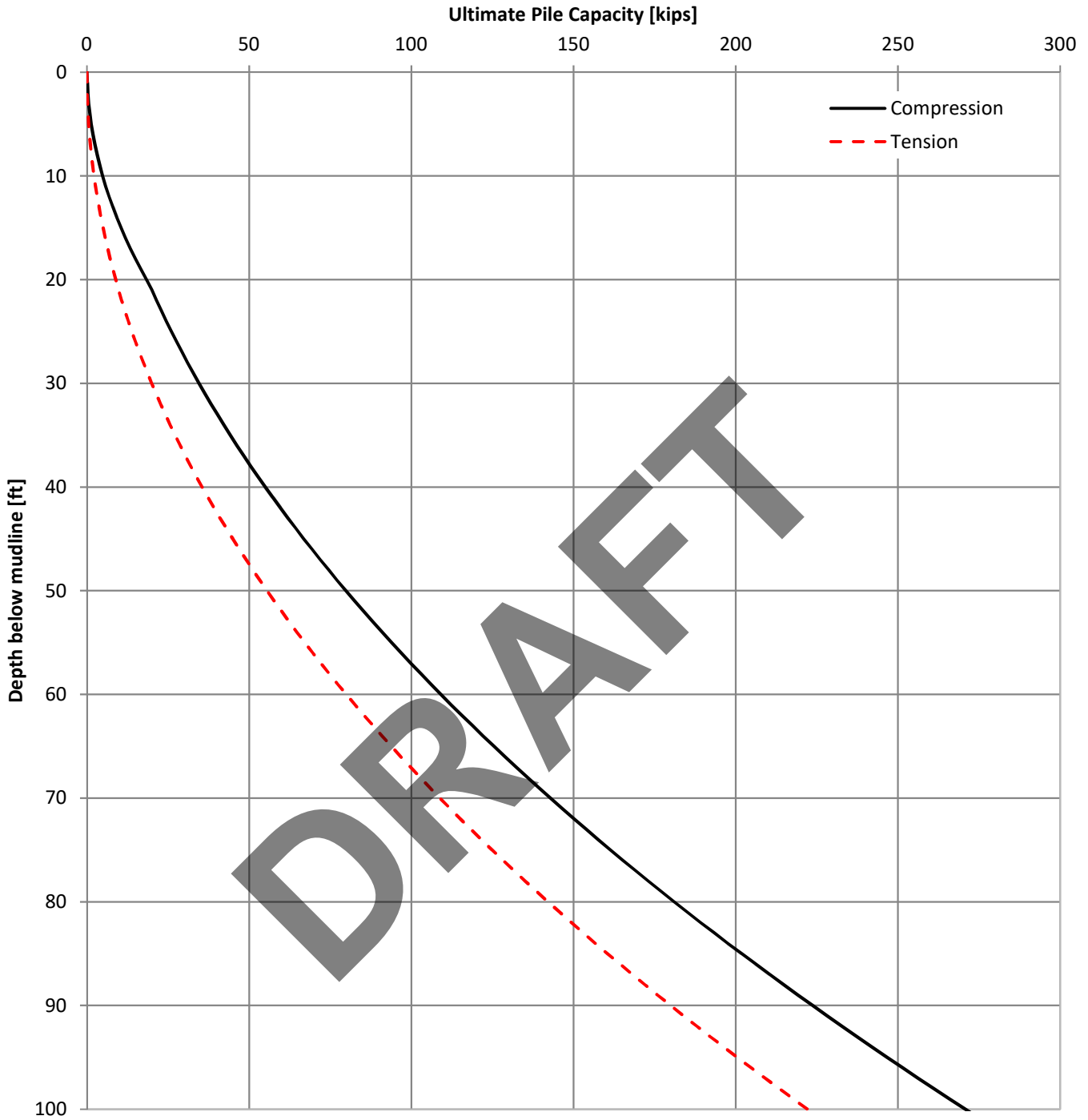
PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.
5. GROUND SURFACE IDENTIFIED APPROXIMATELY ELEVATION 0.
6. CAPACITY INTENTIONALLY LIMITED IN UPPER AND LOWER 5 FEET OF DENSE SAND LAYER.
7. DRIVABILITY OF LARGE DIAMETER SOLID SECTION CIRCULAR PILES SHOULD BE CONSIDERED. PILES LONGER THAN 80 FEET MAY BE INFEASIBLE.



SUBJECT ULTIMATE PILE CAPACITY 36-IN CONCRETE PILE, DICKINSON SITE			
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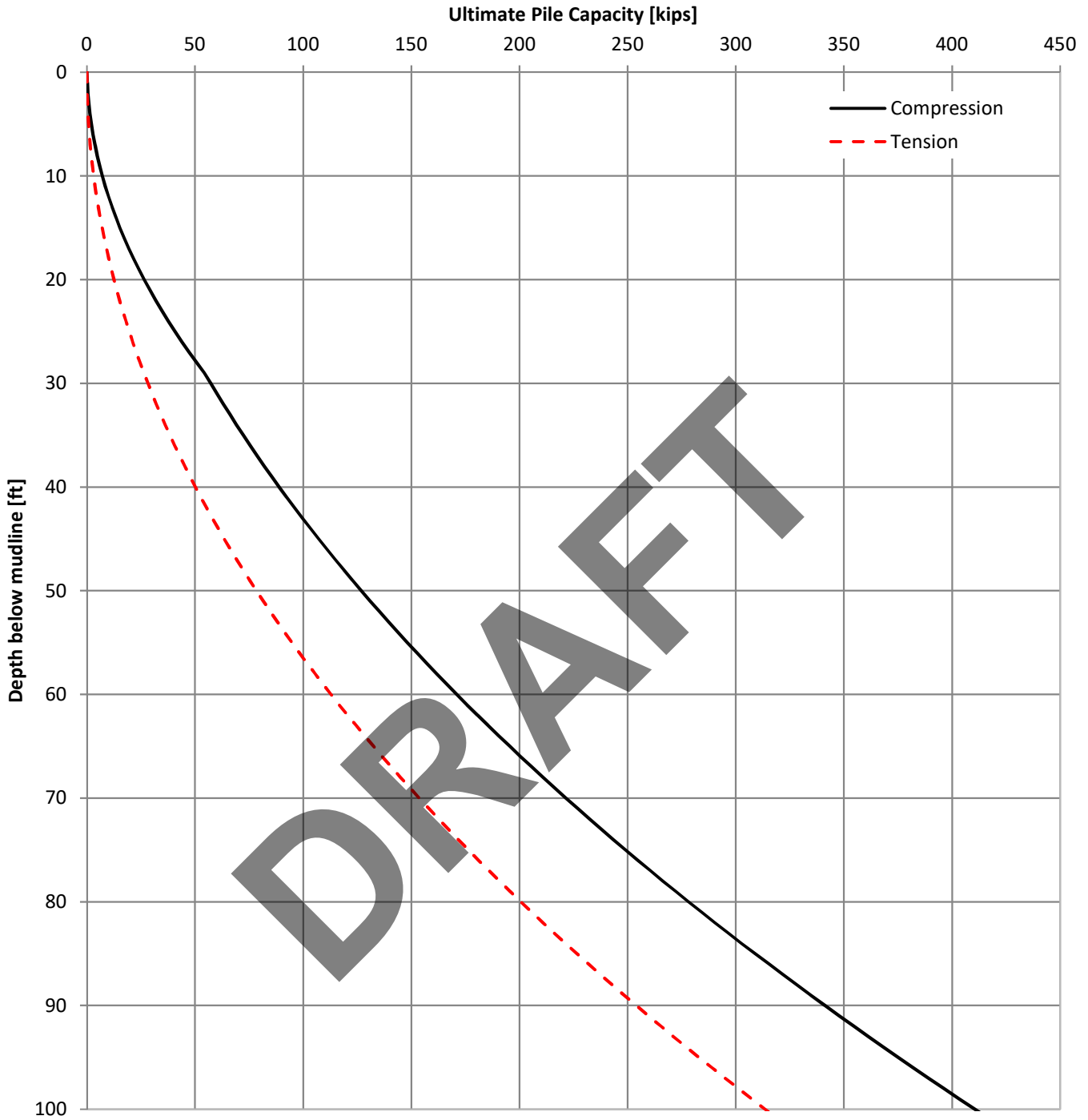


PILE DIAMETER: 12.75-IN PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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3. GEOTECHNICAL REFERENCE INFORMATION FROM TURNER & COLLIE, DECEMBER 1961, AND FUGRO SOUTH, JUNE 10, 2003. MAX DEPTH OF LOCAL INFORMATION IS 35 FEET BELOW GRADE, AND CAPACITIES CALCULATED BEYOND THAT DEPTH ARE EXTRAPOLATIONS.
4. CAPACITIES PROVIDED FOR FEASIBILITY DESIGN ONLY. NOT FOR CONSTRUCTION OR DETAILED DESIGN.

	SUBJECT ULTIMATE PILE CAPACITY 12.75-IN STEEL PIPE PILE, GALVESTON EAST SITE		
	PROJECT COASTAL TEXAS	DATE 8/3/2018	REVISION Draft
	CLIENT THE TEXAS GENERAL LAND OFFICE	BY CB / MJW	PLATE
		CHECK Draft	XX



PILE DIAMETER: 18-IN

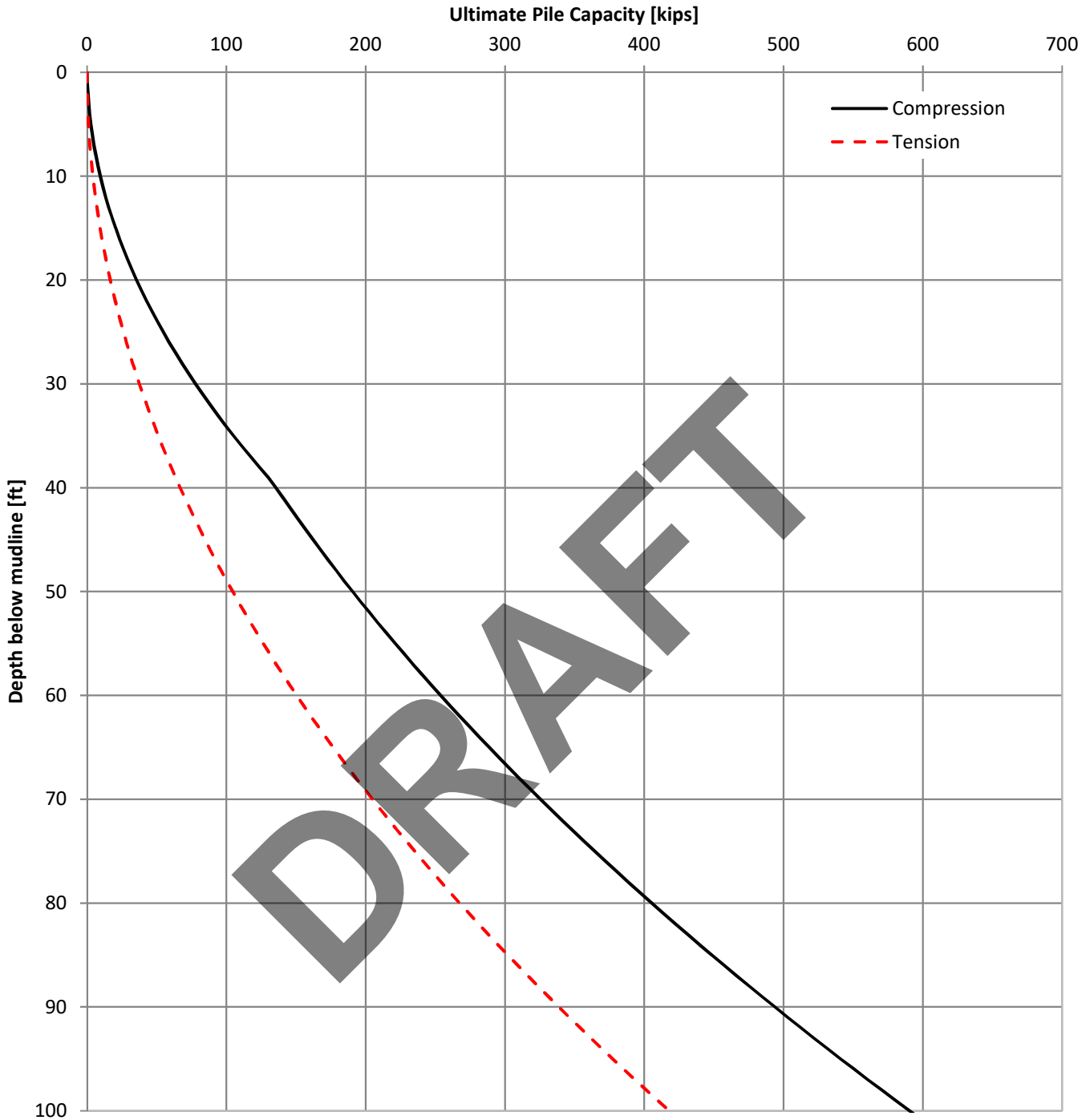
PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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SUBJECT ULTIMATE PILE CAPACITY 18-IN STEEL PIPE PILE, GALVESTON EAST SITE			
PROJECT	COASTAL TEXAS	DATE	8/3/2018
CLIENT	THE TEXAS GENERAL LAND OFFICE	BY	C Brodbaek
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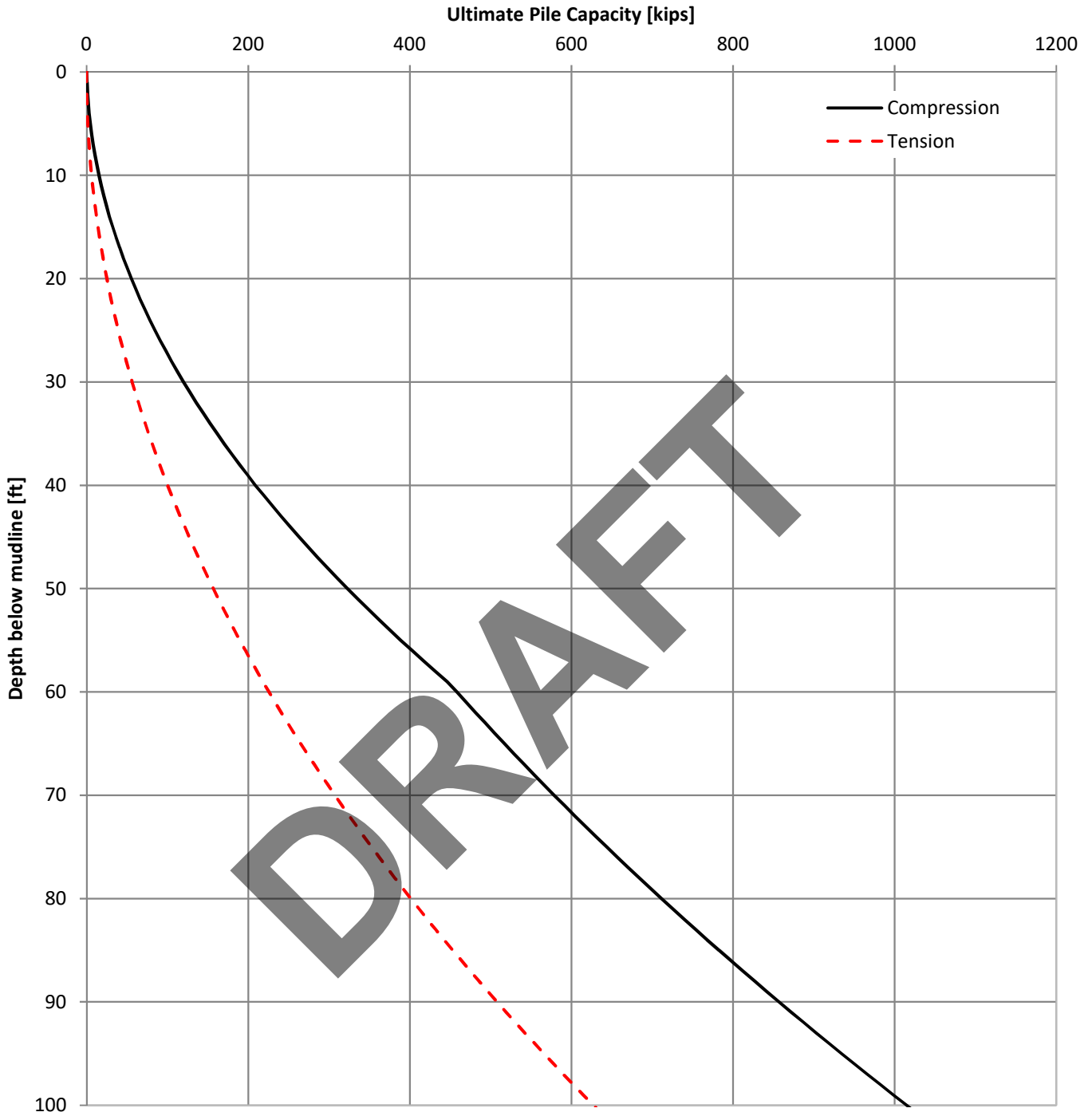
PILE DIAMETER: 24-IN

PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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	SUBJECT ULTIMATE PILE CAPACITY 24-IN STEEL PIPE PILE, GALVESTON EAST SITE			
	PROJECT	COASTAL TEXAS	DATE	8/3/2018
	CLIENT	THE TEXAS GENERAL LAND OFFICE	BY	C Brodbaek
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PILE DIAMETER: 36-IN

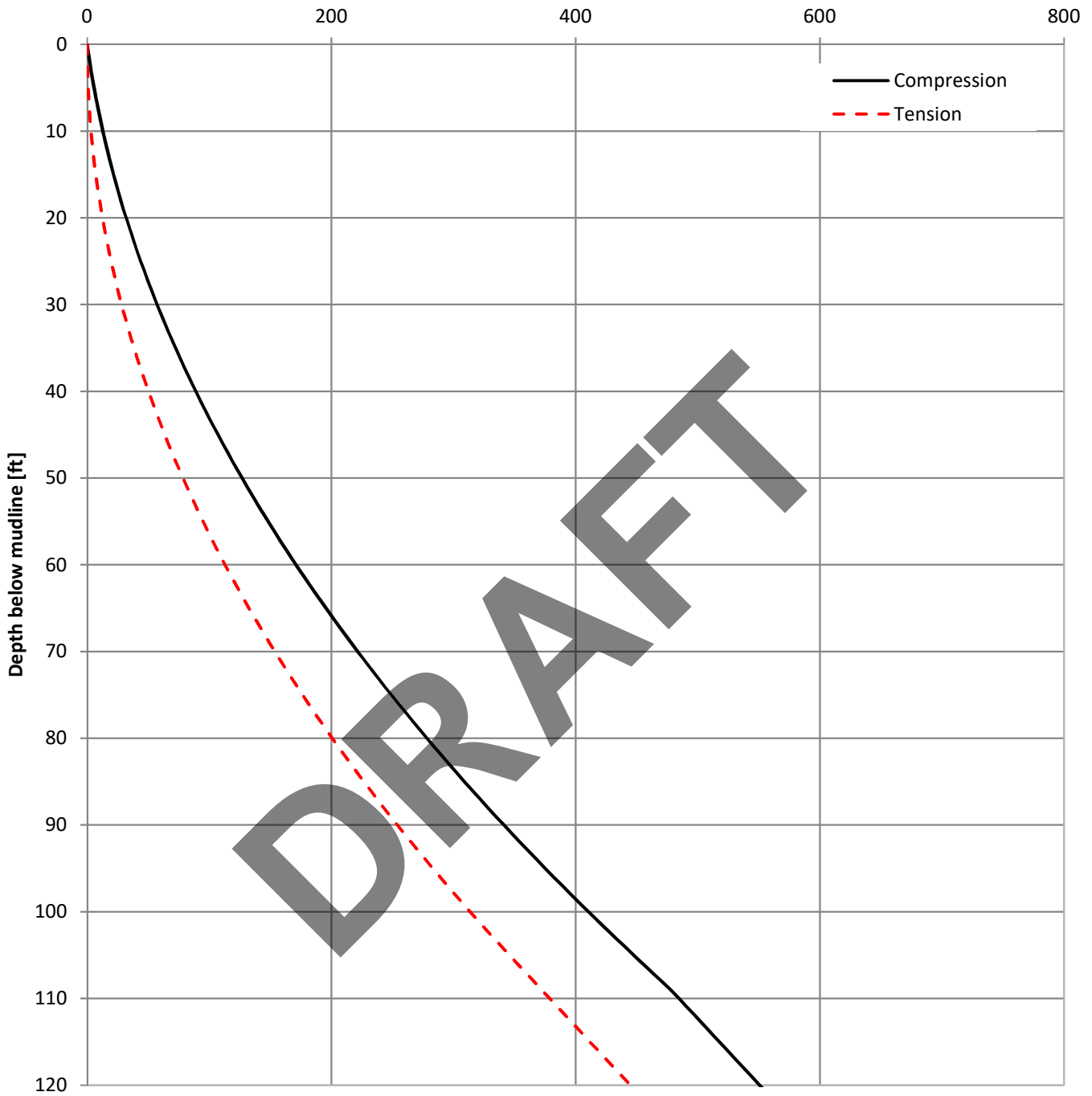
PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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	SUBJECT ULTIMATE PILE CAPACITY 36-IN STEEL PIPE PILE, GALVESTON EAST SITE					
	PROJECT	COASTAL TEXAS	DATE	8/3/2018	REVISION	Draft
	CLIENT	THE TEXAS GENERAL LAND OFFICE	BY	CB / MJW	PLATE	XX
			CHECK	Draft		

Ultimate Pile Capacity [kips]



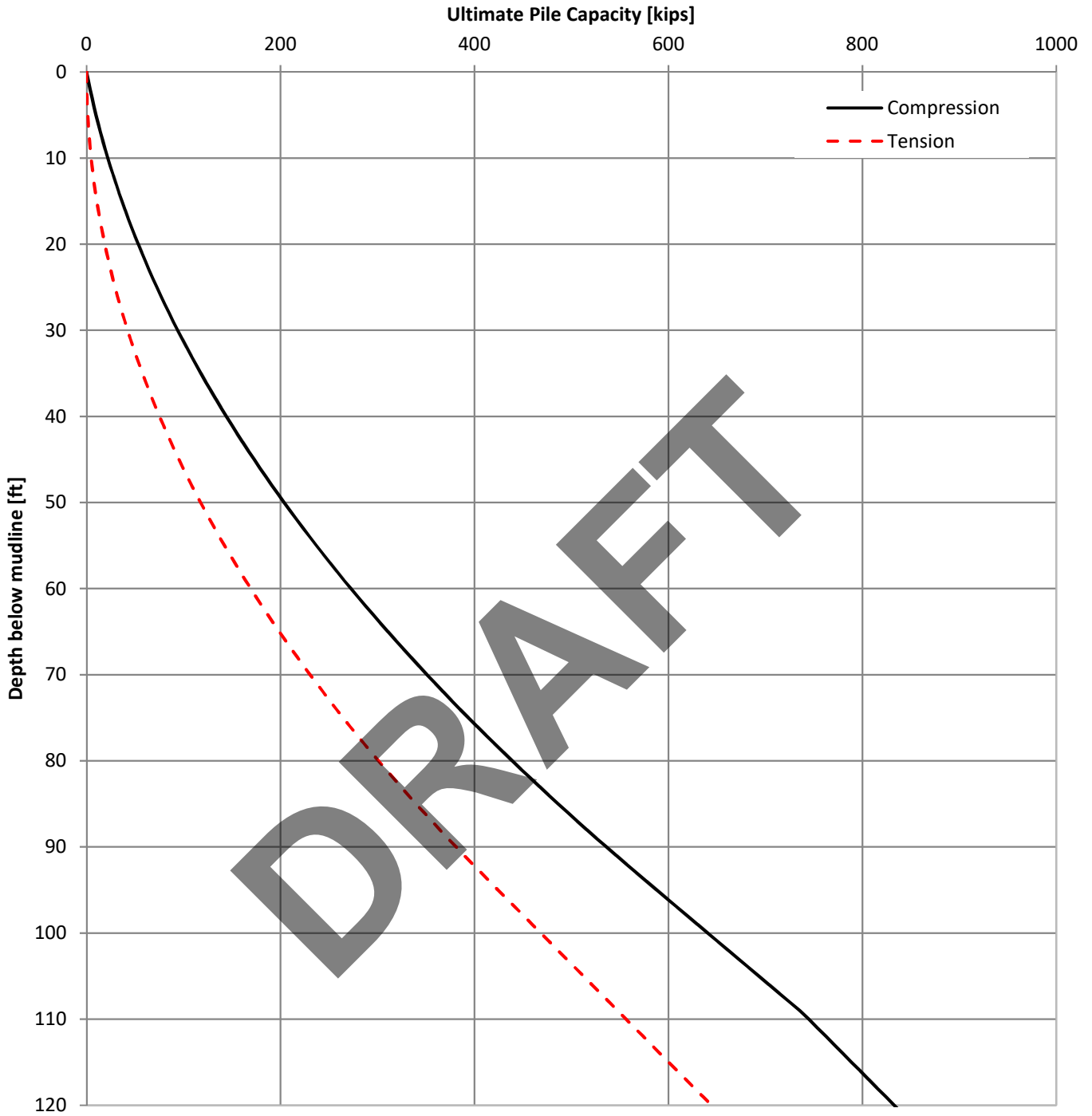
PILE DIAMETER: 18-IN

PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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	SUBJECT ULTIMATE PILE CAPACITY 18-IN CONCRETE PILE, GALVESTON EAST SITE		
	PROJECT COASTAL TEXAS	DATE 8/3/2018	REVISION Draft
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PILE DIAMETER: 24-IN

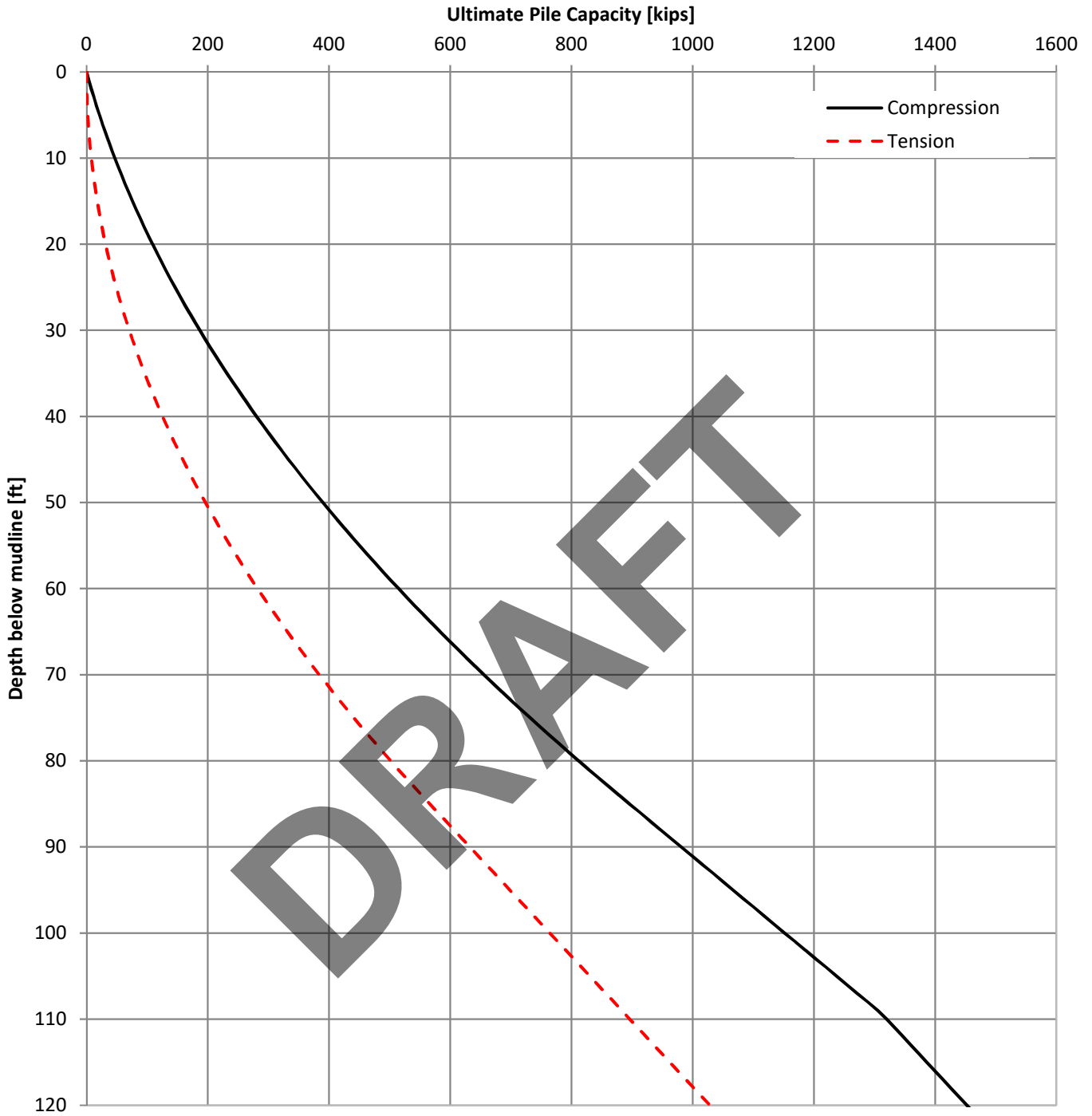
PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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SUBJECT	ULTIMATE PILE CAPACITY 24-IN CONCRETE PILE, GALVESTON EAST SITE		
PROJECT	COASTAL TEXAS	DATE	8/3/2018
CLIENT	THE TEXAS GENERAL LAND OFFICE	BY	CB / MJW
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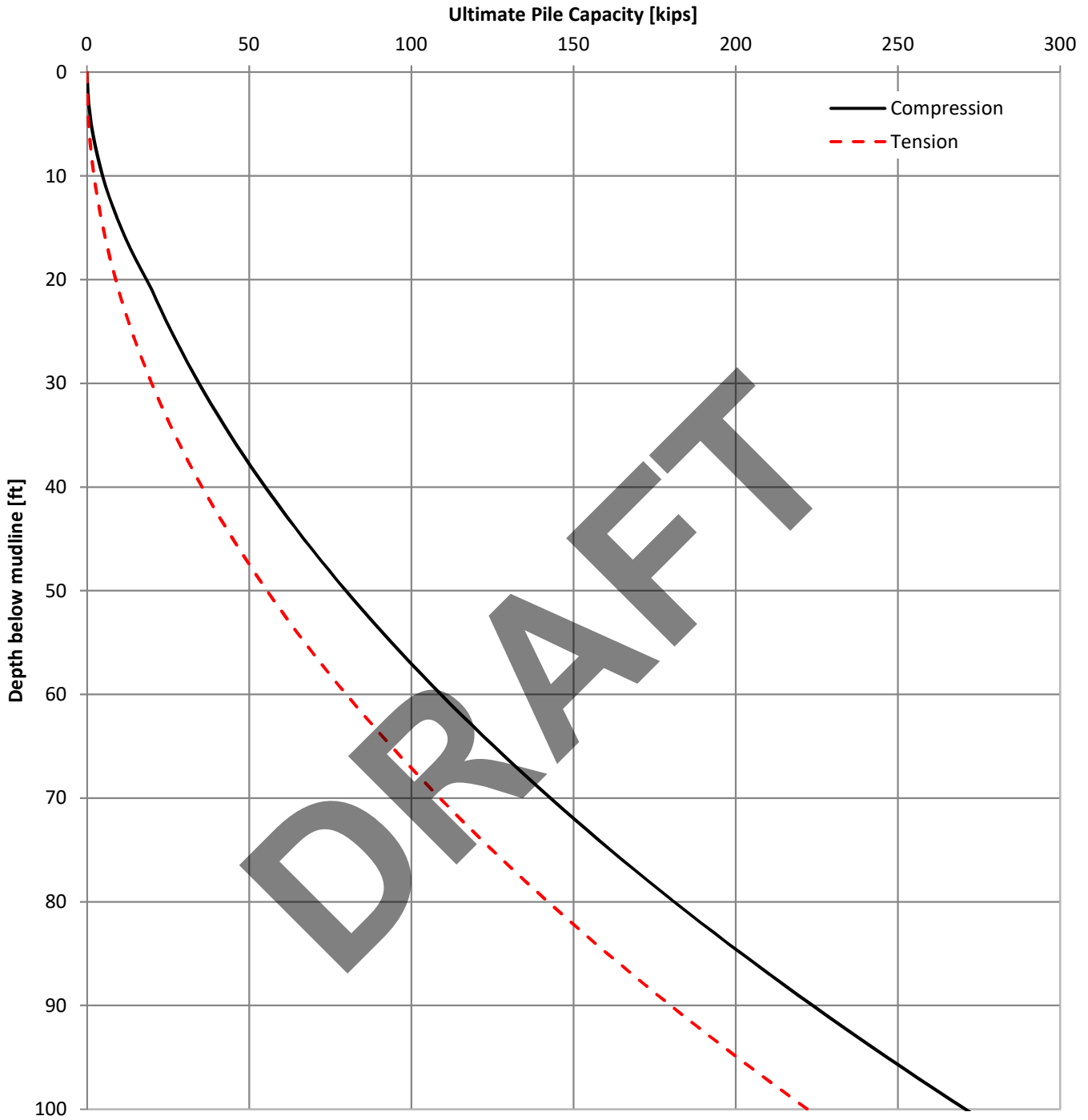
PILE DIAMETER: 36-IN

PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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	SUBJECT ULTIMATE PILE CAPACITY 36-IN CONCRETE PILE, GALVESTON EAST SITE		
	PROJECT COASTAL TEXAS	DATE 8/3/2018	REVISION Draft
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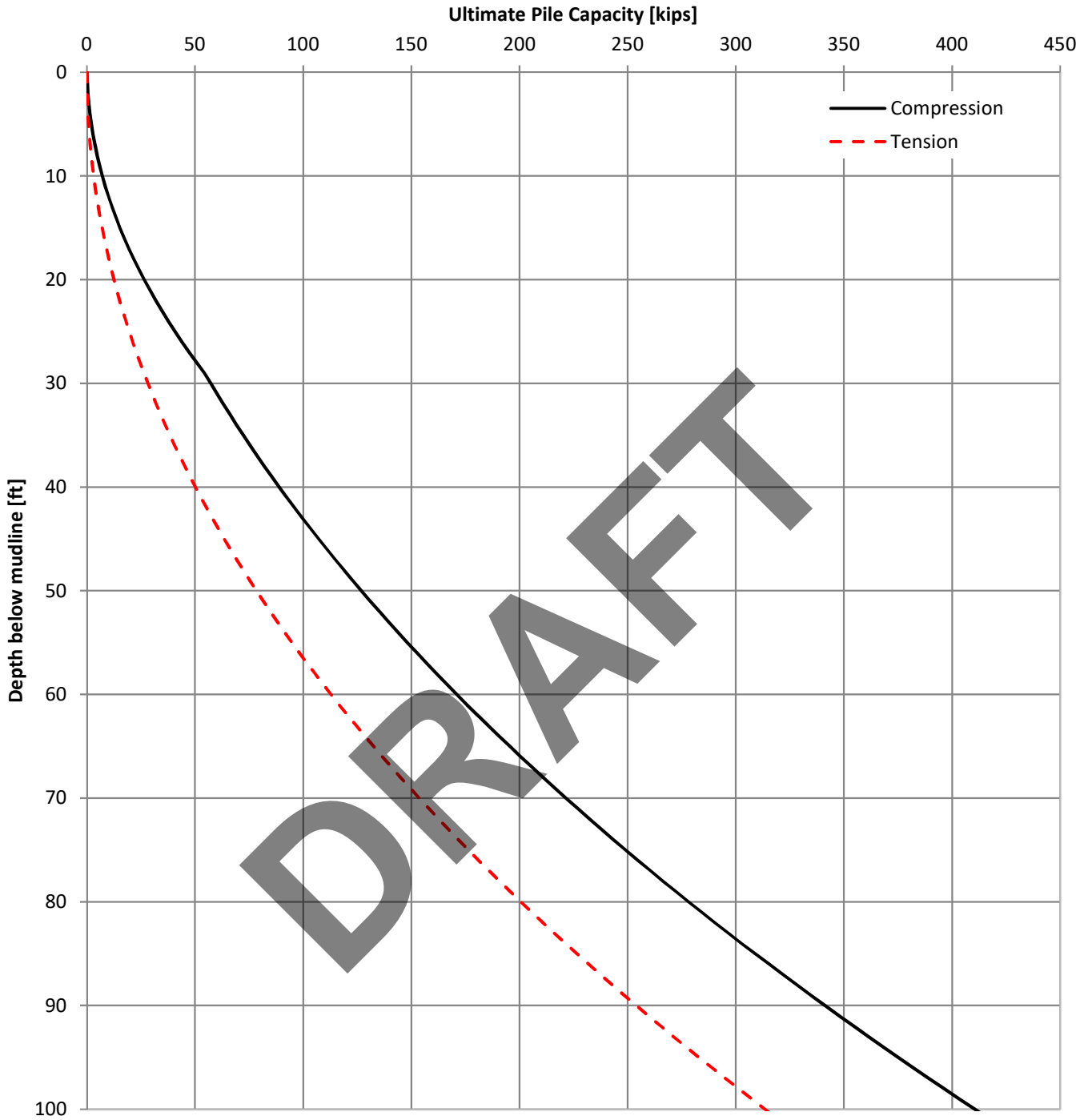


PILE DIAMETER: 12.75-IN PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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	SUBJECT ULTIMATE PILE CAPACITY 12.75-IN STEEL PIPE PILE, GALVESTON WEST SITE		
	PROJECT COASTAL TEXAS	DATE 8/3/2018	REVISION Draft
	CLIENT THE TEXAS GENERAL LAND OFFICE	BY CB / MJW	PLATE
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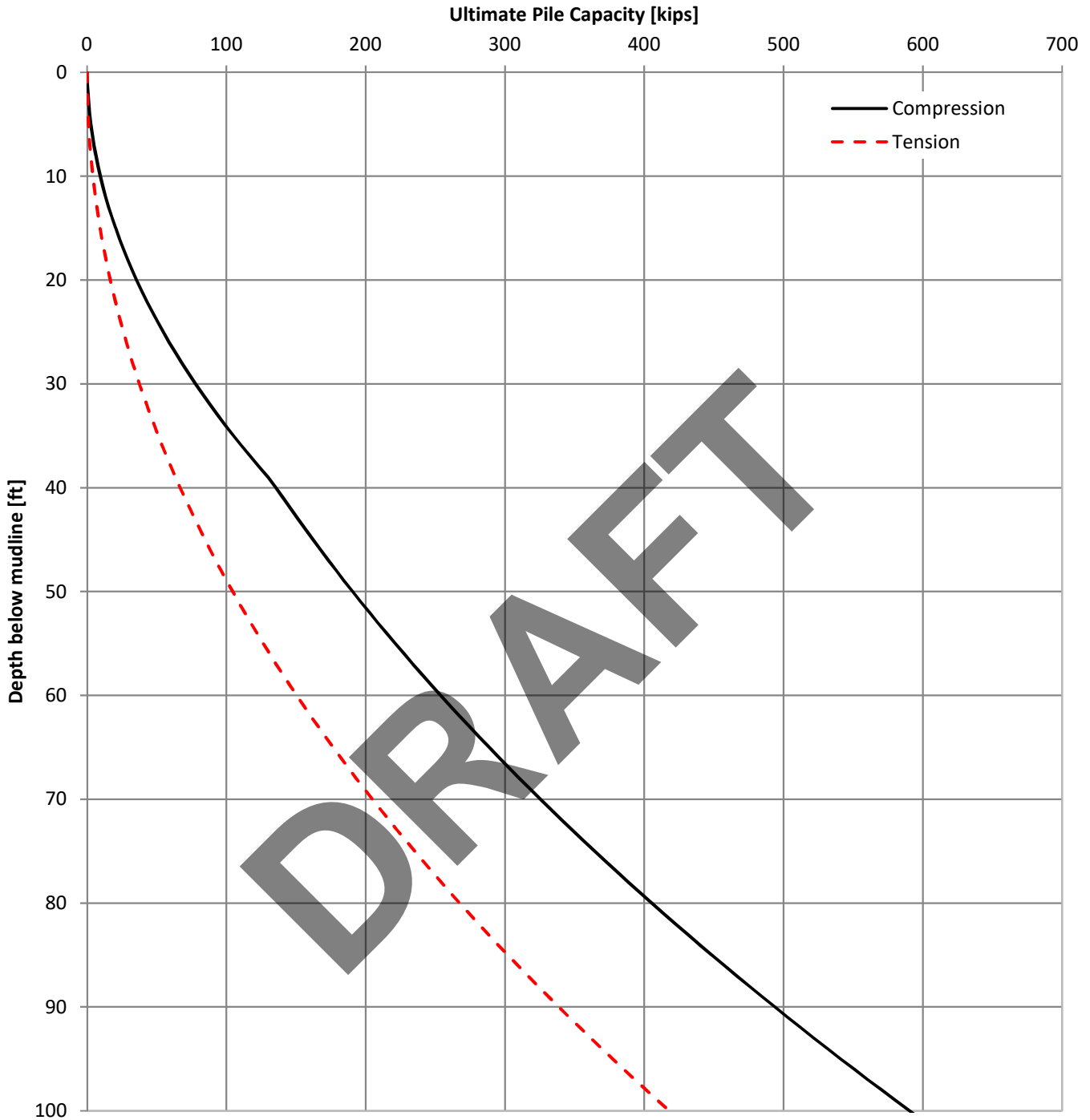
PILE DIAMETER: 18.00-IN

PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
2. PILE CAPACITIES CALCULATED PER API RP 2A.
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	SUBJECT ULTIMATE PILE CAPACITY 18-IN STEEL PIPE PILE, GALVESTON WEST SITE		
	PROJECT COASTAL TEXAS	DATE 8/3/2018	REVISION Draft
	CLIENT THE TEXAS GENERAL LAND OFFICE	BY CB / MJW	PLATE XX
		CHECK Draft	



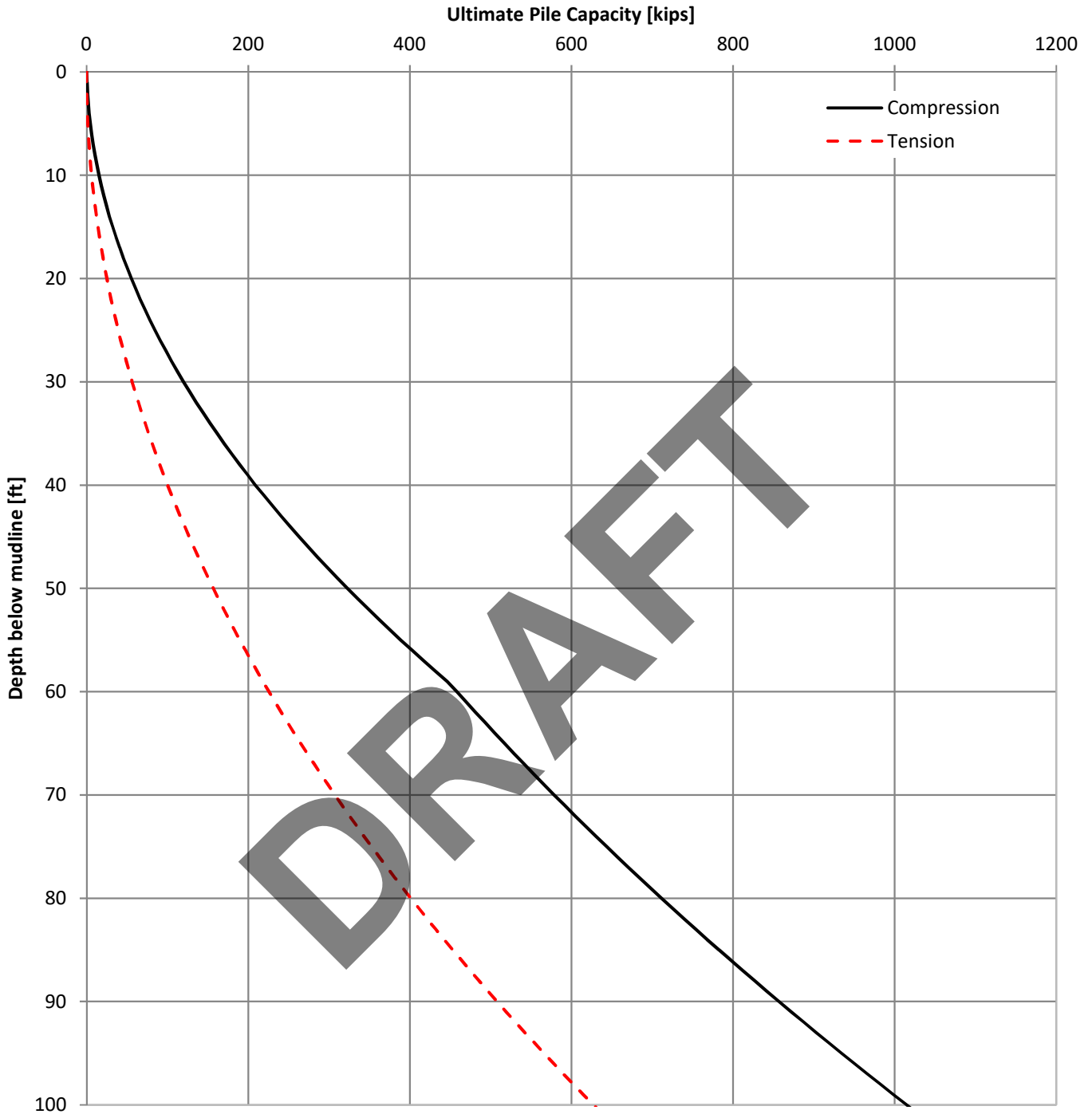
PILE DIAMETER: 24.00-IN

PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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	SUBJECT ULTIMATE PILE CAPACITY 24-IN STEEL PIPE PILE, GALVESTON WEST SITE		
	PROJECT COASTAL TEXAS	DATE 8/3/2018	REVISION Draft
	CLIENT THE TEXAS GENERAL LAND OFFICE	BY CB / MJW	PLATE
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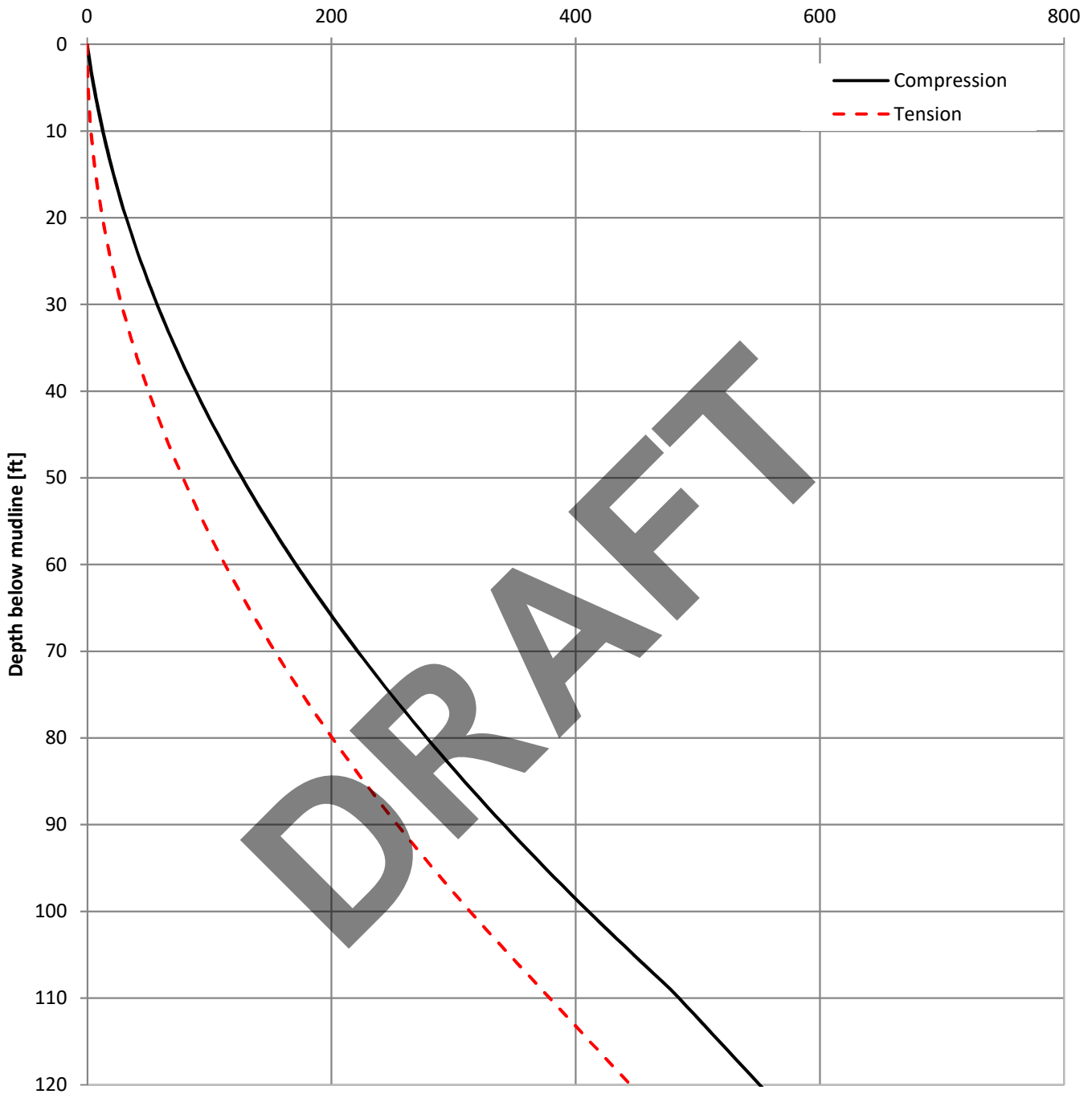
PILE DIAMETER: 36.00-IN PILE TYPE: STEEL PIPE PILE

NOTES:

1. REDUCE ULTIMATE PILE CAPACITY WITH SAFETY FACTOR (SF) TO OBTAIN ALLOWABLE AXIAL PILE LOAD. COMPRESSION SF = 2. TENSION: SF = 3.
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	SUBJECT ULTIMATE PILE CAPACITY 36-IN STEEL PIPE PILE, GALVESTON WEST SITE		
	PROJECT COASTAL TEXAS	DATE 8/3/2018	REVISION Draft
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Ultimate Pile Capacity [kips]



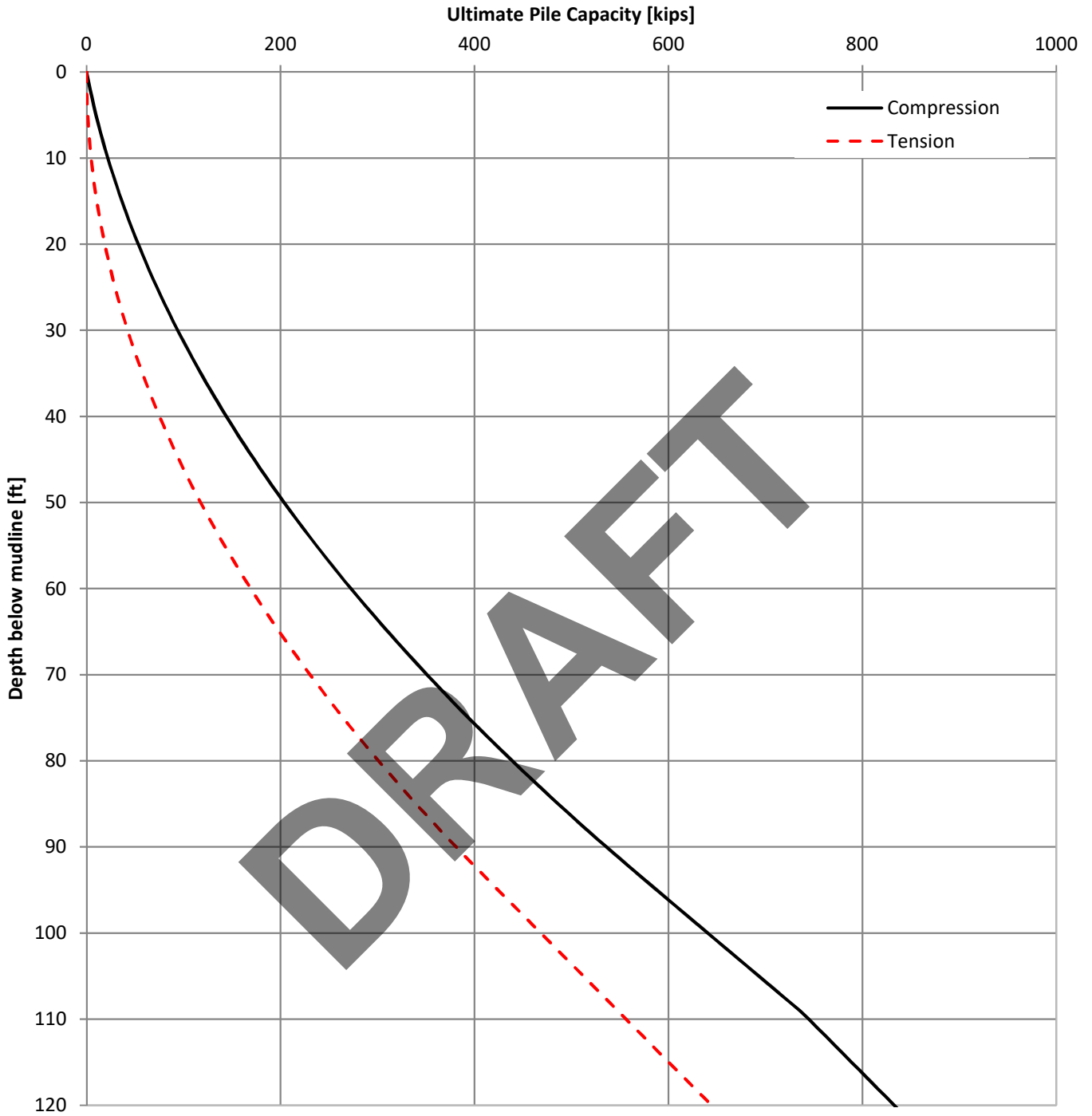
PILE DIAMETER: 18-IN

PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

NOTES:

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	SUBJECT ULTIMATE PILE CAPACITY 18-IN CONCRETE PILE, GALVESTON WEST SITE		
	PROJECT COASTAL TEXAS	DATE 8/3/2018	REVISION Draft
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PILE DIAMETER: 24-IN

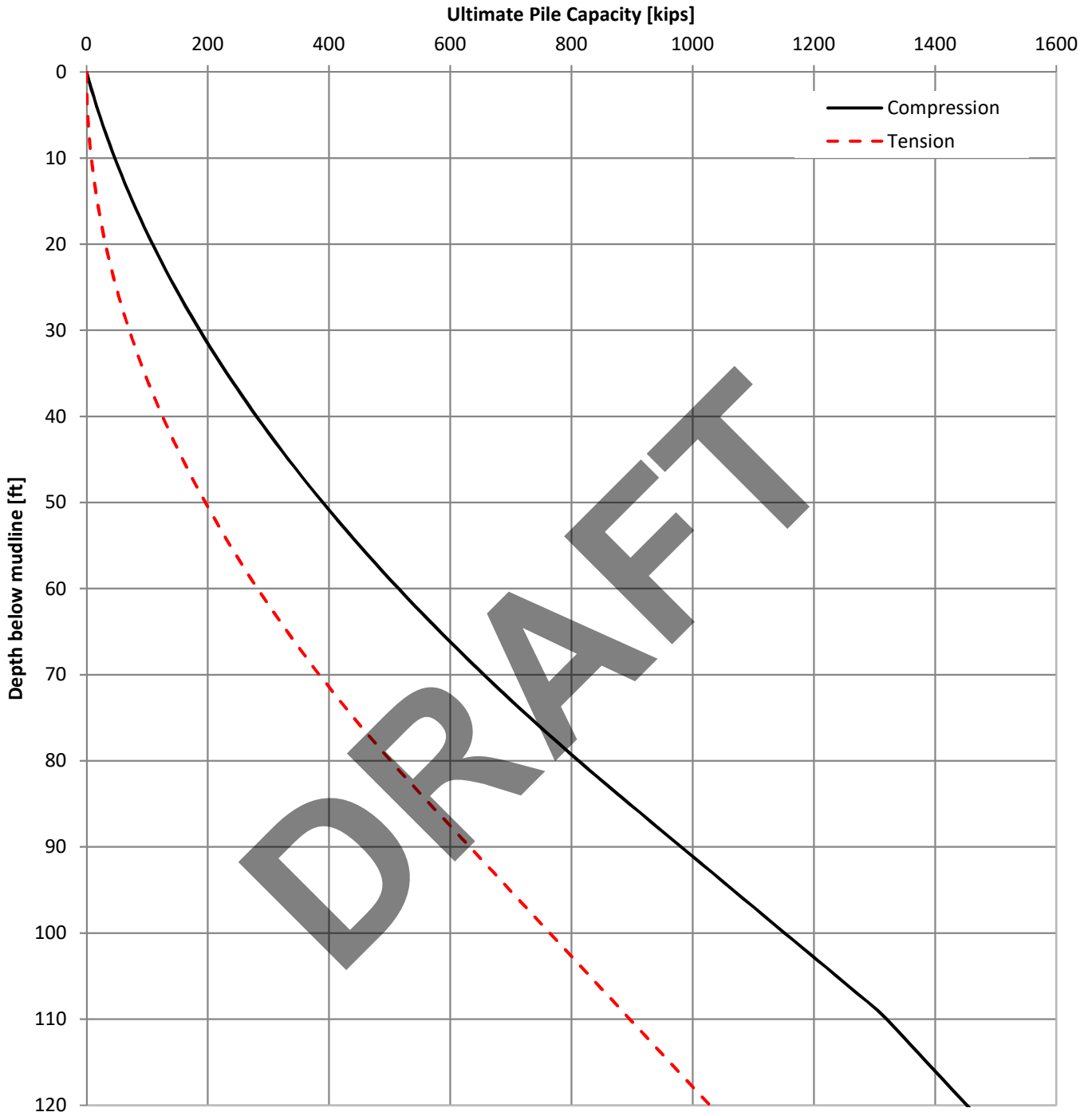
PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

NOTES:

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SUBJECT	ULTIMATE PILE CAPACITY 24-IN CONCRETE PILE, GALVESTON WEST SITE		
PROJECT	COASTAL TEXAS	DATE	8/3/2018
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PILE DIAMETER: 36-IN

PILE TYPE: PRE-CAST CONCRETE CIRCULAR PILE

NOTES:

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SUBJECT	ULTIMATE PILE CAPACITY 36-IN CONCRETE PILE, GALVESTON WEST SITE		
PROJECT	COASTAL TEXAS	DATE	8/3/2018
CLIENT	THE TEXAS GENERAL LAND OFFICE	BY	CB / MJW
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**Draft Lateral Pile Capacity
Memorandum**

October 24, 2018

DRAFT

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DRAFT

Draft Lateral Pile Capacity Memorandum

October 24, 2018

DRAFT

Issue and revision record

Revision	Date	Originator	Checker	Approver	Description
A	10/18/2018	M.J.Walker	C.Brodbaek, P.McLaughlin	J.Carter	Rev A

Document reference: 393582-C1

Information class: Standard

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1 Purpose

The Tentatively Selected Plan for the Texas Coastal Protection and Restoration project calls for construction a coastal flood barrier along portions of Galveston Island and Bolivar Peninsula. This barrier is being designed to reduce risk of inundation from storm surge. This memorandum presents the results of lateral pile design calculations and provides those results as recommendations for feasibility-level design. Pile lateral capacities have been developed using available historical geotechnical data at the following locations:

- Clear Creek
- Dickinson Bayou
- City of Galveston Pump Stations (East and West)

This memorandum uses the same stratigraphic information developed for a similar feasibility-level analysis of axial pile capacities, published under separate cover.

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2 Data Collection

2.1 Geotechnical Data

Mott MacDonald has compiled a GIS database of the available geotechnical information in the region of the proposed improvements. The historical geotechnical reports date from the 1950s through the early 2000s. Few cone penetration tests are available in the data, and soil sampling typically used standard penetration tests without hammer energy measurements. Some locations include geotechnical data to depths appropriate for design of foundations for the Coastal Texas project, but many are for shallow improvements such as roadways or low levees.

Geotechnical data to support foundation design would include boreholes with sampling at regular intervals or cone penetration tests extending below specified pile tip elevations.

2.1.1 Vertical Datums

The various sources use various datums for reference. Some refer to the National Geodetic Vertical Datum (NGVD29), North American Vertical Datum (NAVD88), or simply depth below mudline. During subsequent phases of the project, these datums should be reconciled.

2.1.2 Borings

The available data for four flood protection and pump station sites have been evaluated: Clear Creek, Dickinson Bayou, Galveston East, and Galveston West. The data available for each site are described in Table 1 below. See the references section of this memorandum for details regarding each report. Generally, only the Clear Creek and Dickinson geotechnical conditions are well characterized for the purposes of feasibility level lateral pile capacity estimates.

Table 1: Source Geotechnical Information for Site.

Site Name	Geotechnical Data	Reference
Clear Creek	Site-specific boreholes to 75 feet below onshore grade.	McBride-Ratcliff and Associates (1982)
Clear Creek	Site-specific boreholes to 90 feet below onshore grade.	McBride-Ratcliff and Associates (1985)
Clear Creek	Site-specific boreholes to -80 feet, NGVD. Stratigraphy and unconfined compression strength.	USACE (1987)
Dickinson Bayou	Boreholes to approximately 20 feet below grade located greater than 2,500 feet away. No SPT N values.	USACE (1962)
Dickinson Bayou	Site specific boreholes for SH146 Bridge over Dickinson Bayou.	Geotest Engineering (2000)
Galveston West	Site-specific boreholes to 27.5 feet depth below mudline.	USACE (1958)
Galveston East	Site-specific boreholes to 35 feet depth below nearby site grade.	Fugro South (2003)
Galveston East	Boreholes to 5 feet below nearby site grade approximately 1,500 feet to the south. Generally not relevant for this study.	McLellend Engineers (1961)

2.1.3 Soil Samples

The soil sampling documented in the historical geotechnical reports comprises standard penetration tests (SPT), which yield generally disturbed samples not appropriate for advanced laboratory testing and strength characterization. For the Clear Creek site, the USACE (1987) work included unconfined compression test profiles. This profile indicates a relatively weak (unconfined compressive strength values between 400 and 800 psf), near-surface clay layer overlying stiff to very stiff clays (unconfined compressive strength values between 1,200 and 3,000 psf).

For locations with geotechnical investigation information that did not reach sufficient depth, the conditions documented in the available data were extended for the analysis. For instance, the generally sandy profiles at Galveston east and west sites are extrapolated from data extending only to about 35 feet. Data at Clear Creek and Dickinson Bayou extend to approximately 100 feet.

2.2 Bathymetry and Topography

Generally, the available geotechnical data were collected referencing the onshore ground surface, NGVD29 vertical datum, or the mudline. A generalized stratigraphy has been developed at each location for depth below surface grade or mudline and does not consider depth of water above.

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3 Geotechnical Design Parameter Development

Soil profiles have been developed for axial capacity evaluations. Inputs to the pile capacity calculations include soil strength (either undrained shear strength, s_u , or friction angle, ϕ), unit weight (γ), and lateral soil stiffness parameters. For this feasibility level evaluation, the suggested values provided by ENSOFT in the Technical Manual for LPile 2015 (2015) have been used with adjustments based on soil type and strength results. With additional geotechnical investigation, including cone penetration tests, it may be possible to refine these values.

At the Clear Creek site, the available historical geotechnical information is sufficiently detailed to support a refinement of a typically clay profile with a sand layer. Dickinson has a similar clay profile with a sandy soil layer. The local available data show the sand layer to be dense to very dense and of sufficient thickness to provide a bearing layer for axial pile capacity. This has been incorporated in the Clear Creek soil model for lateral pile analysis.

At the Galveston Pump Station sites (East and West), the available geotechnical information extends to a maximum depth of 35 feet below grade, and shows a profile comprising silty sand. To derive axial pile capacity values beyond that depth, the stratum was assumed to extend to 100 feet. This model has been applied to the lateral pile analysis. Only with further geotechnical data (collected in later project phases or identifying other historical sources) can this estimate be refined.

The resulting lateral pile capacity (shear), moments developed in the piles, and resulting displacements assume the load is applied at the pile head and the pile head is at the soil surface. Thus, if the piles are immersed or extend above the mudline to support the superstructure, the cantilevered rotation and displacement will be larger. No scour has been incorporated into the soil model. These results should be used solely to develop feasibility level design and concept verification. The capacities presented are “ultimate” and should be factored down by a Factor of Safety in accordance with the Hurricane and Storm Damage Risk Reduction System Design Guidelines (HSDRRS, USACE 2012), or other governing design criteria as appropriate. For groups of piles, the use of ENSOFT GROUP can be used with the soil parameters presented in the tables below. GROUP would be used to identify the reductions in pile lateral capacities for shadowing effects caused by rows of piles.

Pile capacities have been developed for the pile types identified in Table 2. The stratigraphic models are described in subsequent sections. The results of the analyses are attached to this memorandum for these piles pushed in free head conditions to 0.25 inches, 0.5 inches, 1.0 inches, and 2.0 inches.

Table 2: Pile Types Used for Lateral Pile Calculations

Pile Types	Structural Properties	Pile Sizes
12-inch dia. steel pipe	E = 30,000ksi, F _y = 50ksi	12.75-inch OD, 3/8-inch thickness
24-inch dia. steel pipe	E = 30,000ksi, F _y = 50ksi	24-inch OD (nominal) 1/2-inch thickness
24-inch dia. concrete	F _c = 4,000psi, 2% steel, 3-inches concrete cover	24-inch outside diameter, round

Note: All pile lengths were assumed 80 feet for this analysis. Actual lengths to be determined by structural engineer.

3.1 Clear Creek

The Clear Creek soil profile is described in Table 3 below. The soil profile has been developed from information found in McBride-Ratcliff (1982), McBride-Ratcliff (1985), and USACE (1987).

Table 3: L-Pile Soil Profile for Clear Creek Site

Layer Depth (ft)	Soil Type	Su (psf) ¹	ϕ (°)	γ' (pcf) ²	L-Pile Soil Type	k (pci)	ϵ_{50}
0-45	Soft Clay	0.22* σ_{v0}' (225)	--	45	Stiff Clay	300	0.02
45-59	Stiff Clay	1500	--	45	Stiff Clay	500	0.02
59-100	Very Dense Sand	--	36	65	Dense Sand Below GWT	50	

Notes:

¹For a shear strength varying with depth, an average value was calculated using $(\sigma_{v0}')*(\gamma')$ at the midpoint of the layer. The average value is reported in parentheses.

² Effective Unit Weight is denoted by γ'

3.2 Dickinson Bayou

The Dickinson Bayou soil profile is described in Table 4 below. The soil profile has been developed from geotechnical borehole logs completed for the design of the State Highway 146 Bridge over Dickinson Bayou dated February and March 2000, by Geotest Engineering, Inc. The logs, profile, and location portions of the report were provided on August 10, 2018, as the result of an information request made to the Texas Department of Transportation.

Table 4: Interpreted Soil Profile for Dickinson Site

Layer Depth (ft)	Soil Type	Su (psf) ¹	ϕ (°)	γ' (pcf) ²	L-Pile Soil Type	k (pci)	ϵ_{50}
0-20	Soft Clay	250	--	45	Stiff Clay	250	0.02
20-65	Stiff Clay	0.242* σ_{v0}' (460)	--	45	Stiff Clay	400	0.02
65-70	Very Dense Sand	--	36	65	Dense Sand Below GWT	50	
75-85	Very Dense Sand	--	36	65	Dense Sand Below GWT	50	
85-90	Very Dense Sand	--	36	65	Dense Sand Below GWT	50	
90-100	Stiff Clay	0.22* σ_{v0}' (1050)	--	45	Stiff Clay	500	0.01

Notes:

¹For a shear strength varying with depth, an average value was calculated using $(\sigma_{v0}')*(\gamma')$ at the midpoint of the layer. The average value is reported in parentheses.

² Effective Unit Weight is denoted by γ'

3.3 Galveston Pump Station Locations

The Galveston Island soil profile is described in Table 5 below. The soil profile has been developed from geotechnical borehole logs by McLellend Engineers (1961) and Fugro South (2003). The references do not include data deeper than 35 feet below site grades at the time of investigation, so the soil conditions have been extrapolated to depths. Both references indicate a relatively sandy profile. The assumed soil profile must be validated by site-specific geotechnical investigation, which should extend to depths beyond estimated pile toe elevations.

Table 5: Interpreted Soil Profile for Galveston Sites

Layer Depth (ft)	Soil Type	Su (psf)	ϕ (°)	γ^1 (pcf) ²	LPile Soil Type	k (pci)	ϵ_{50}
0-100	Loose- to Medium-Dense Silty Sand	--	30	55	Medium Dense Submerged Sand	20	--

Notes:
¹ Effective Unit Weight is denoted by γ^1

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4 Conclusions

Lateral pile analyses have been developed from available geotechnical data for the planned sites in Galveston County, Texas.

For the Clear Lake Pump Station site and Dickinson Bayou site, sufficient geotechnical data are available to develop stratigraphy that would support a concept level design estimate of pile lateral capacities. At Galveston sites the available data are more sparse, and actual ground conditions may vary considerable once detailed soils investigations are performed.

Steel pipe piles may require additional thickness for corrosivity in saline environments. The lateral pile capacities provided have considered nominal wall thicknesses only, and are not reduced for corrosion section loss.

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5 References

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A. Lateral Pile Analysis Results

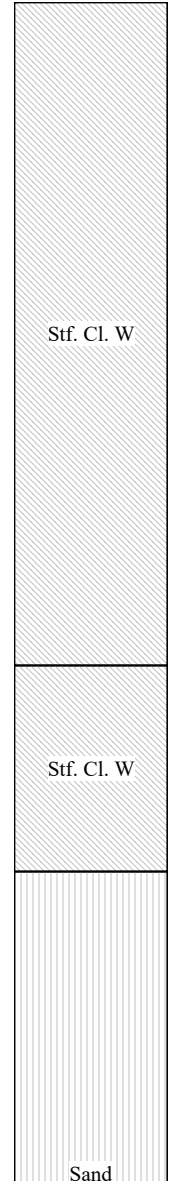
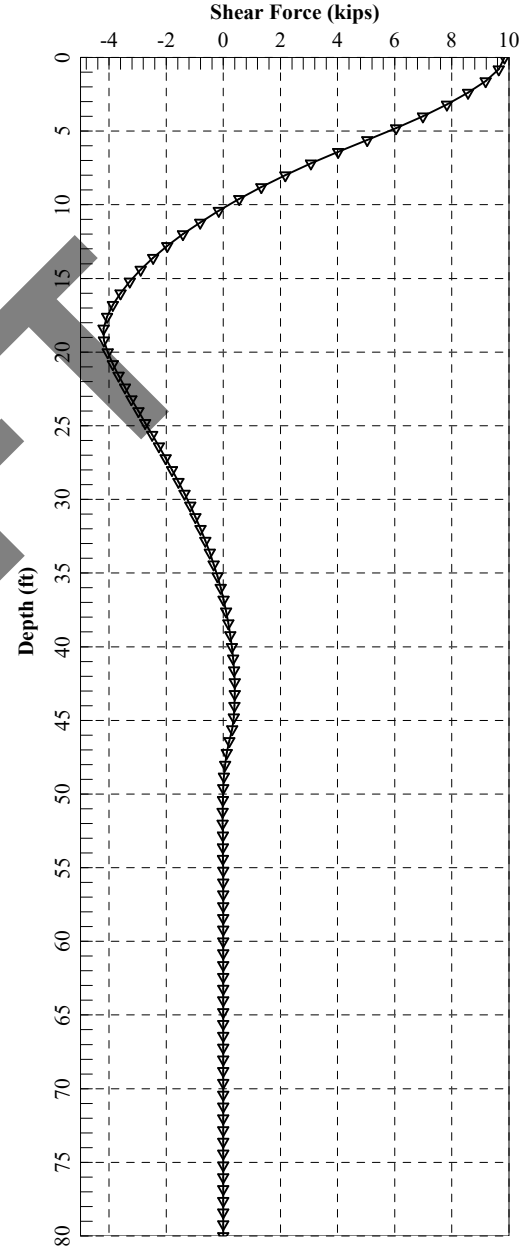
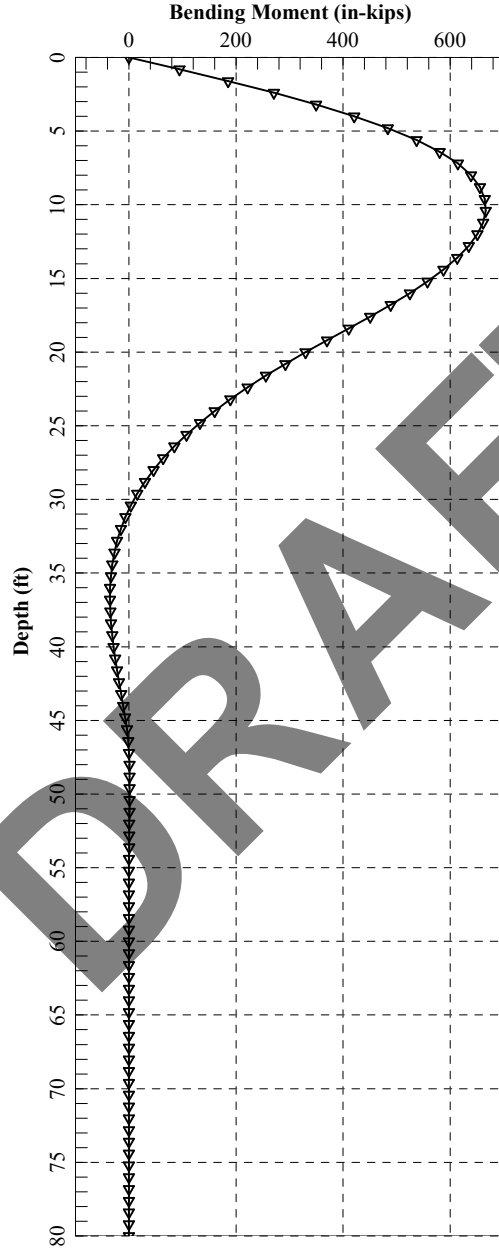
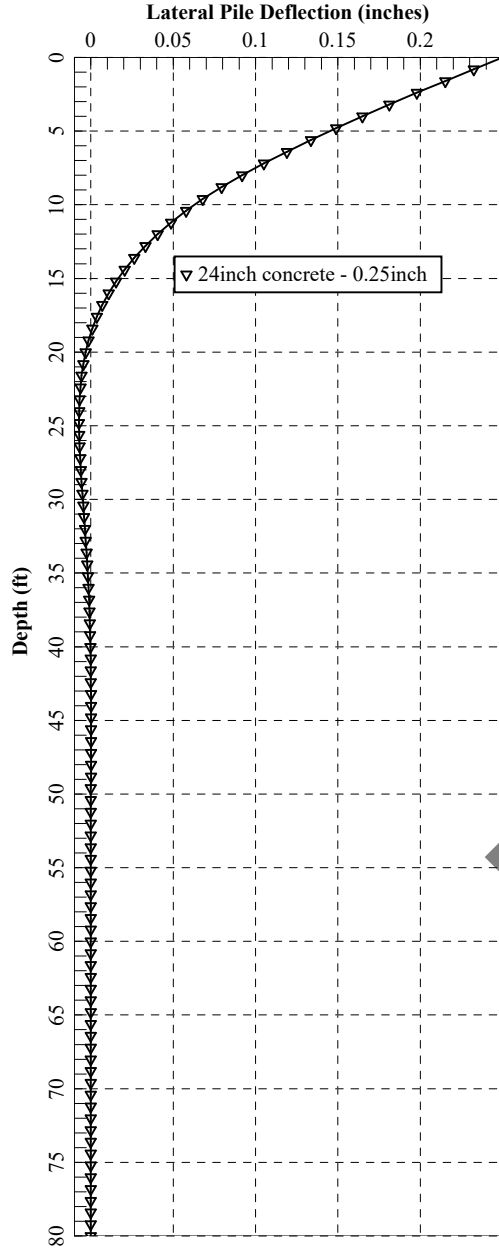
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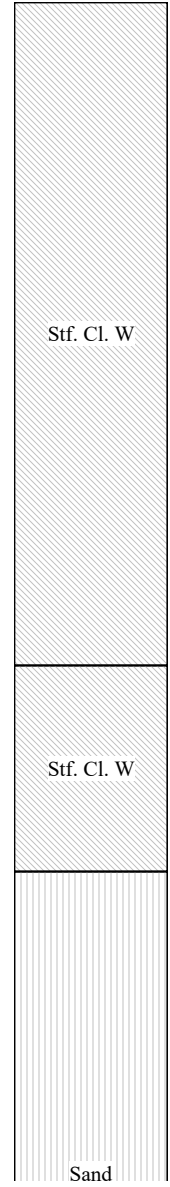
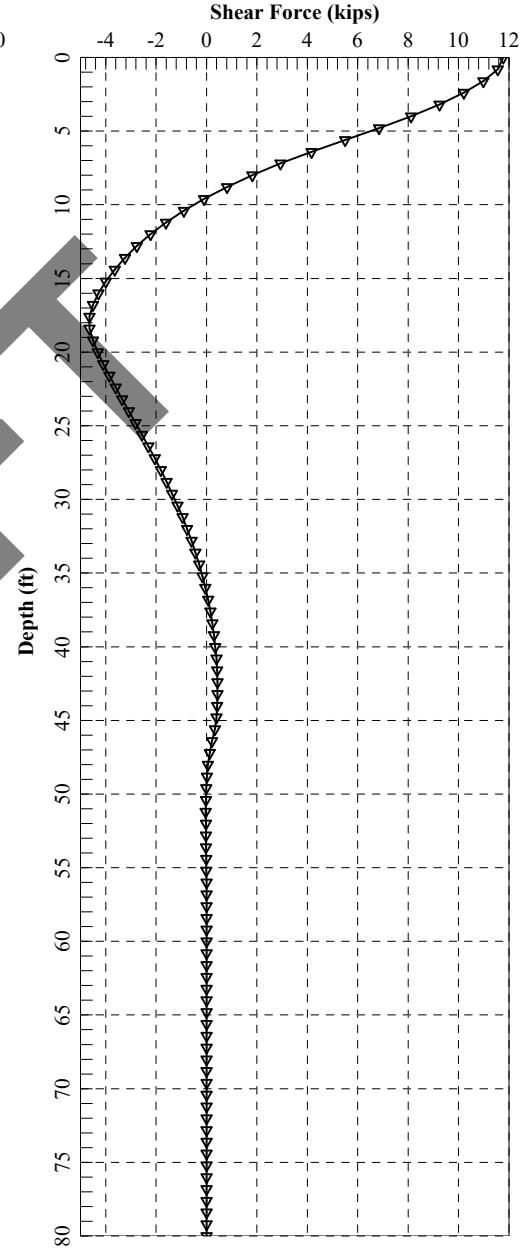
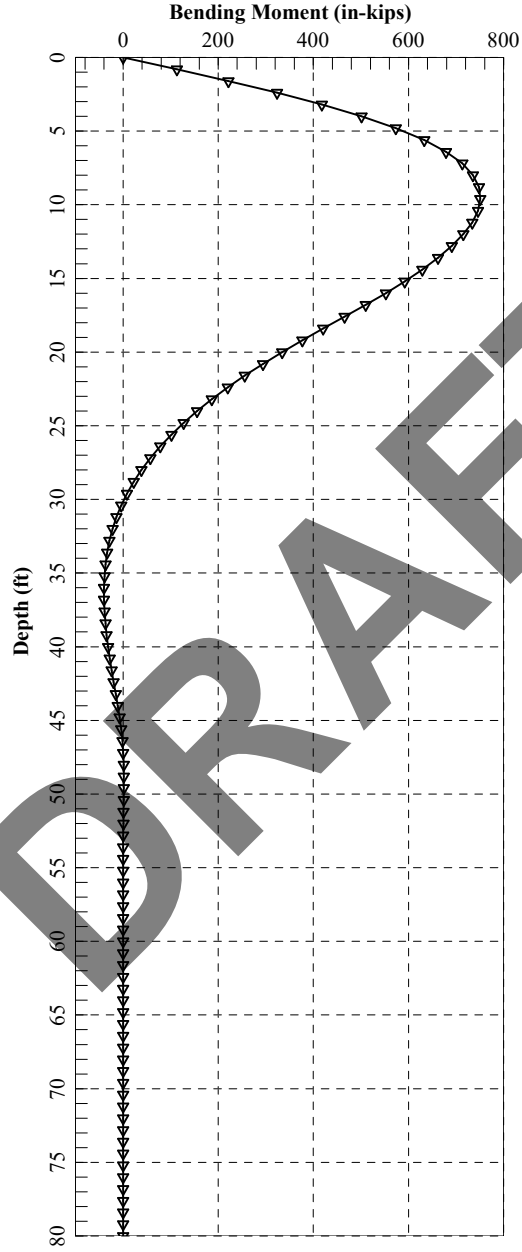
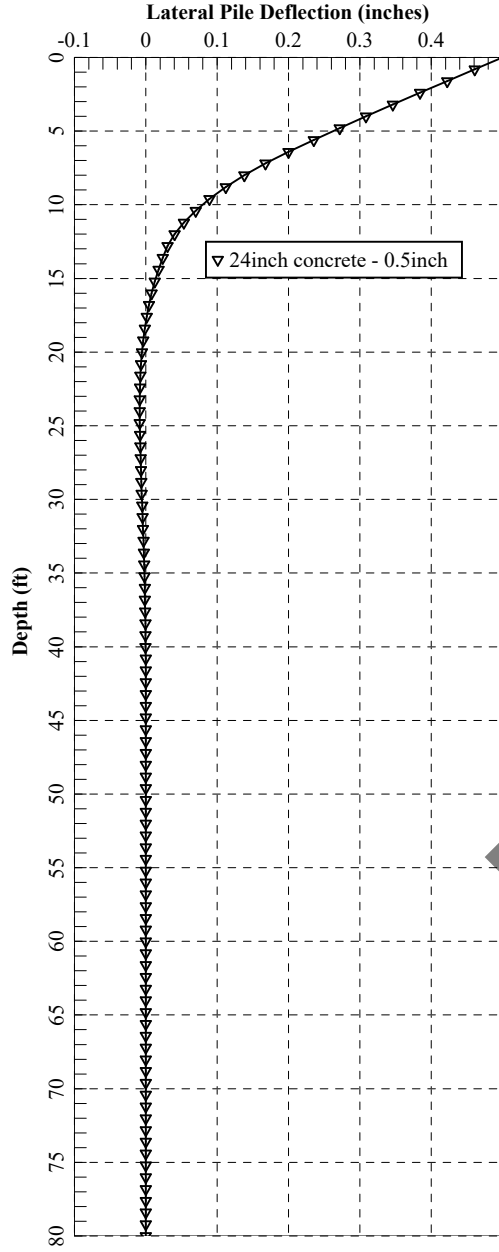
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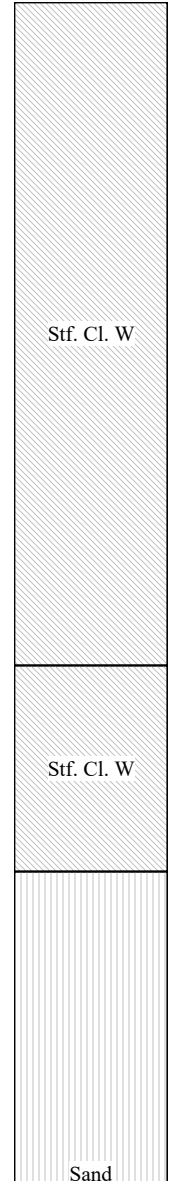
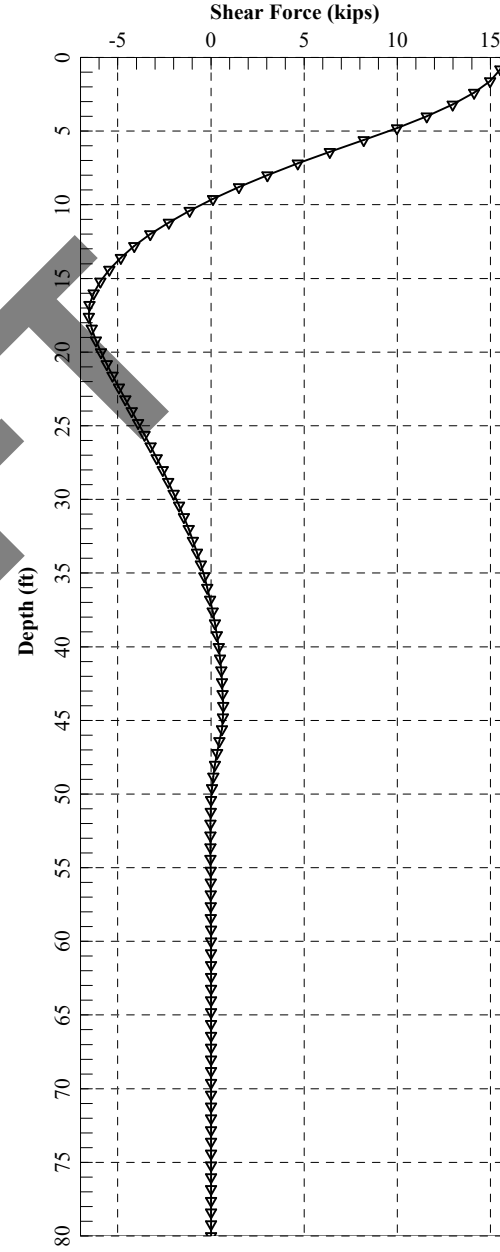
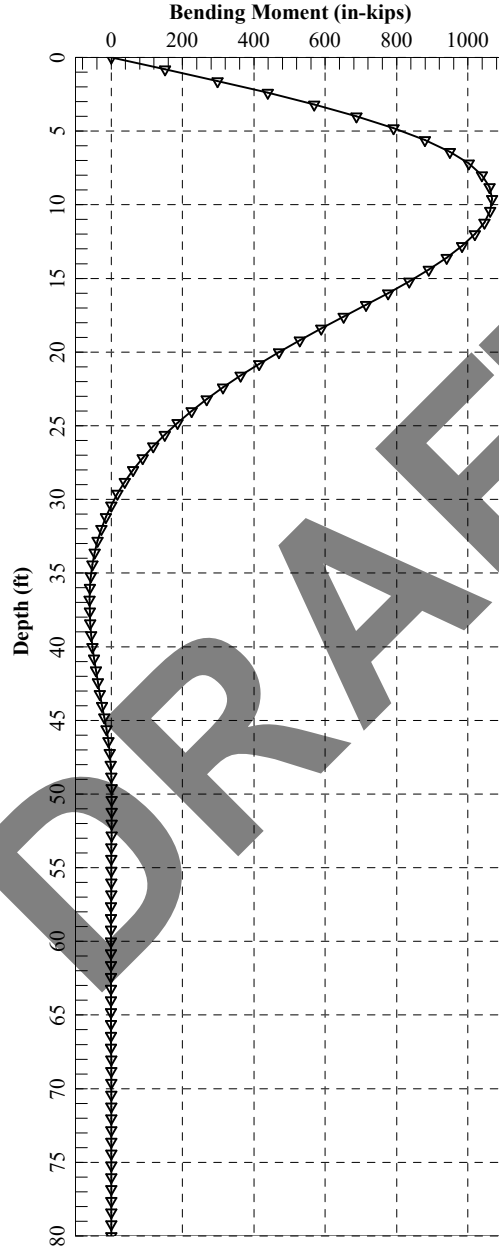
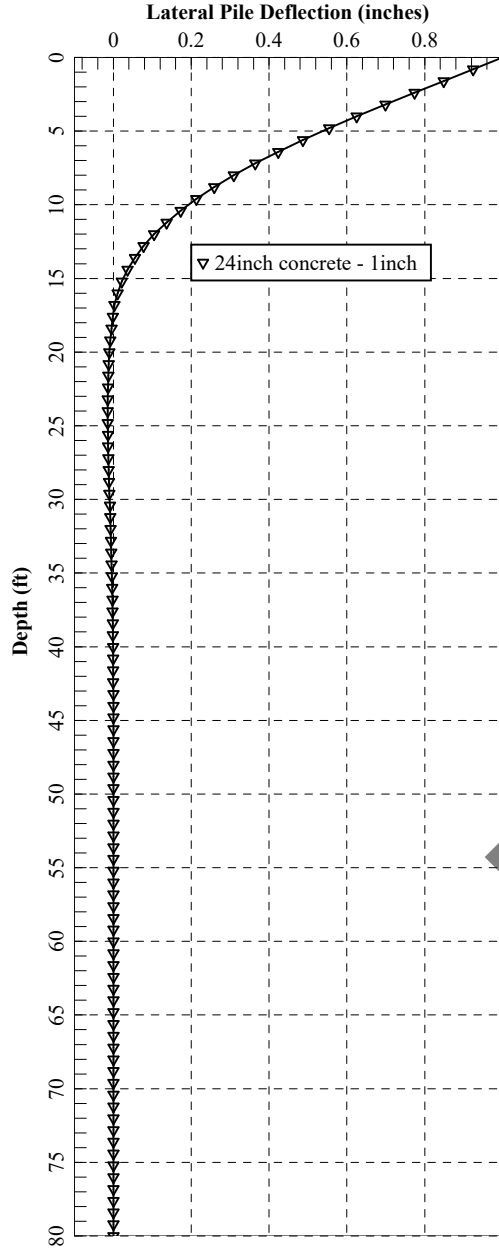
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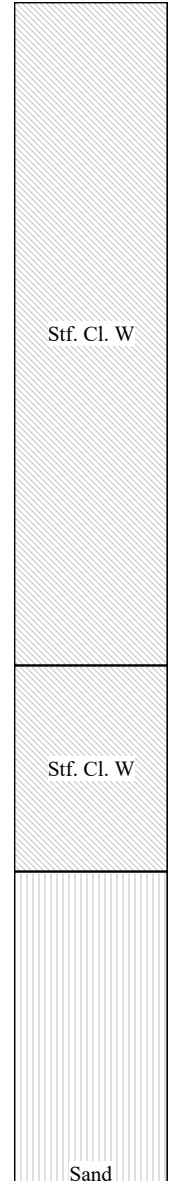
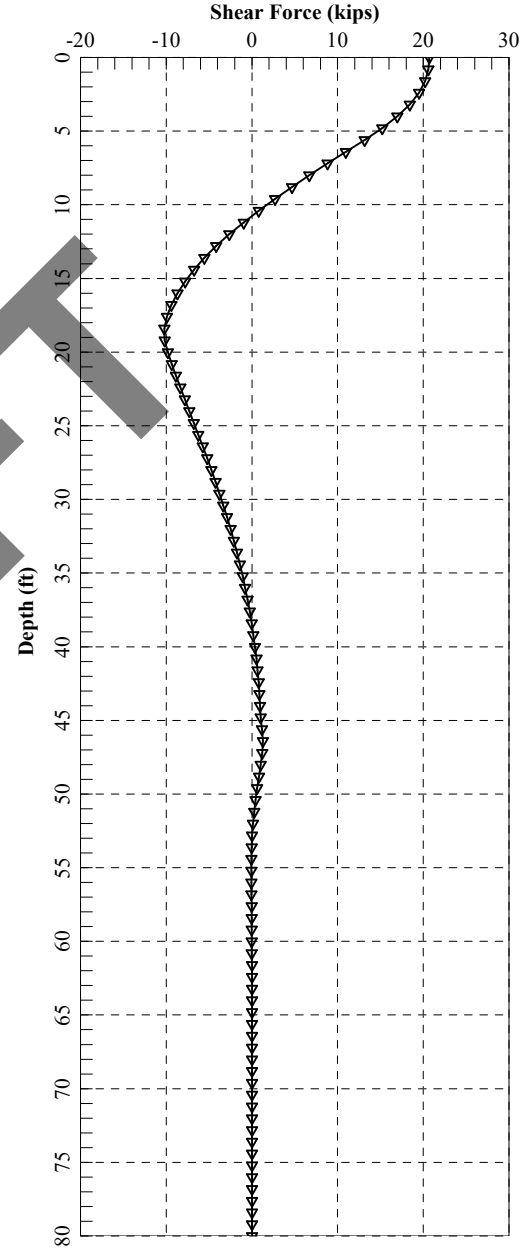
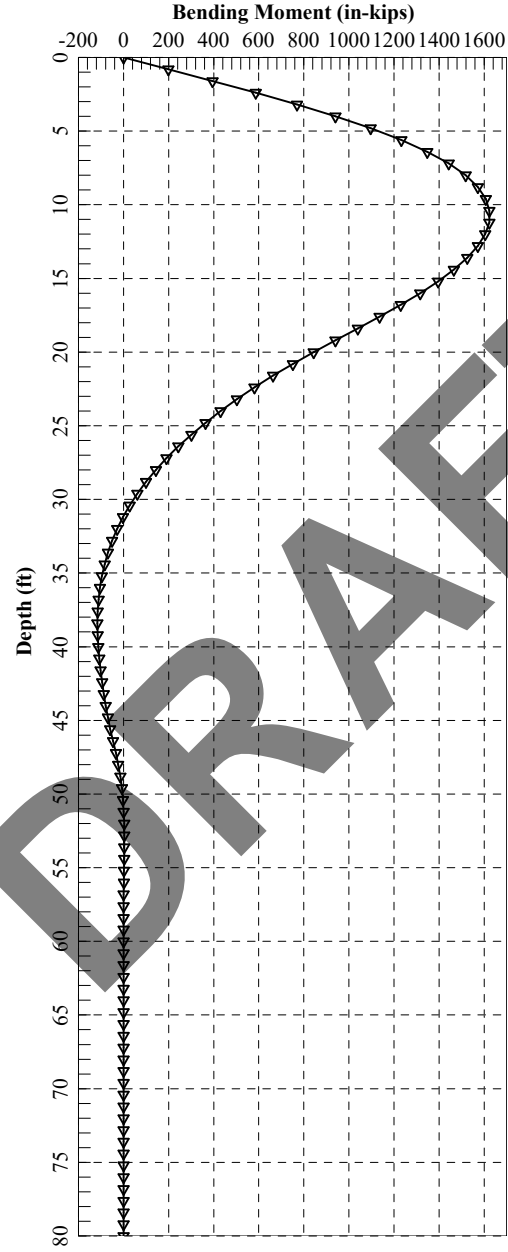
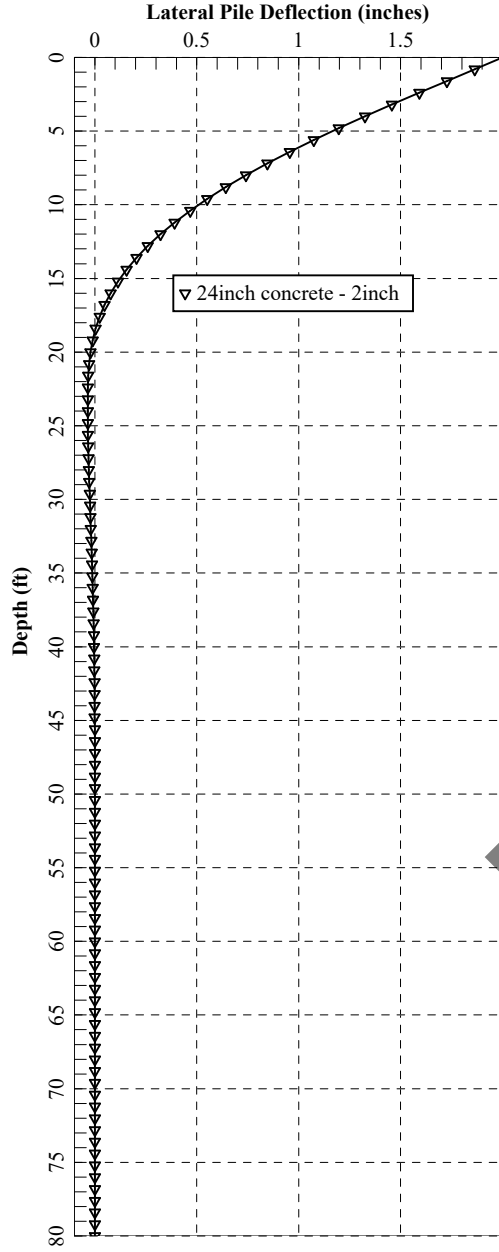
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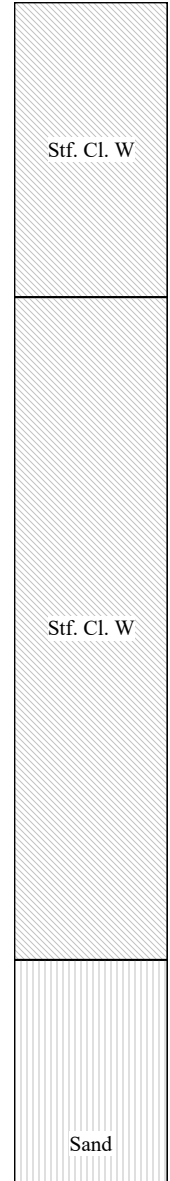
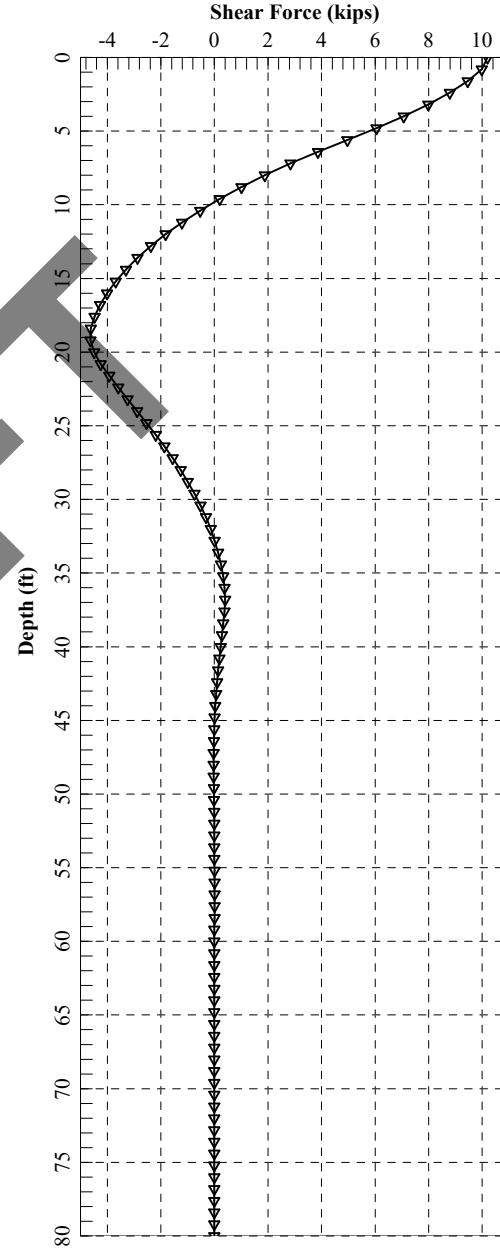
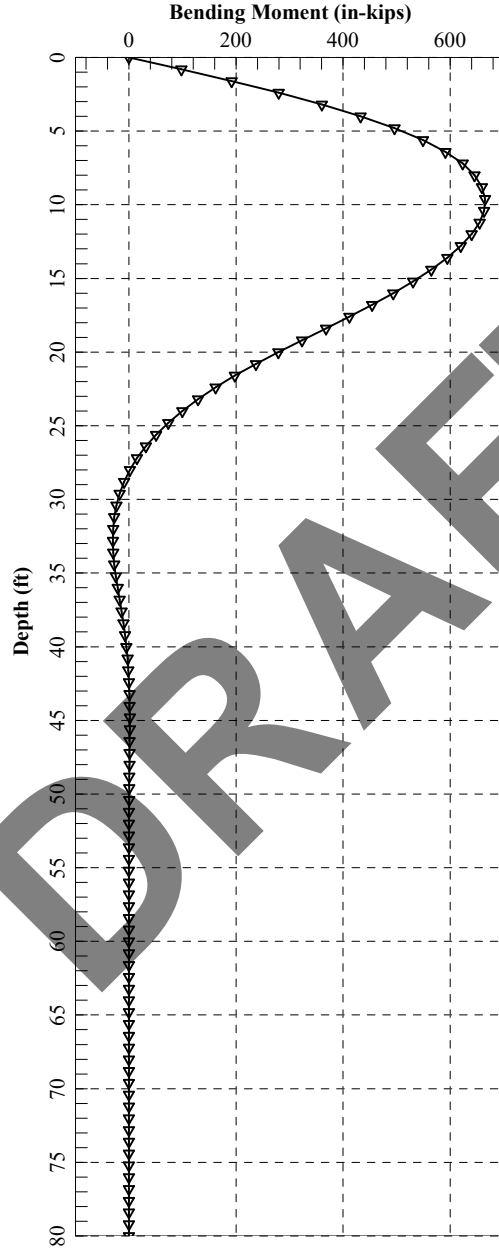
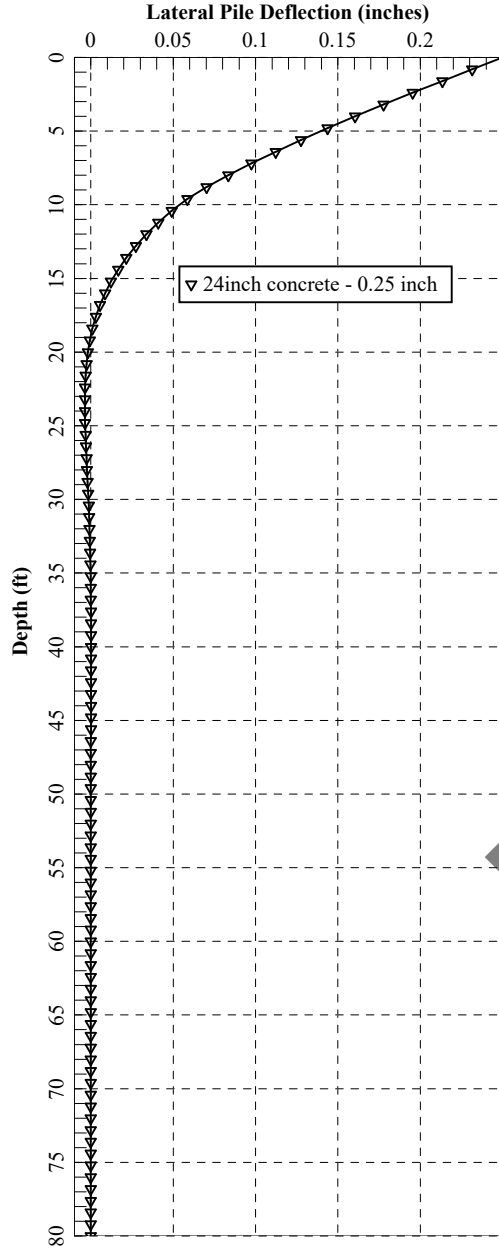


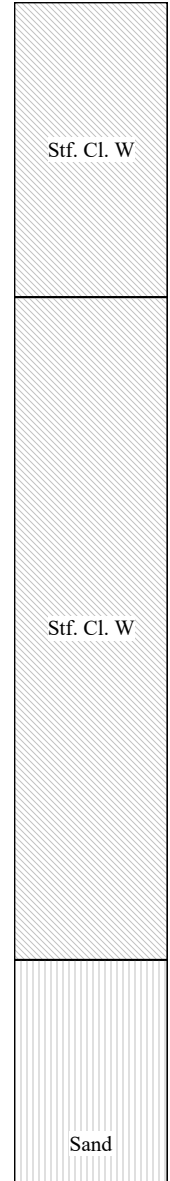
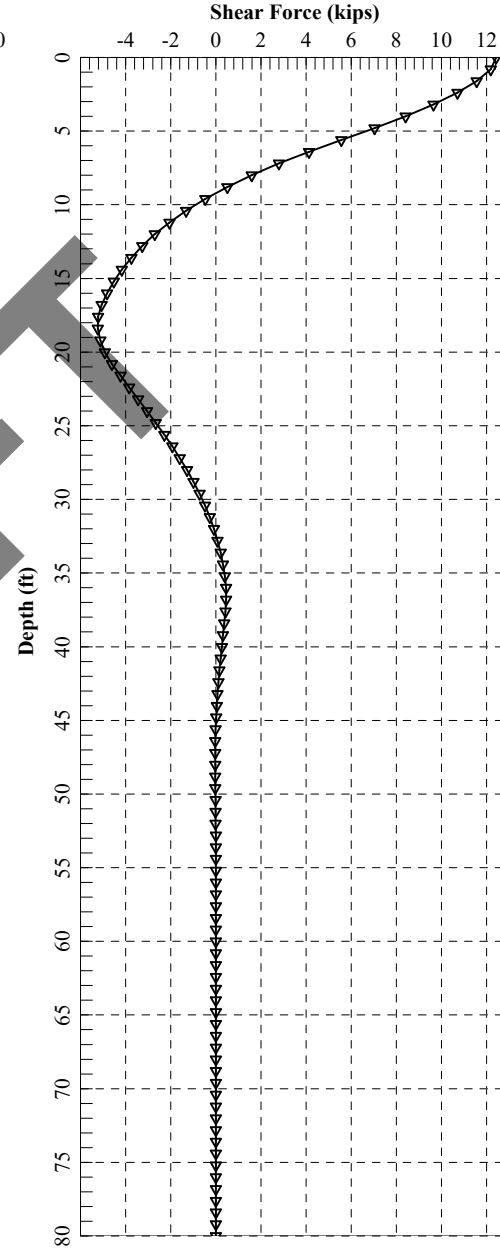
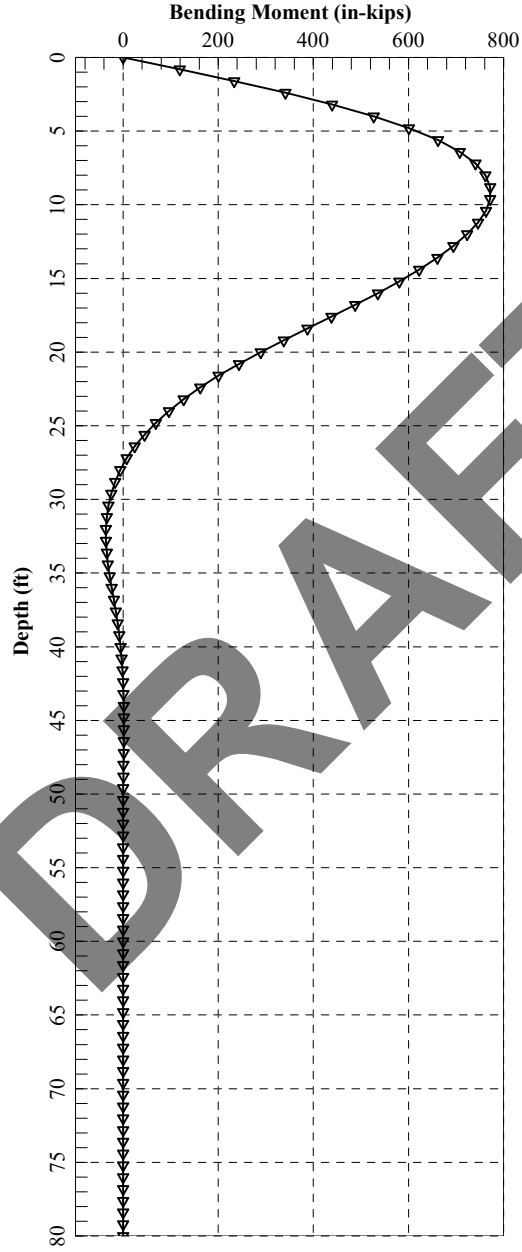
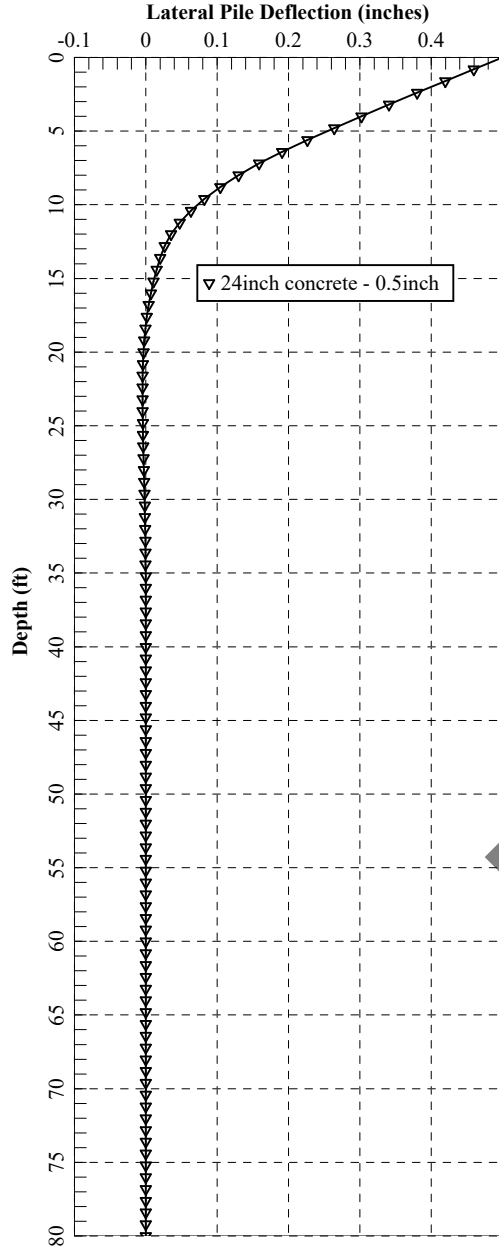
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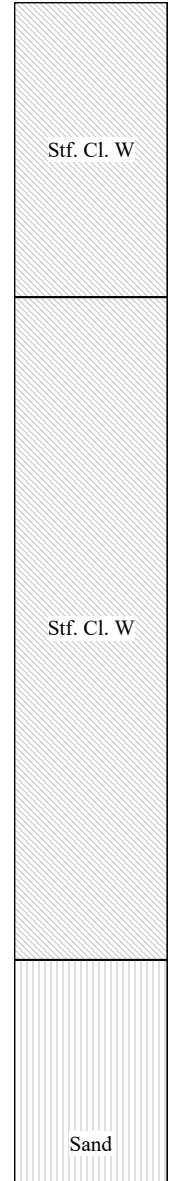
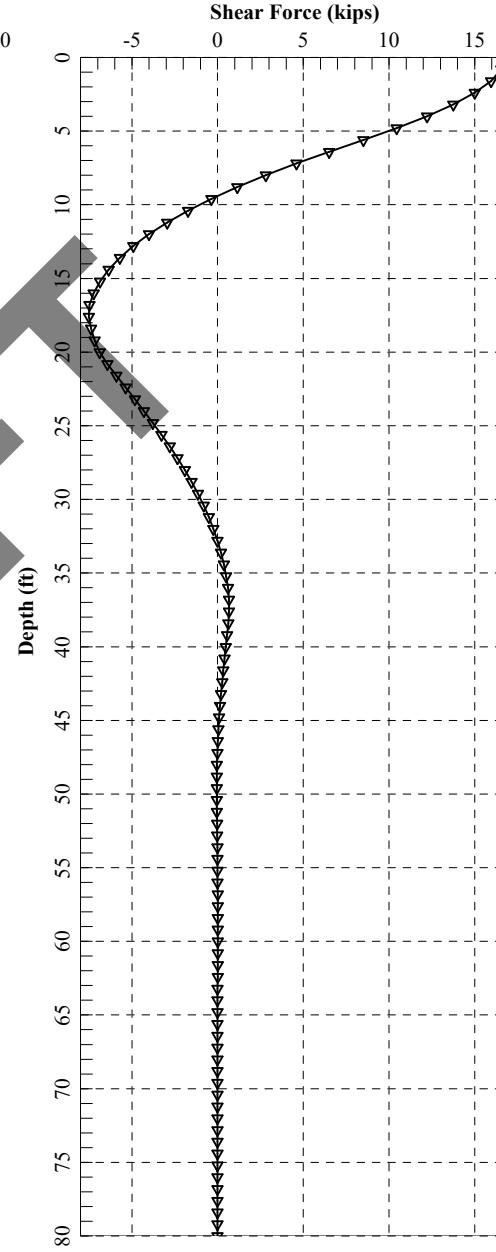
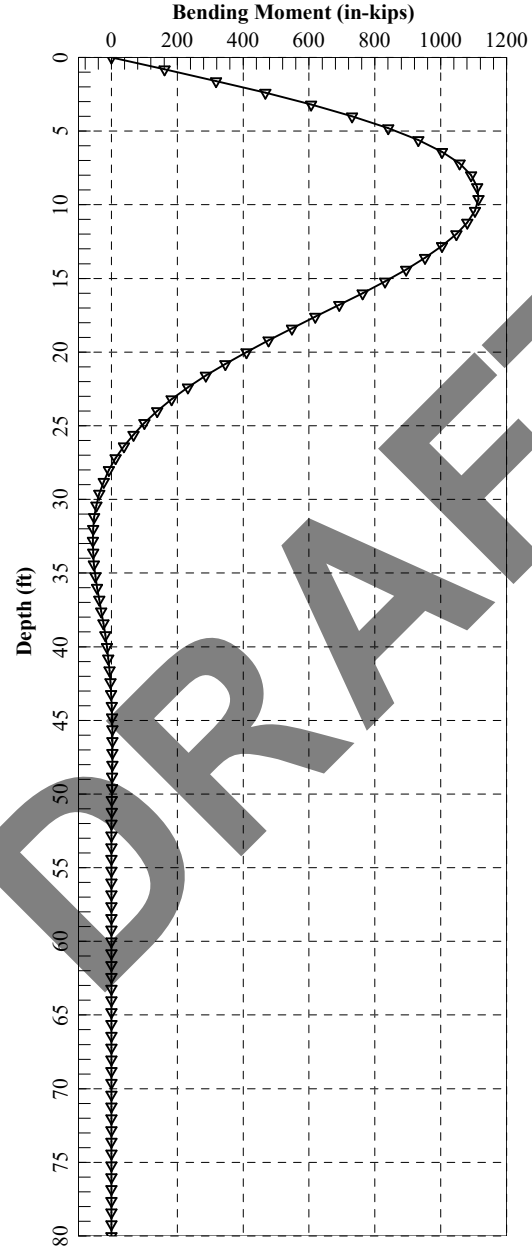
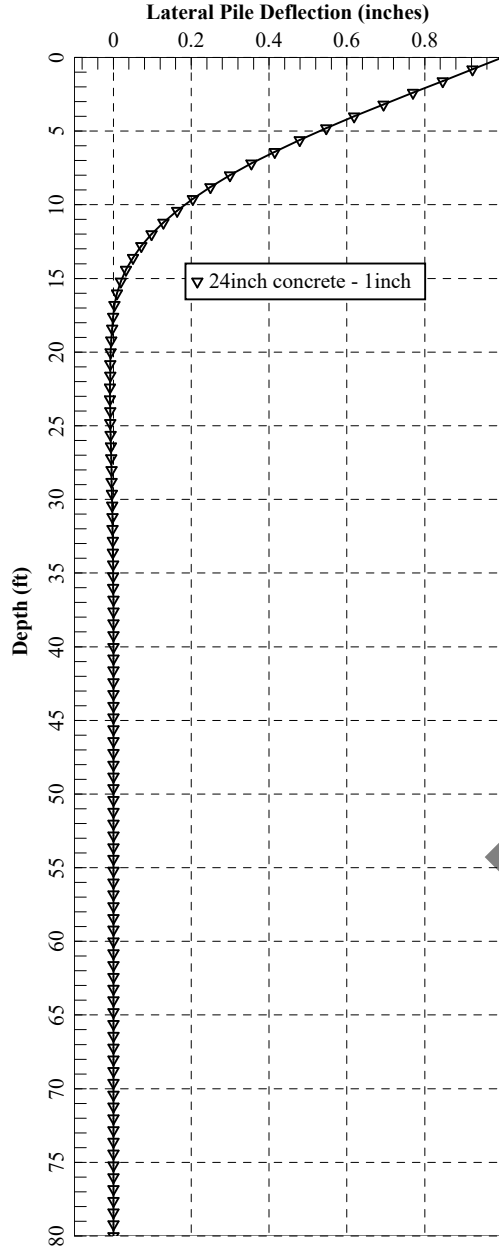
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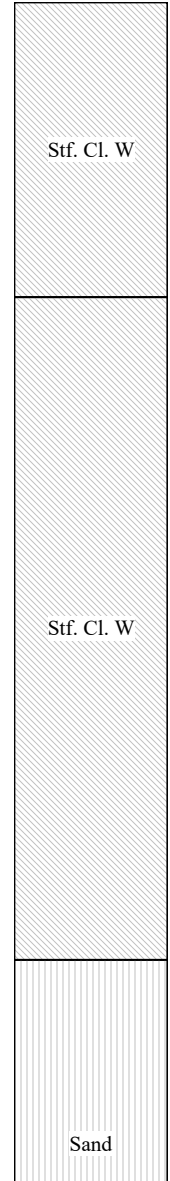
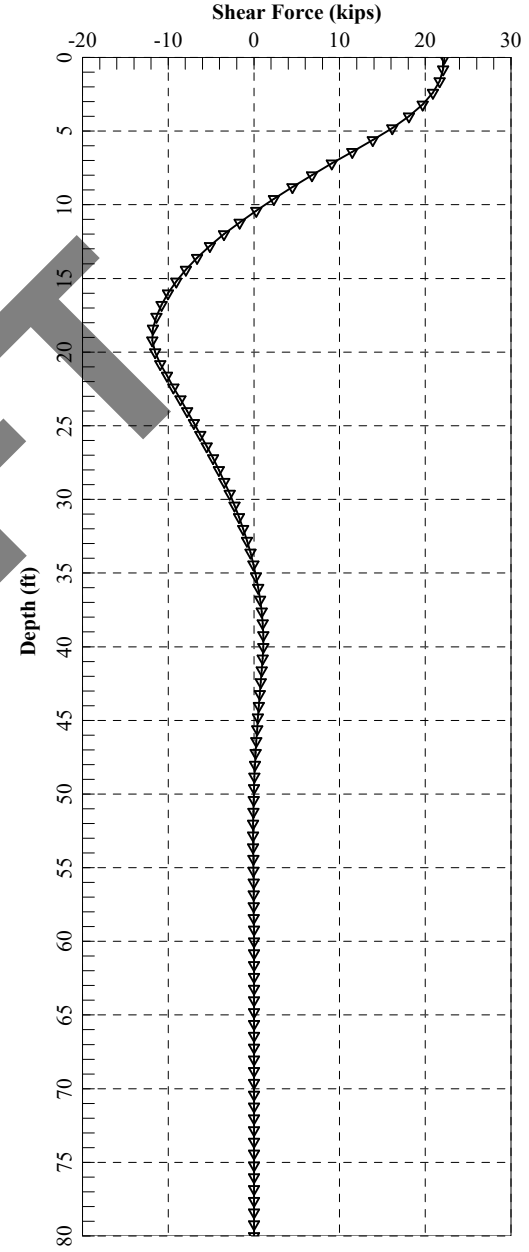
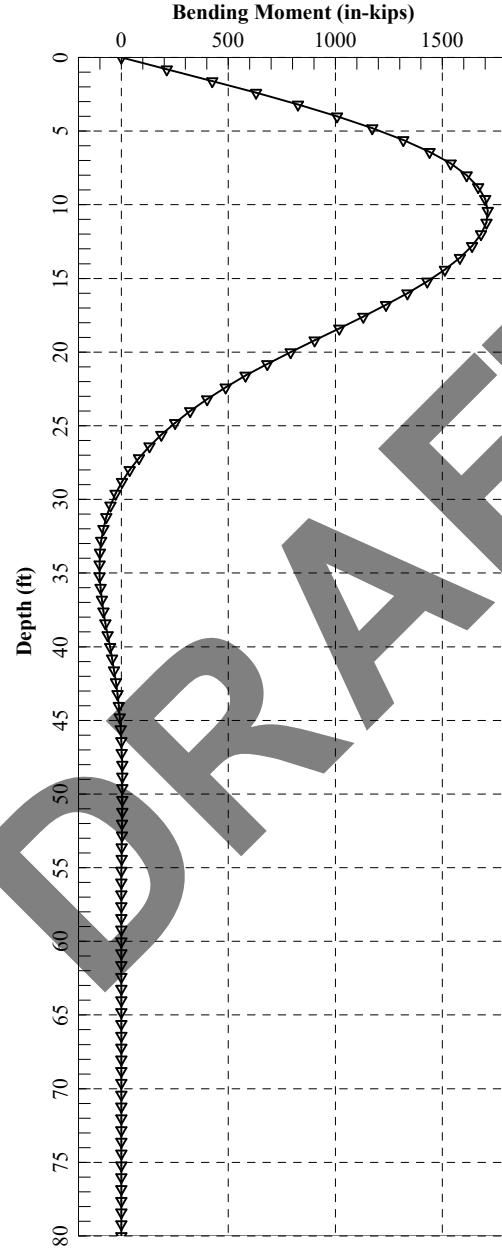
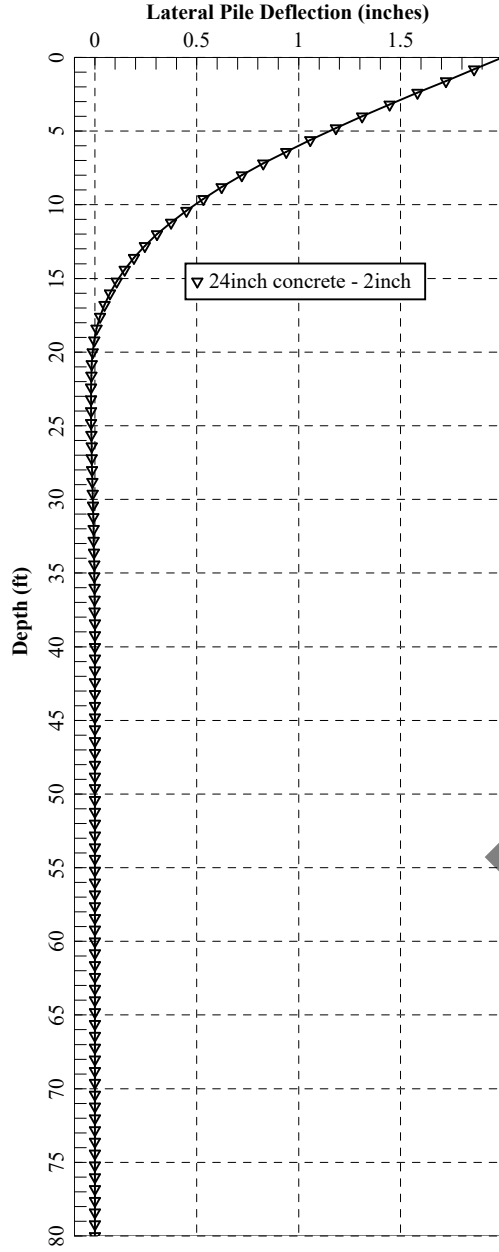
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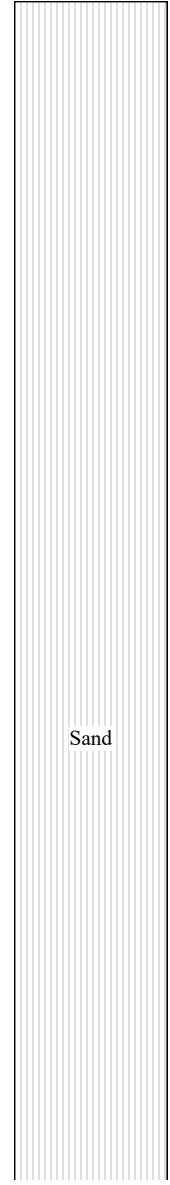
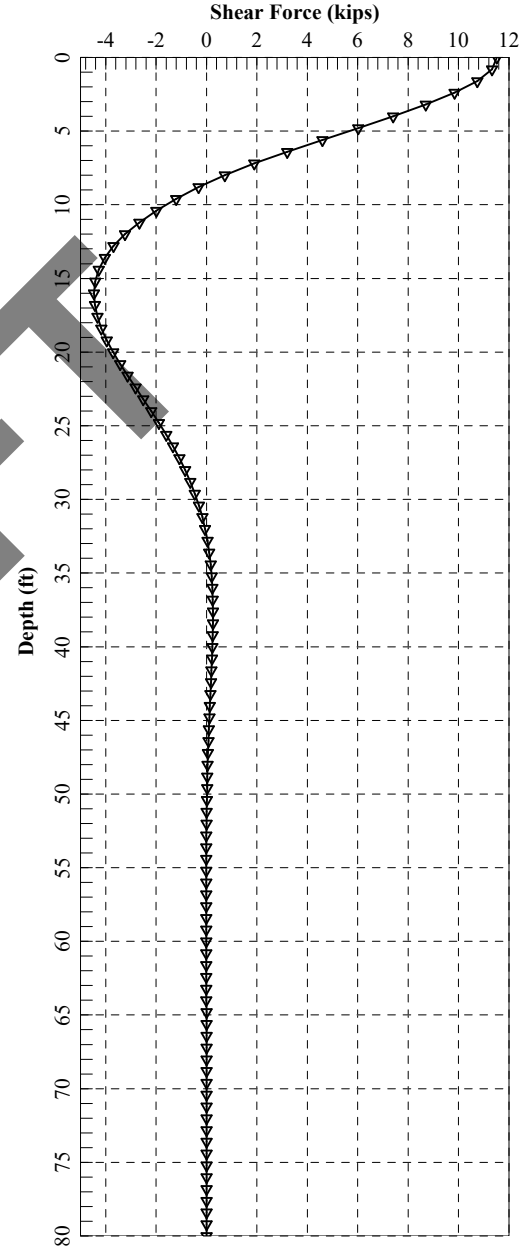
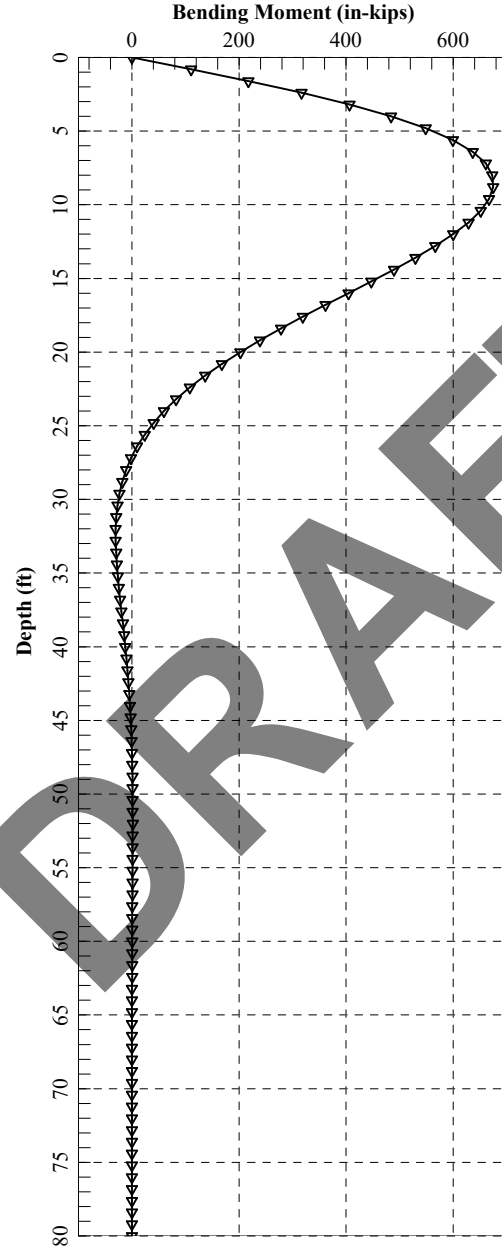
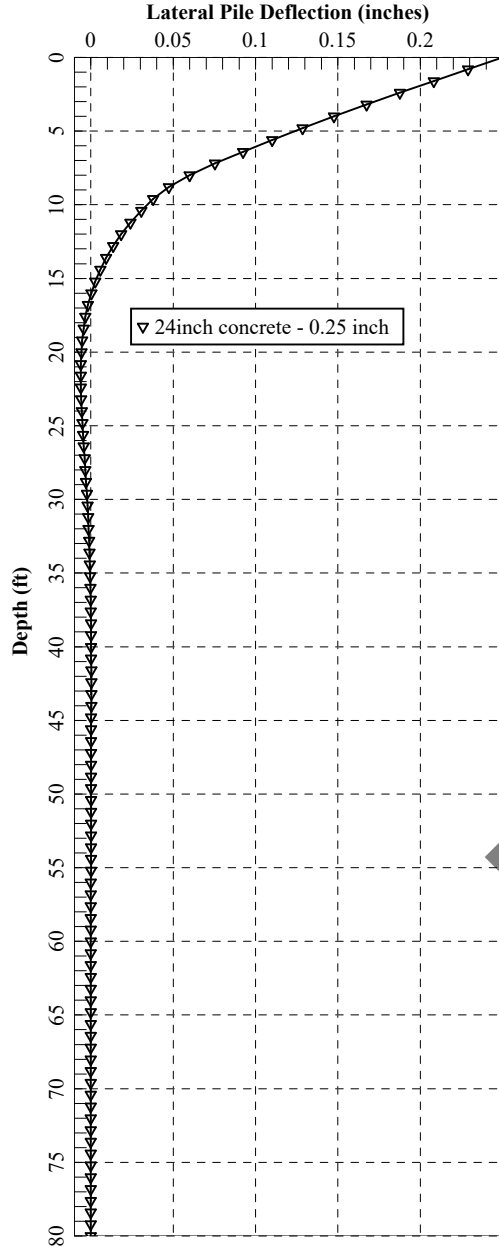


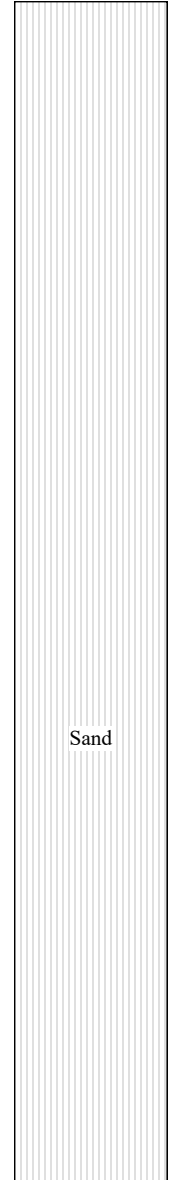
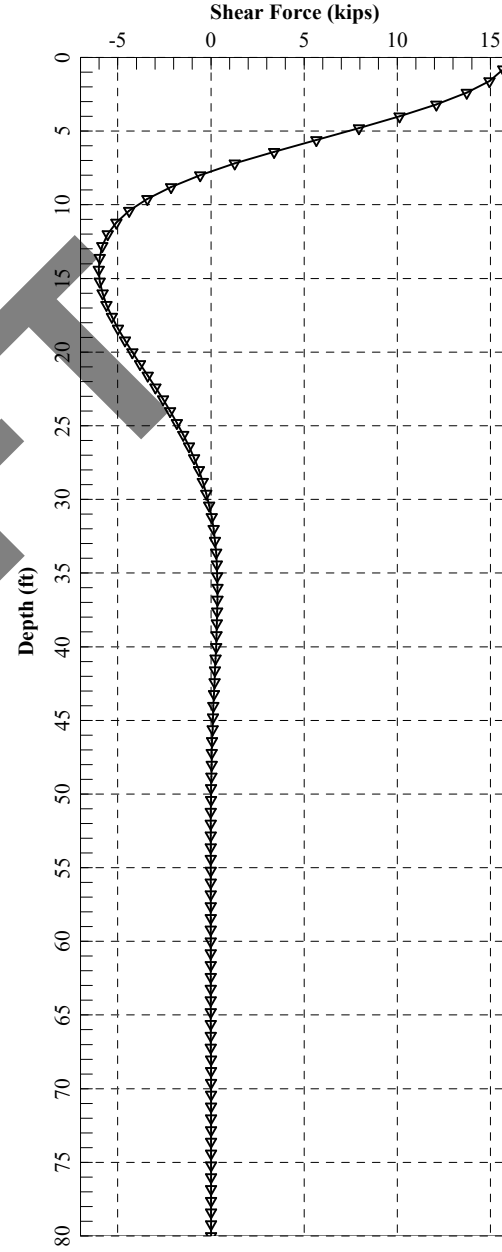
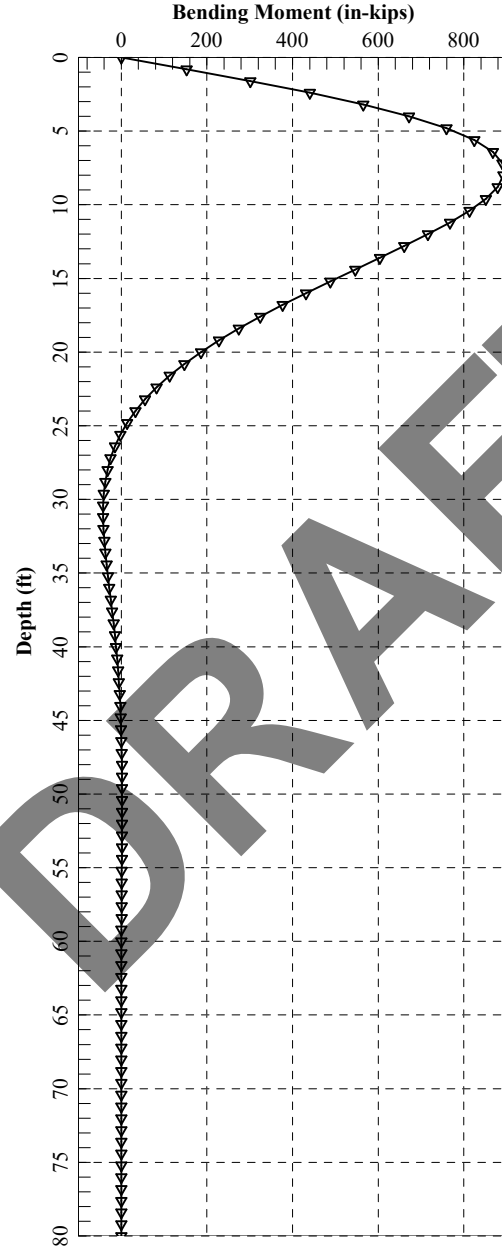
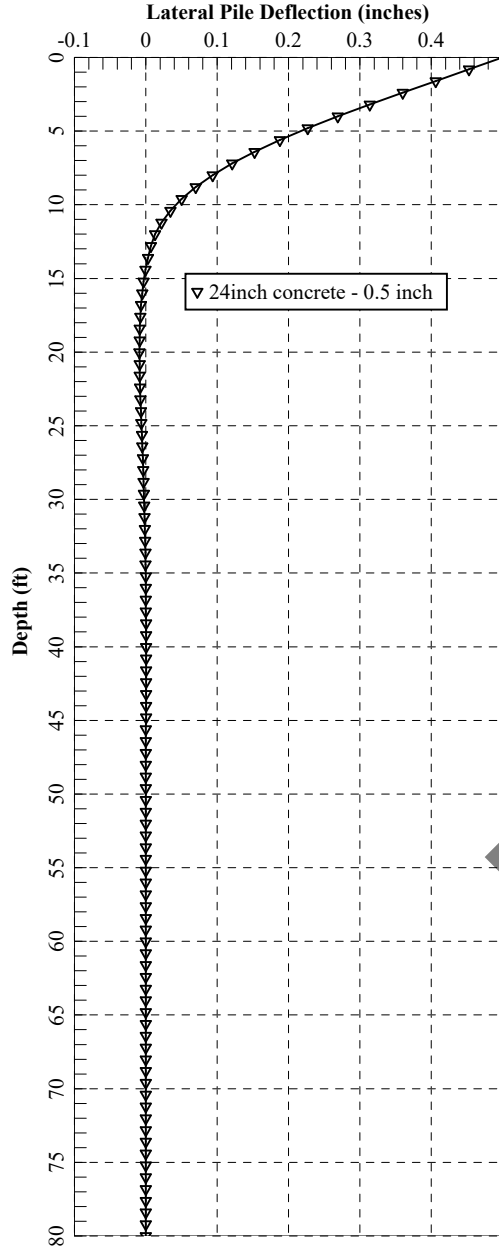
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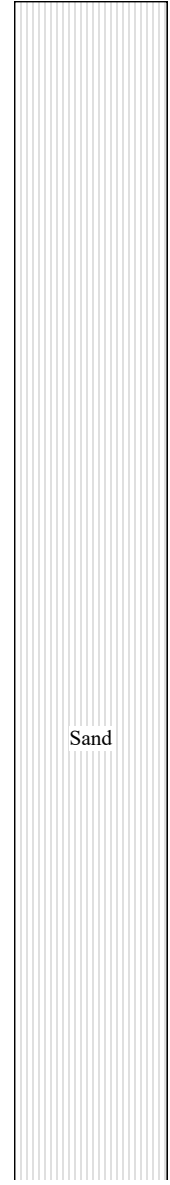
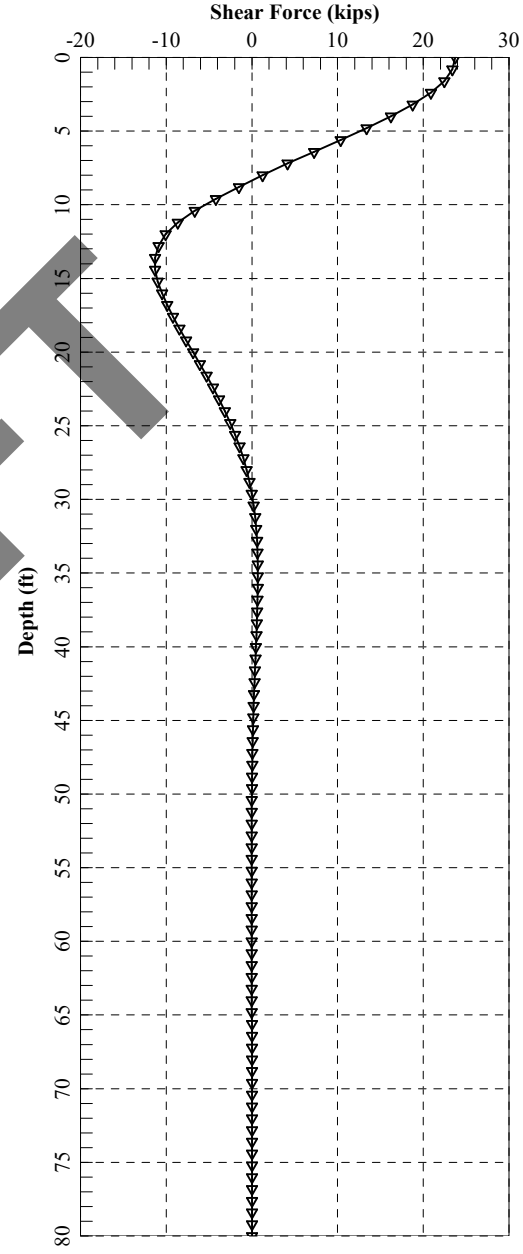
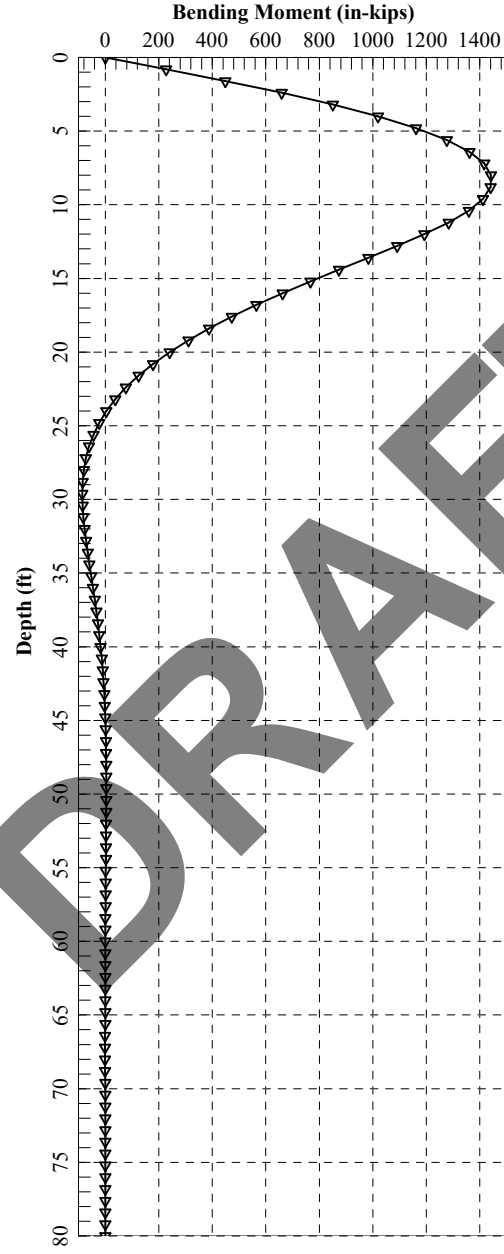
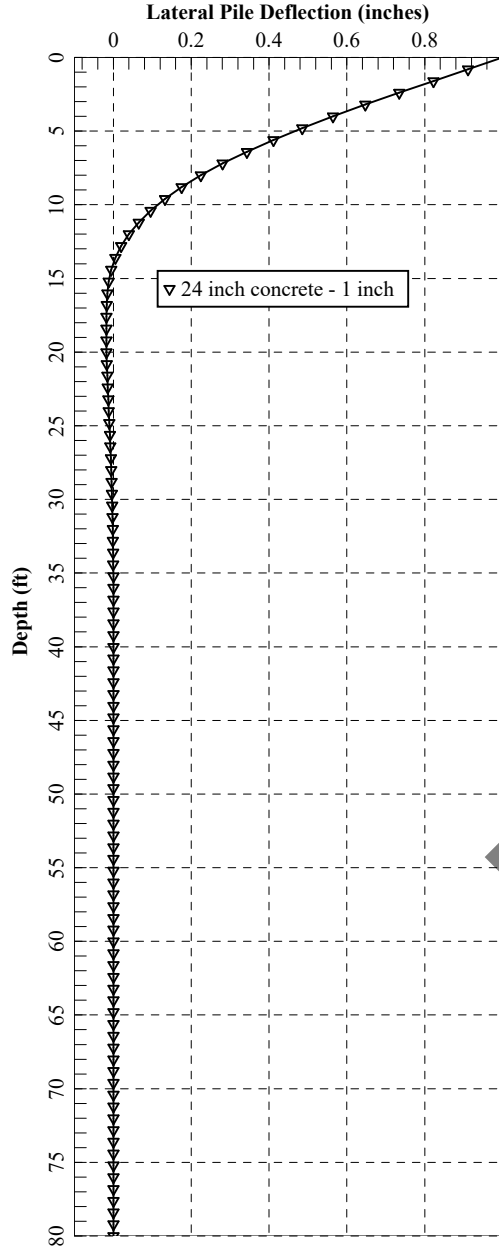
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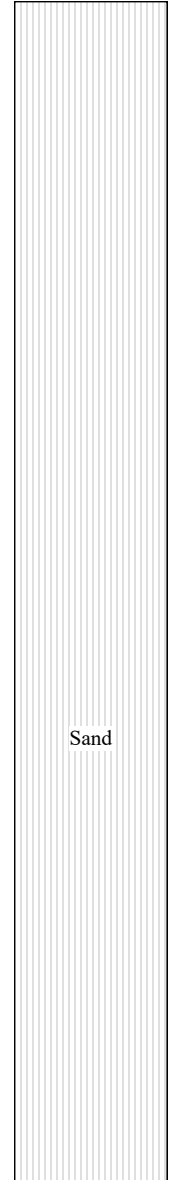
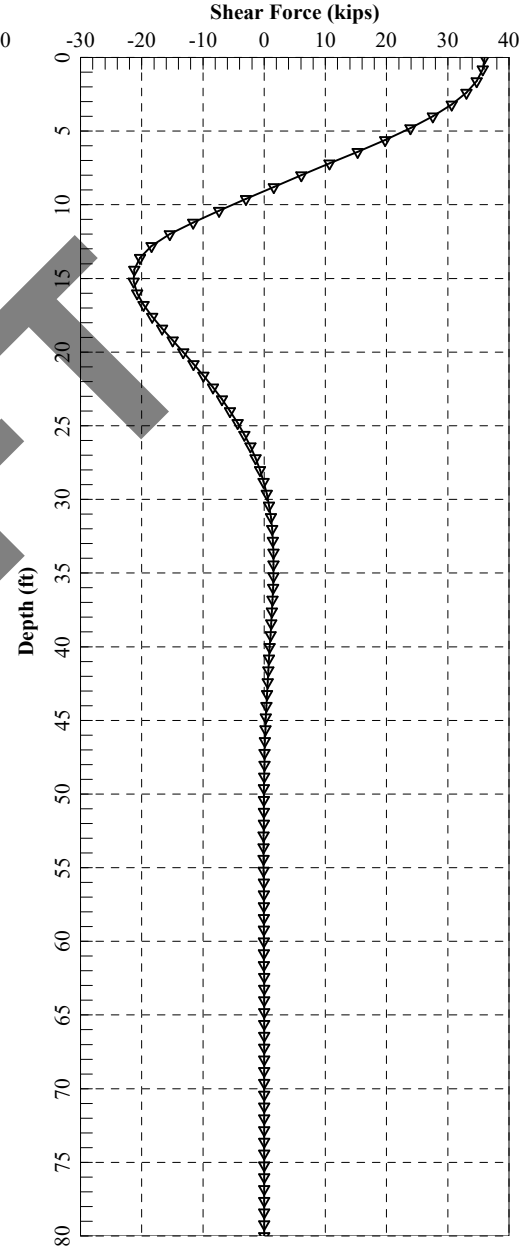
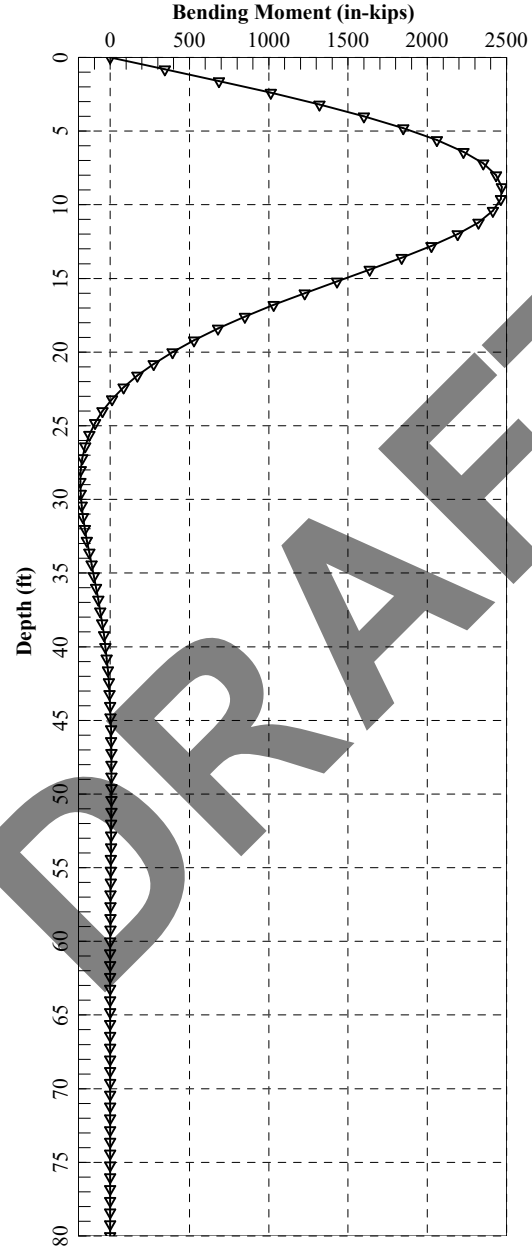
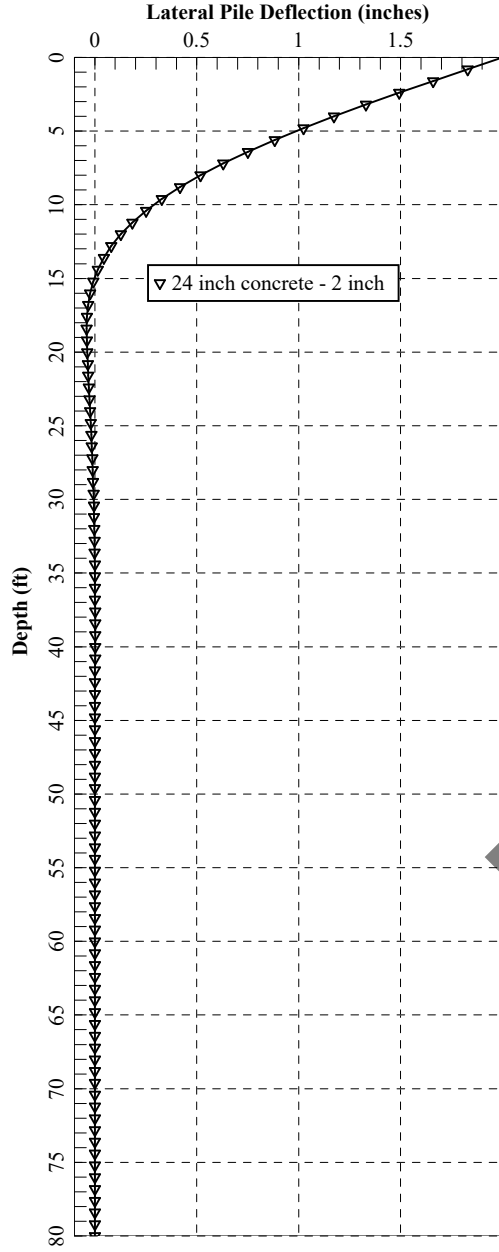
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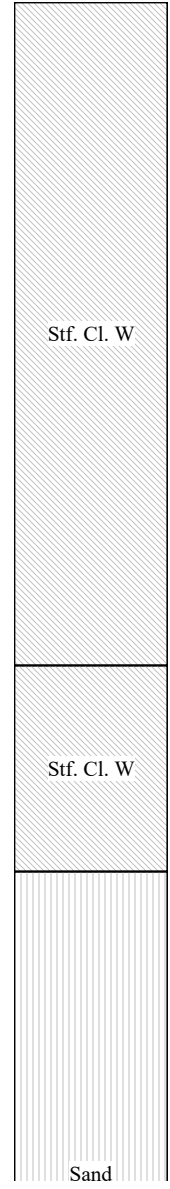
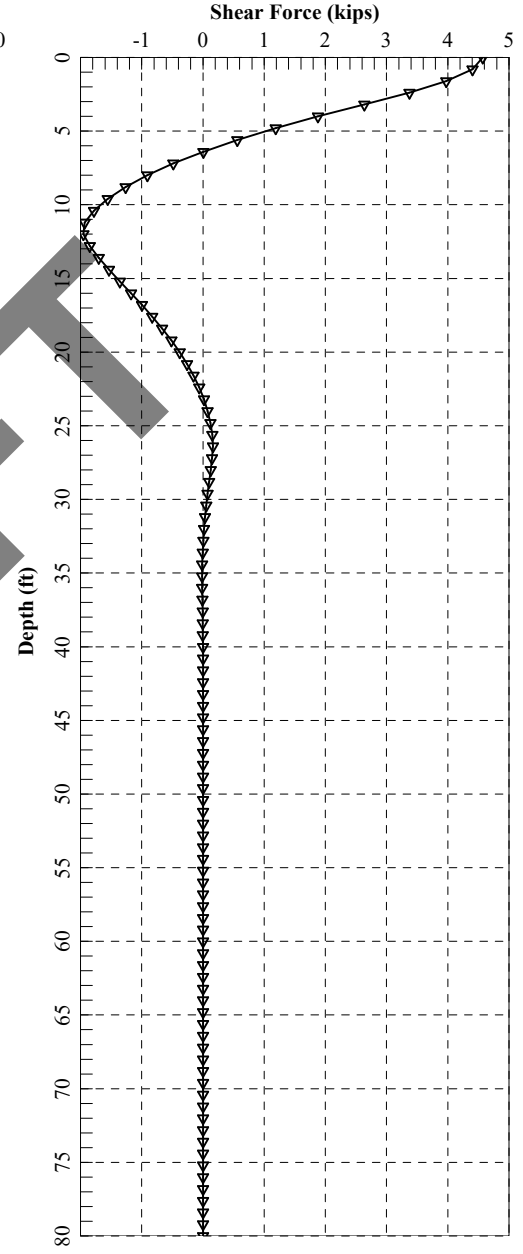
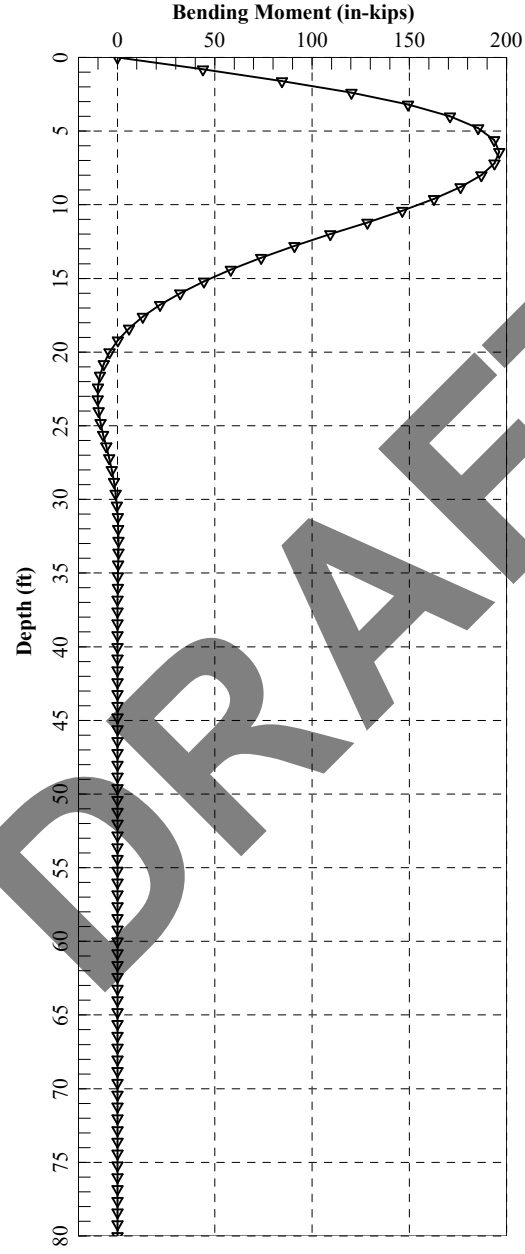
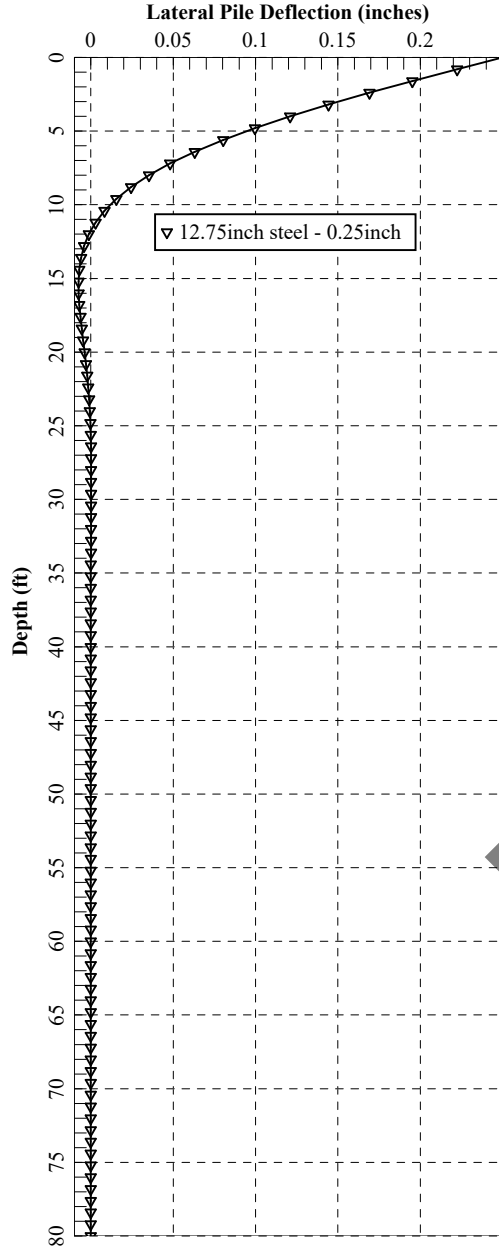


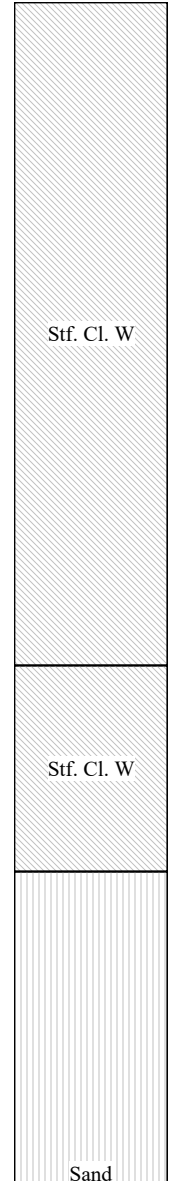
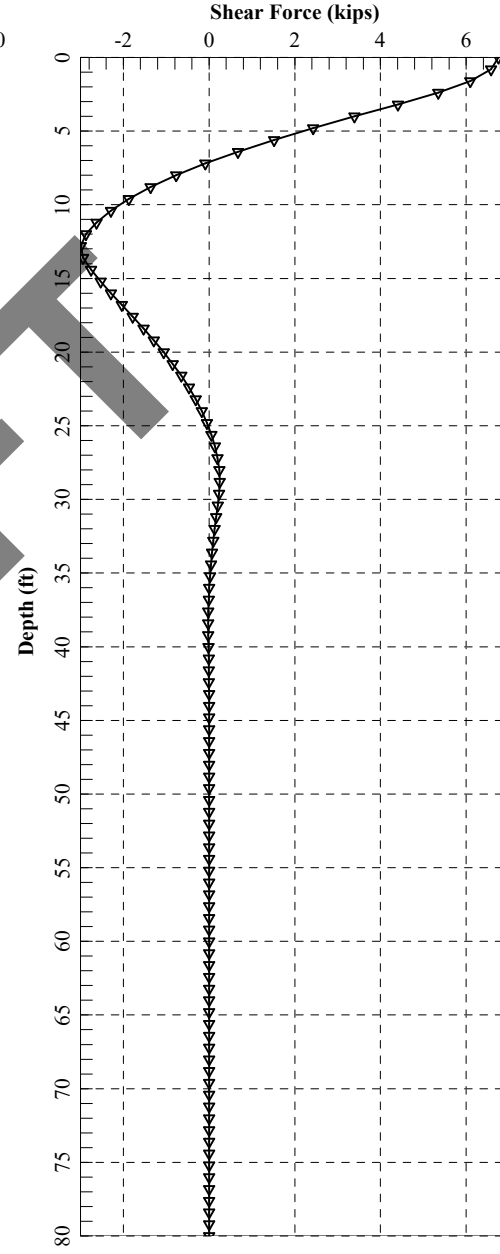
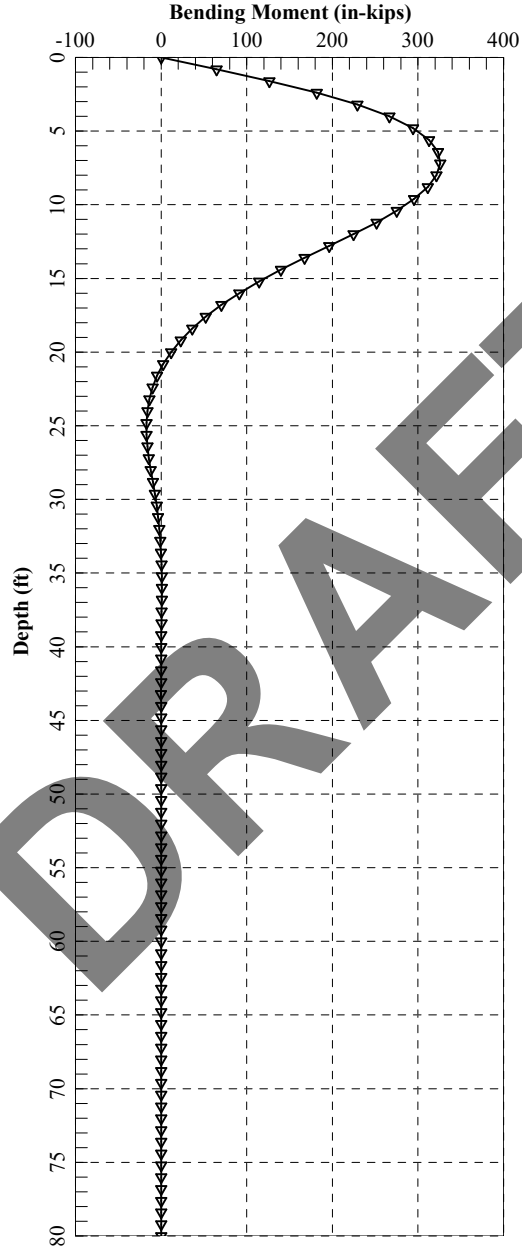
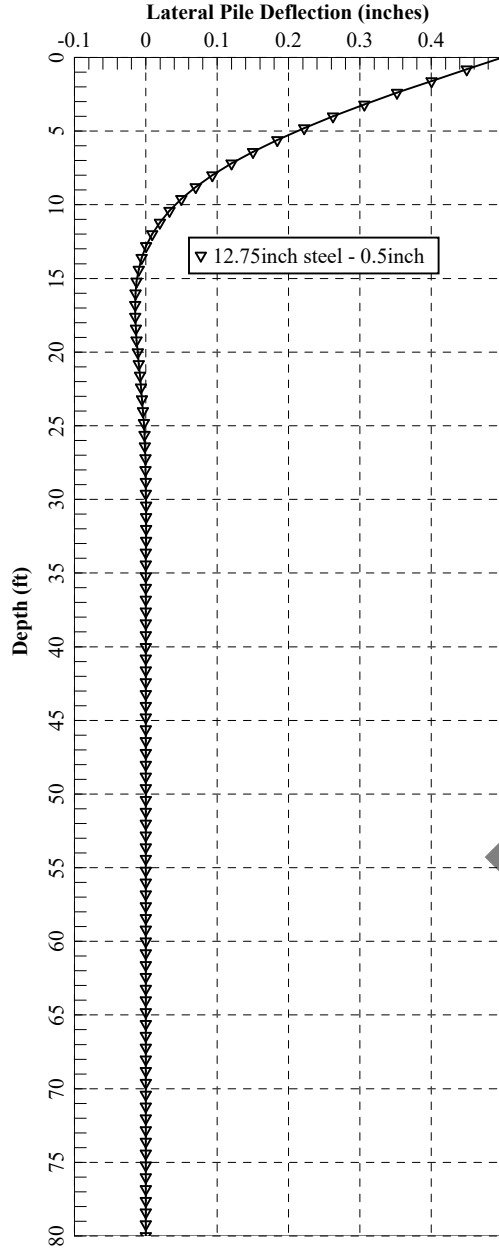
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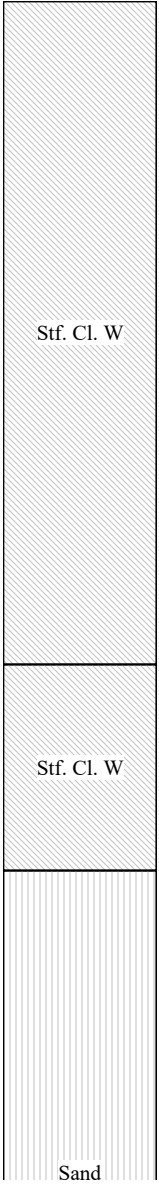
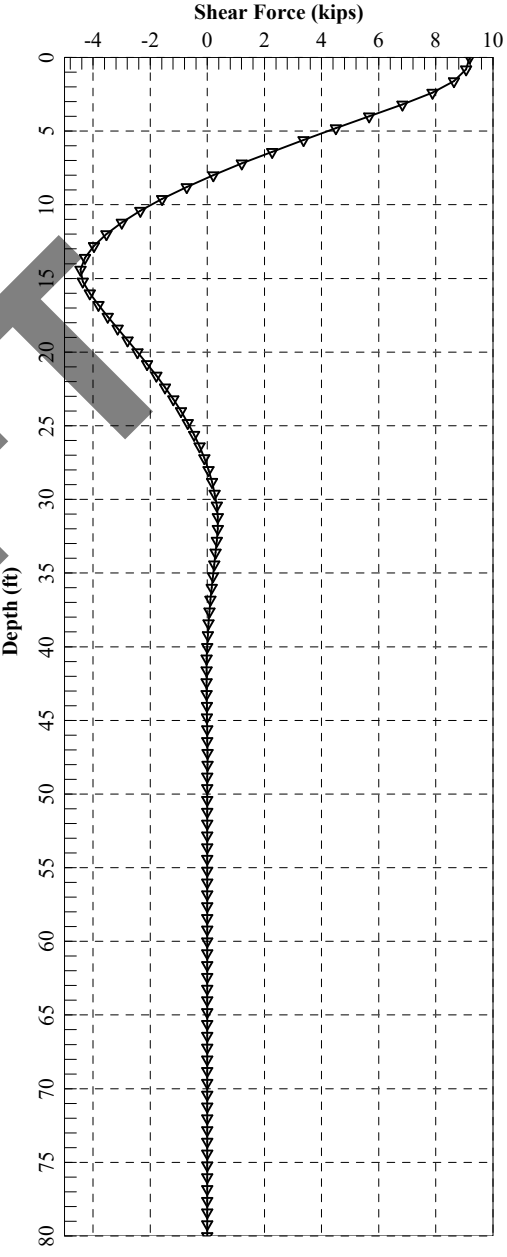
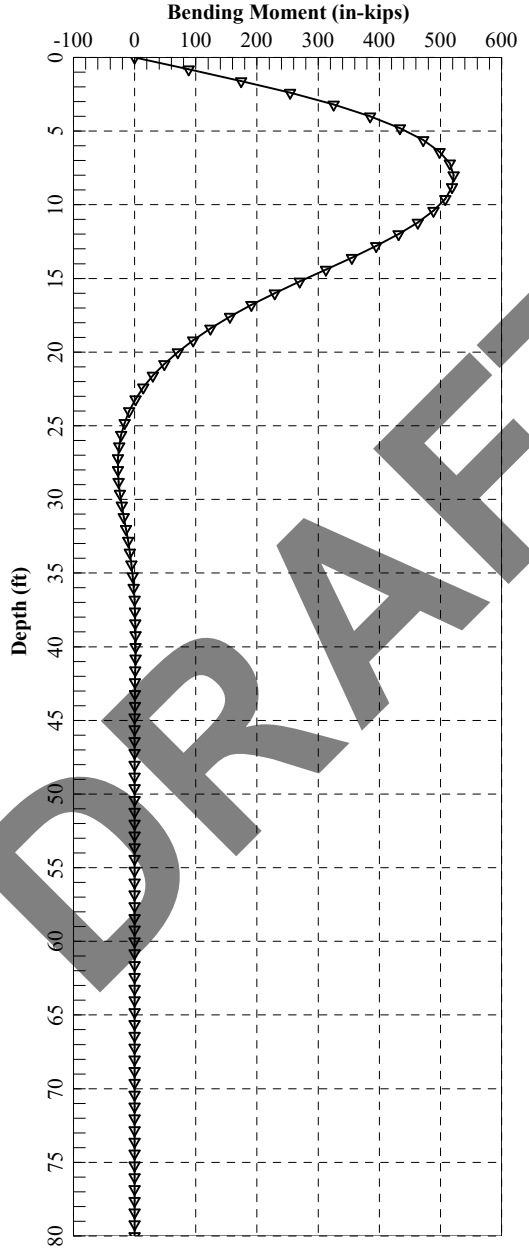
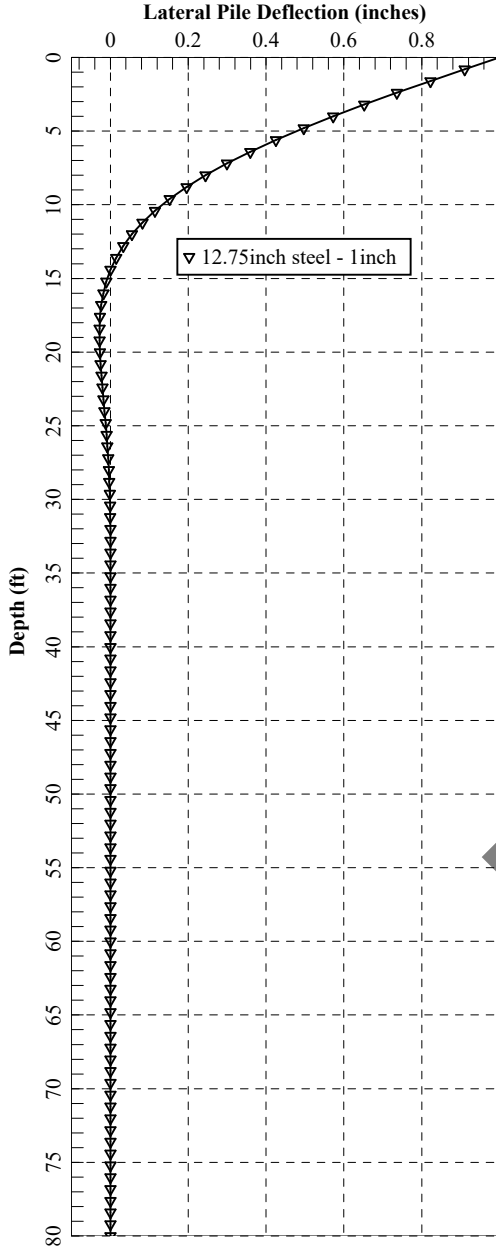
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(0.25", 0.5", 1", 2")

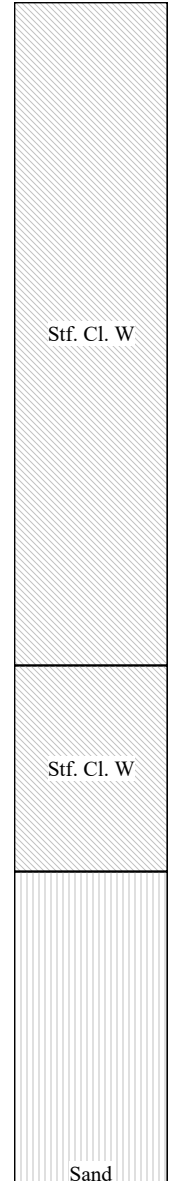
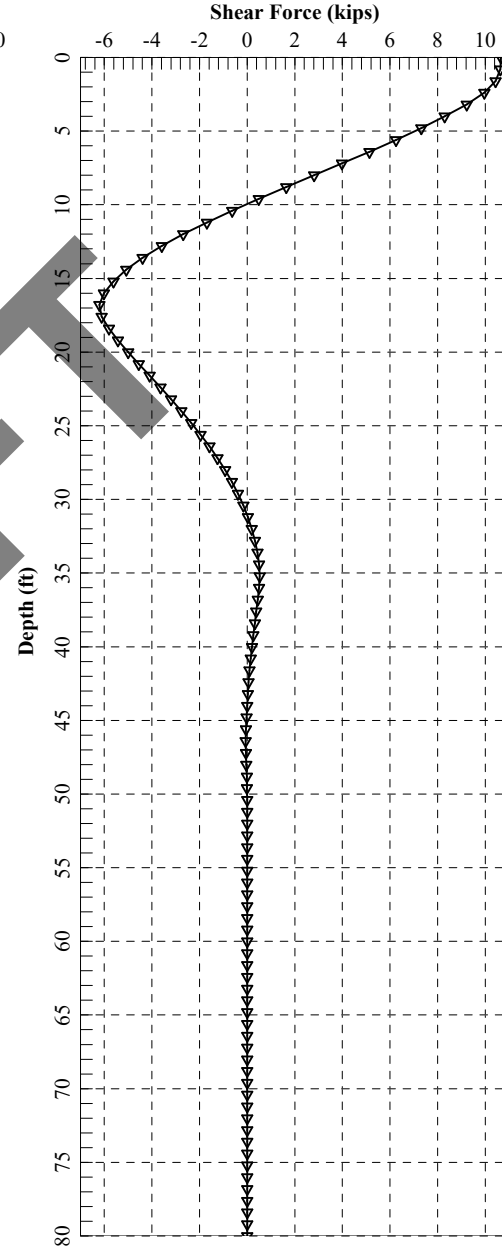
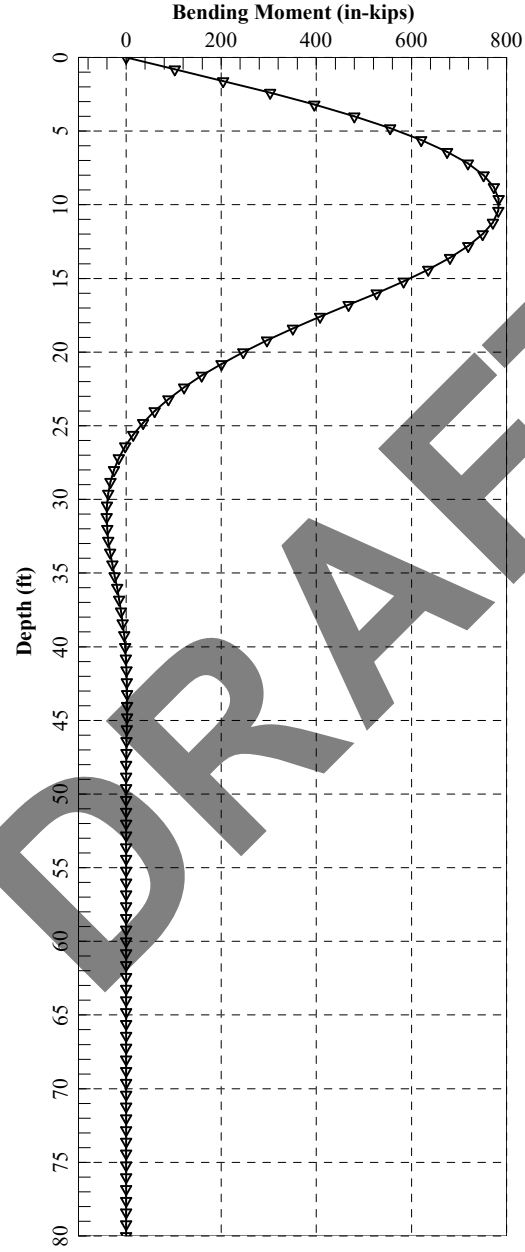
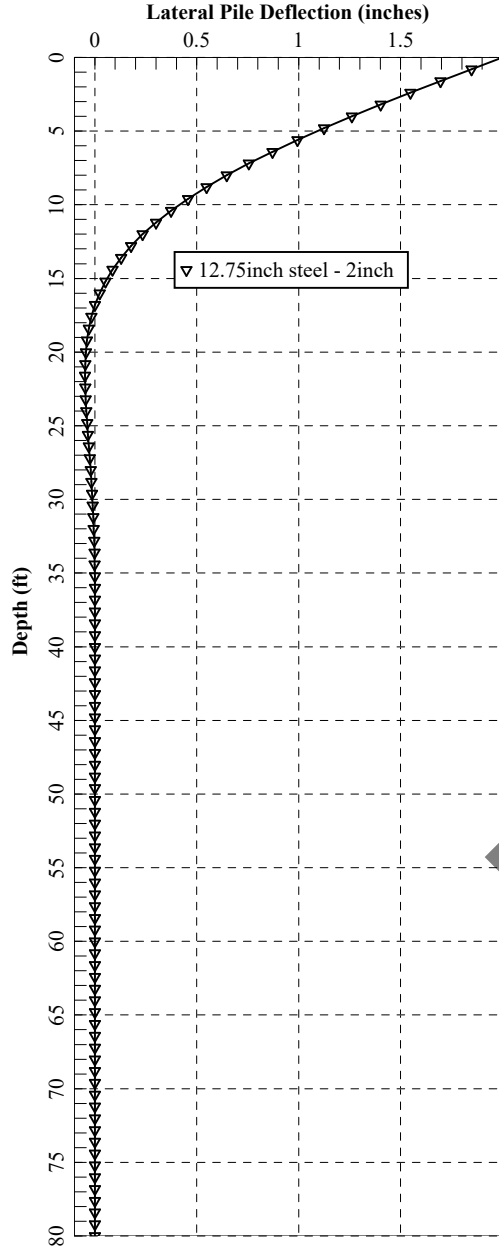
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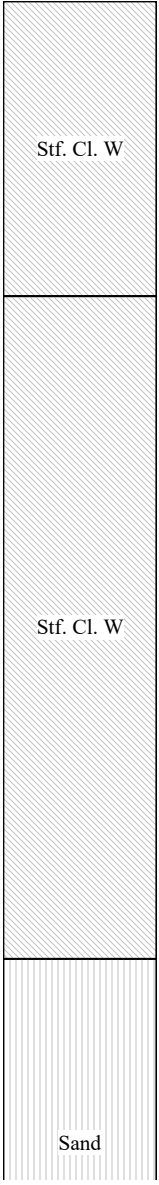
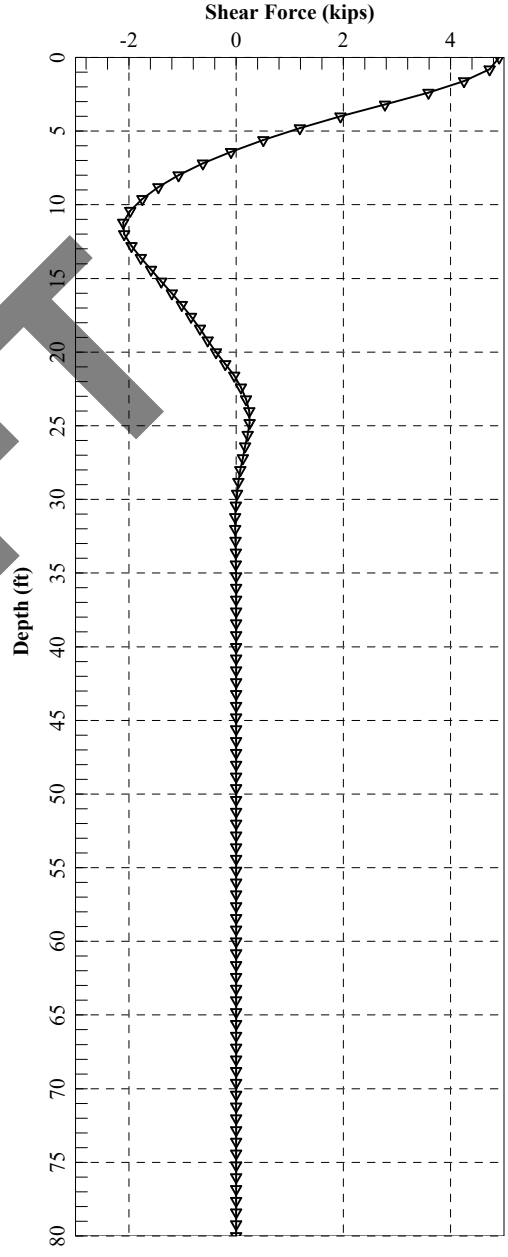
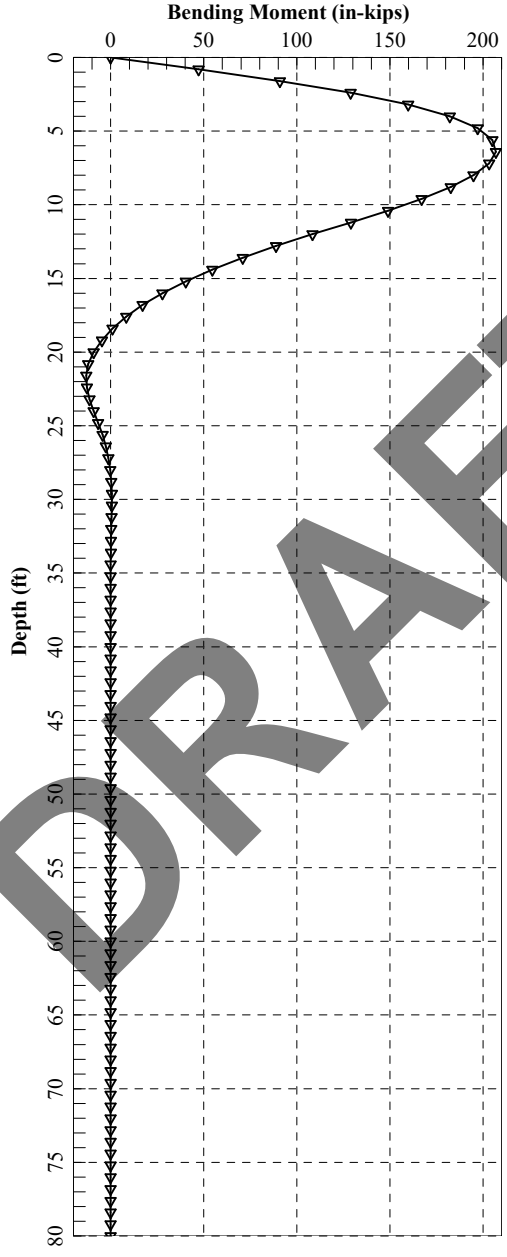
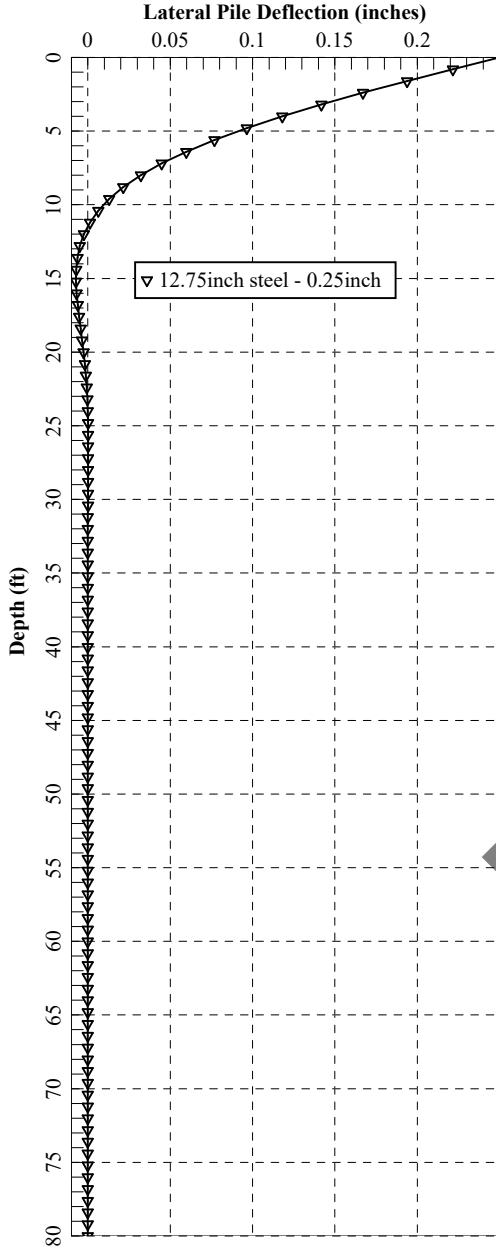


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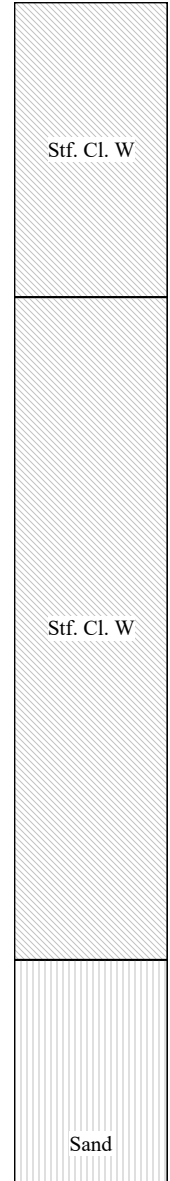
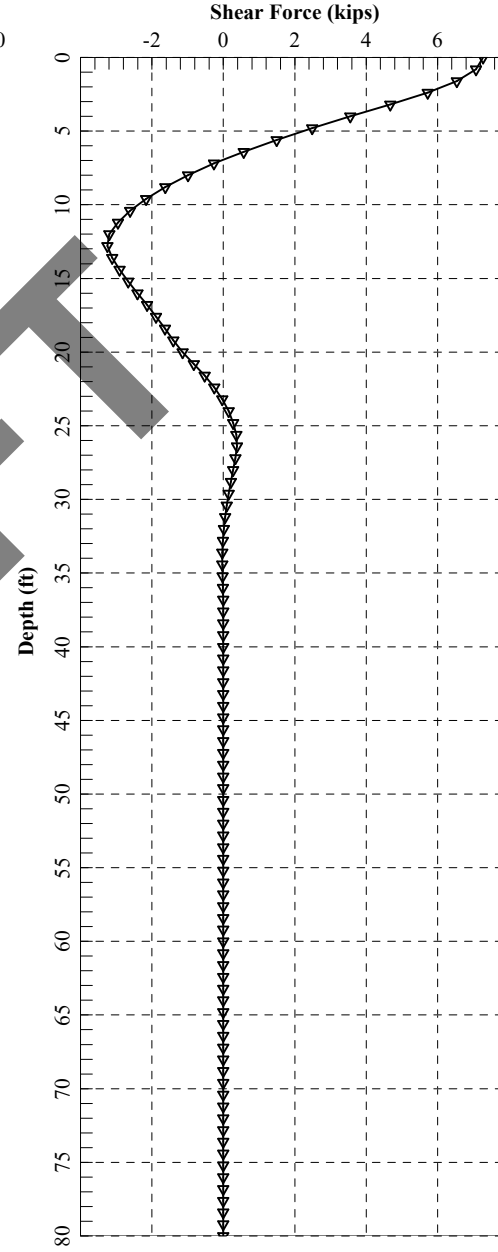
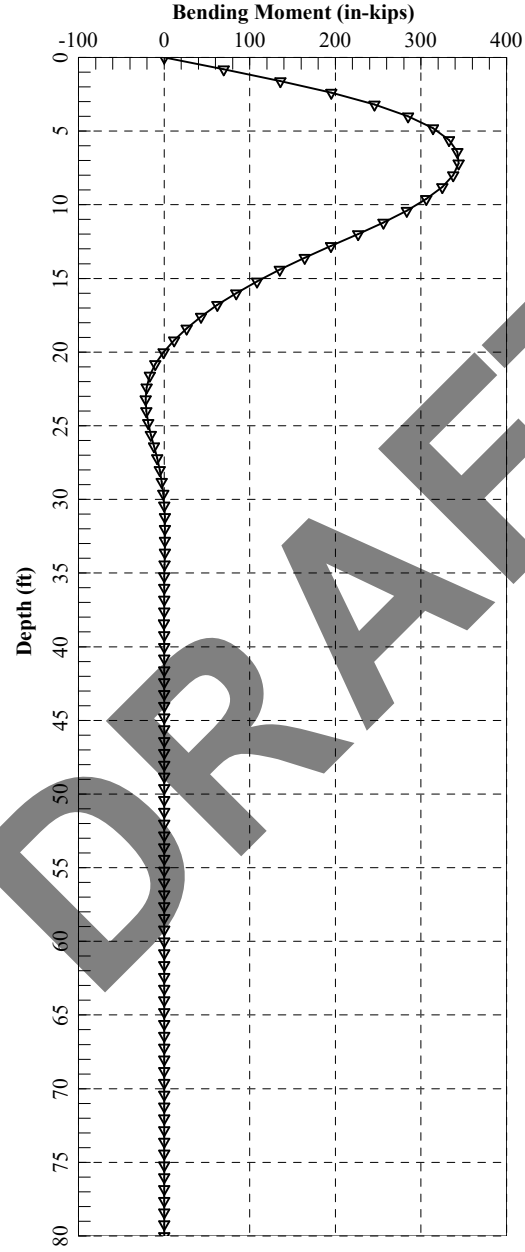
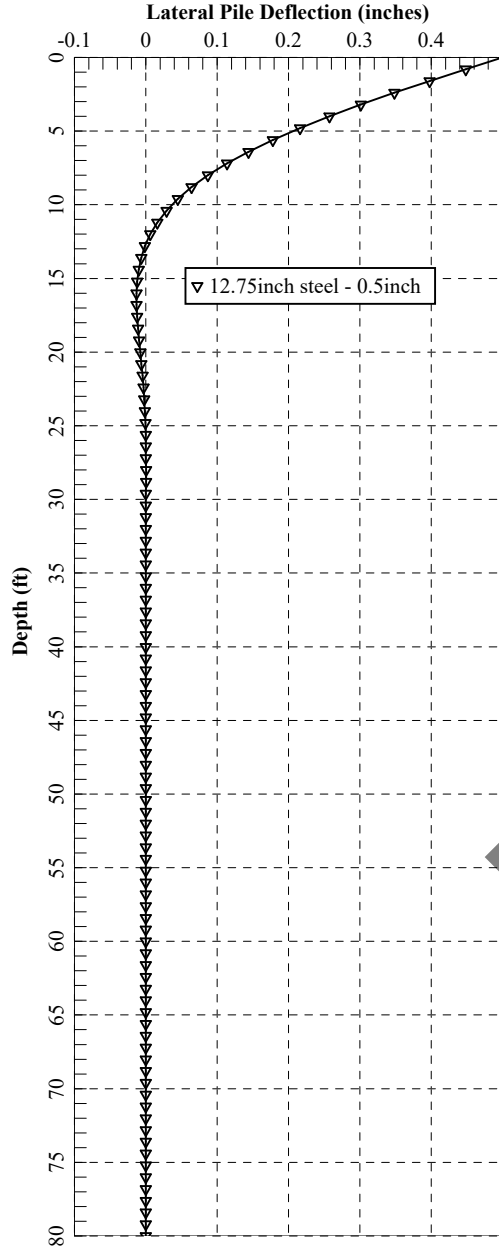
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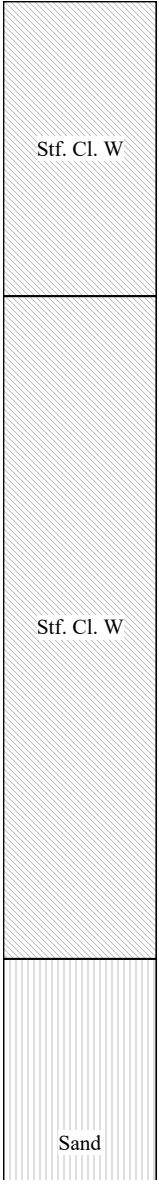
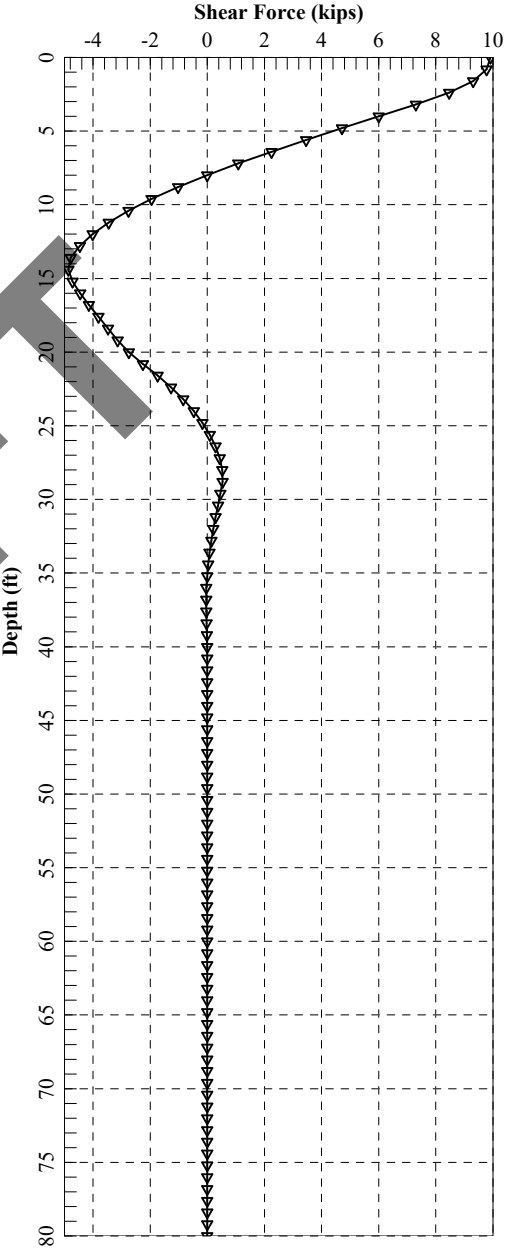
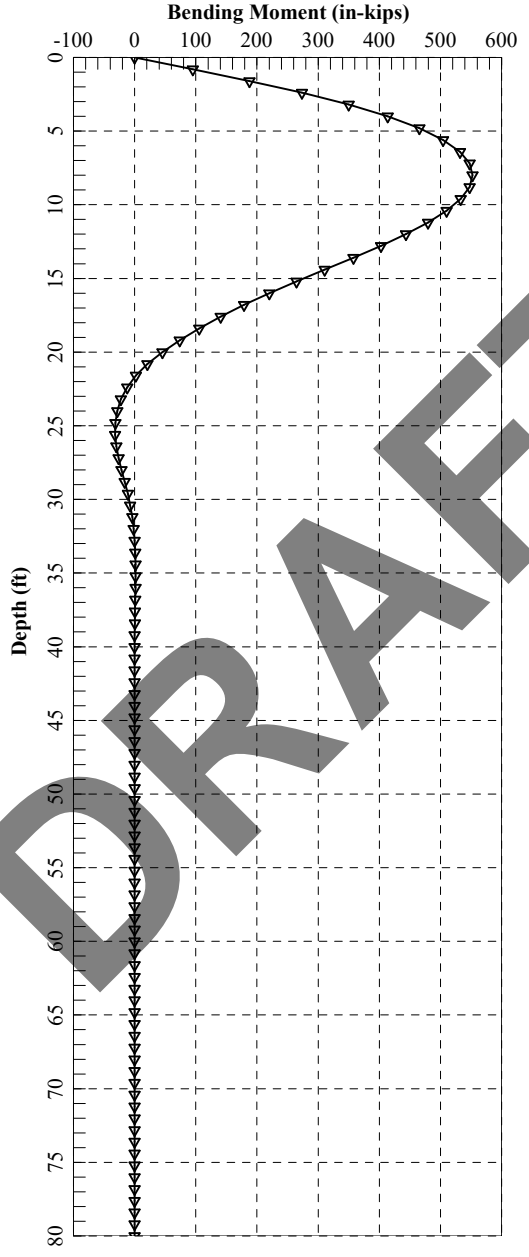
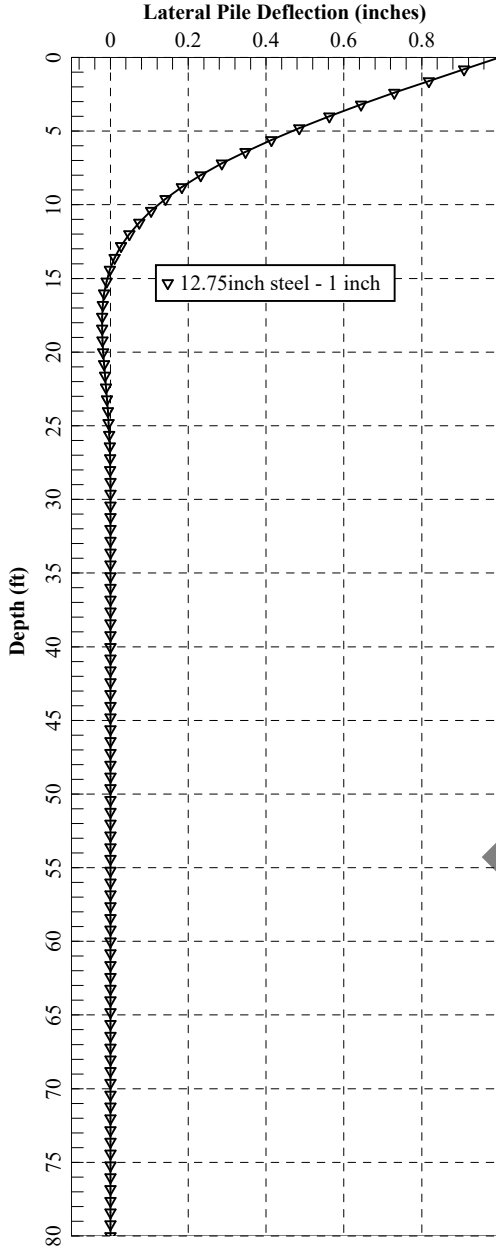
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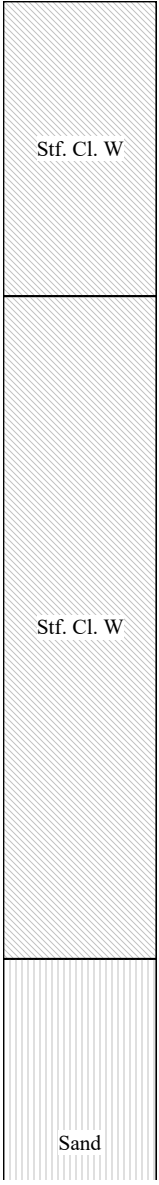
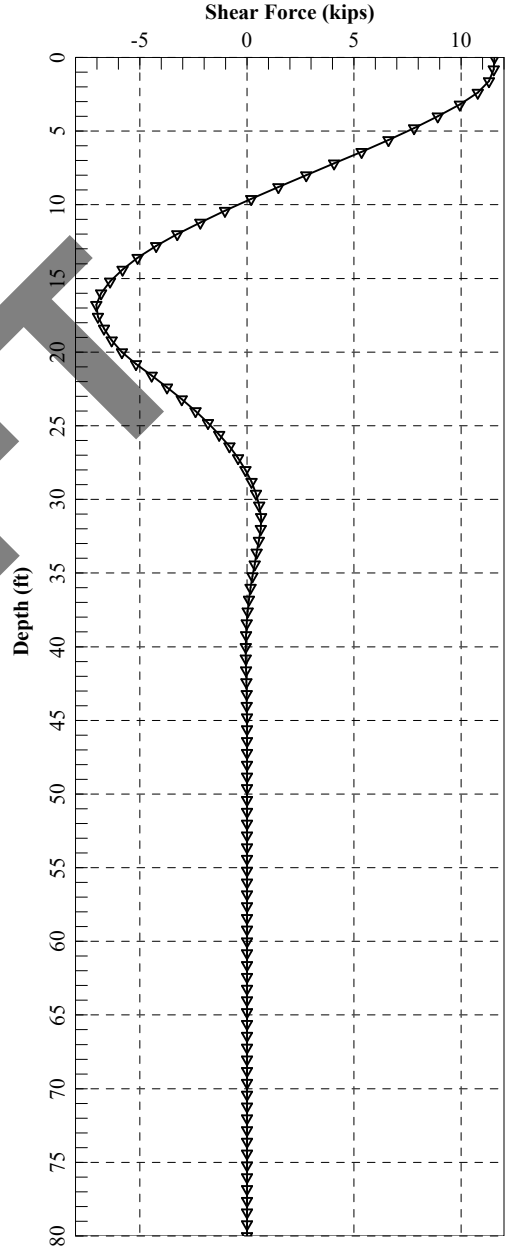
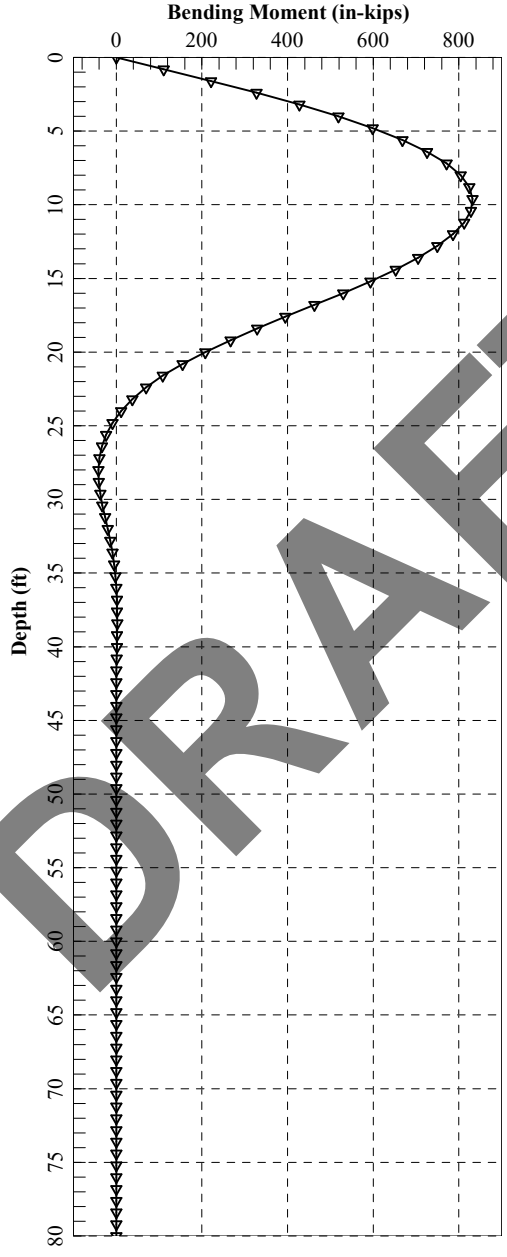
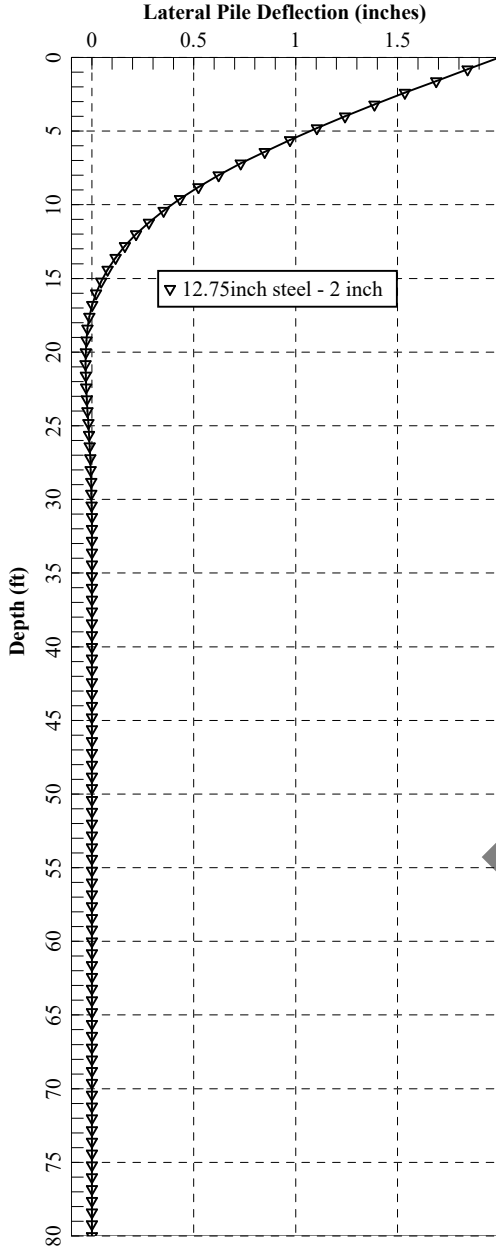
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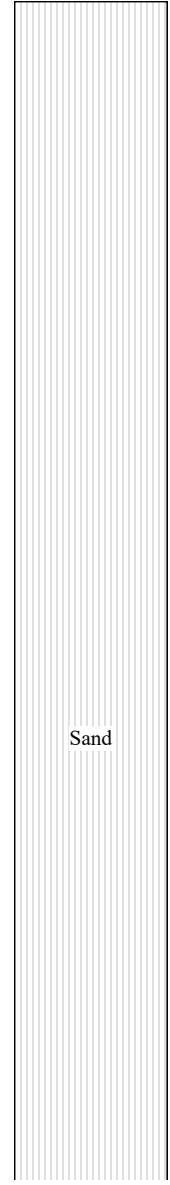
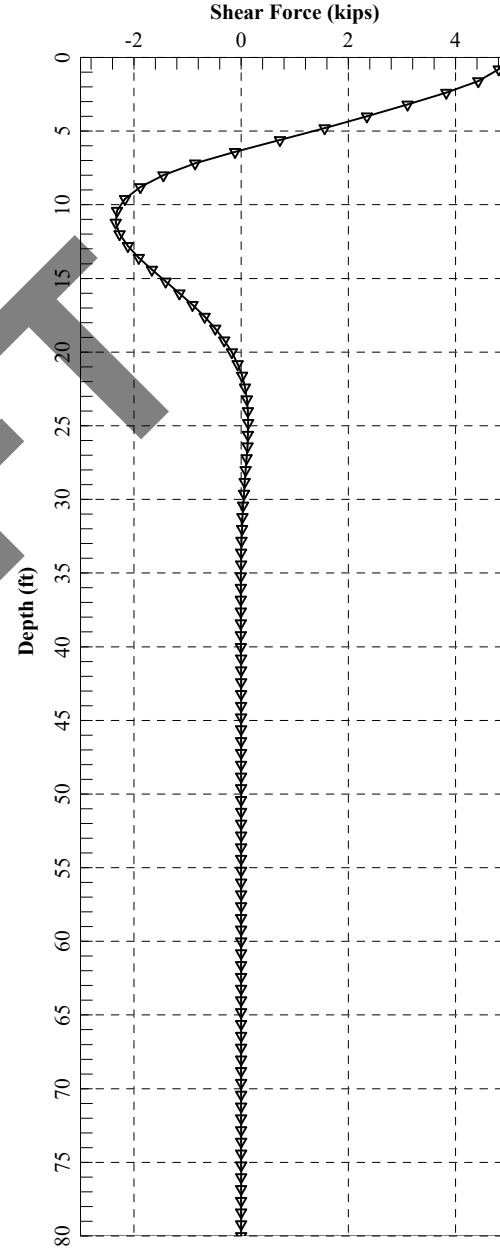
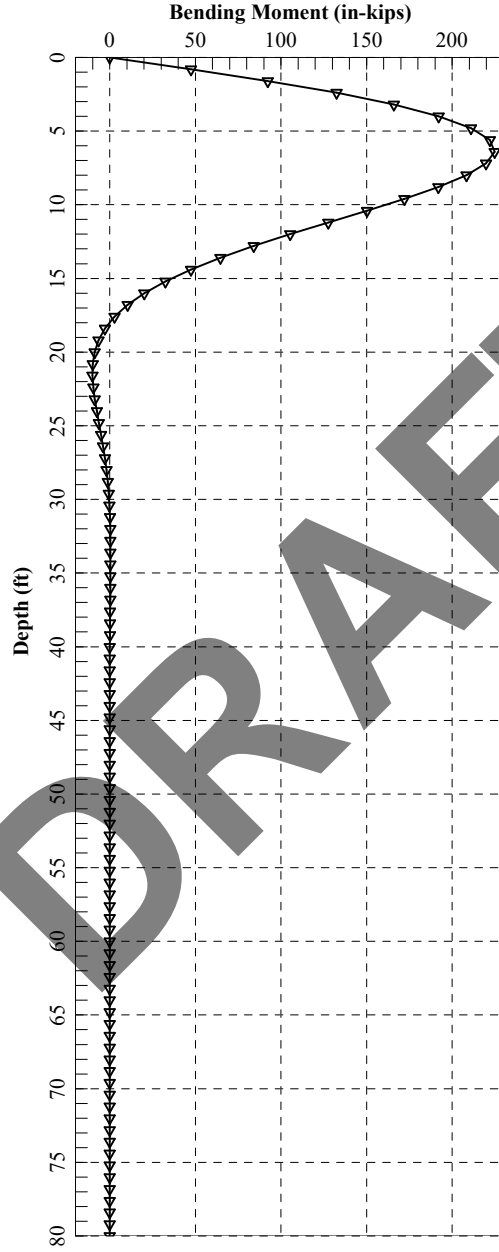
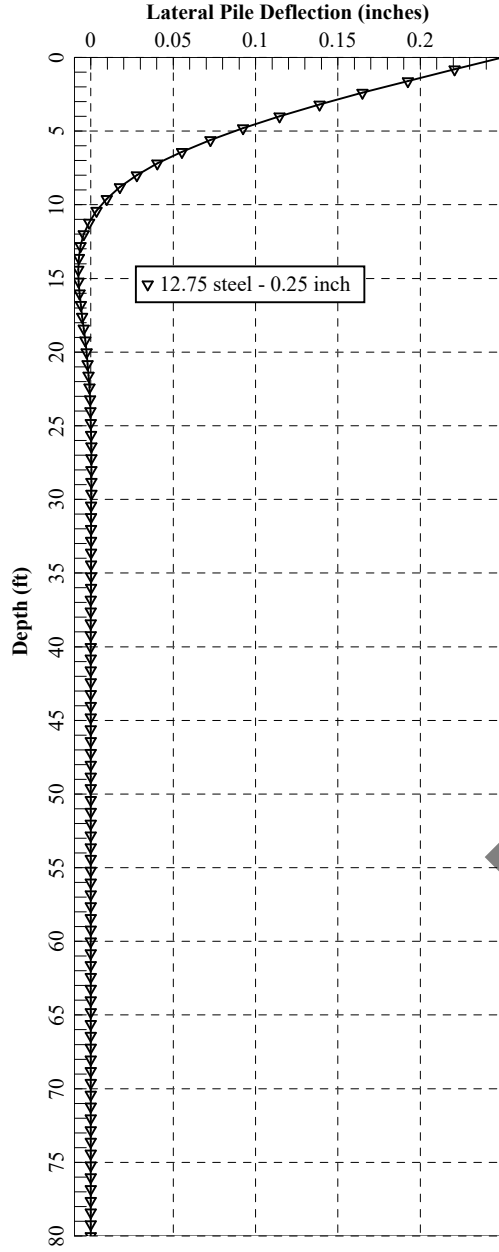
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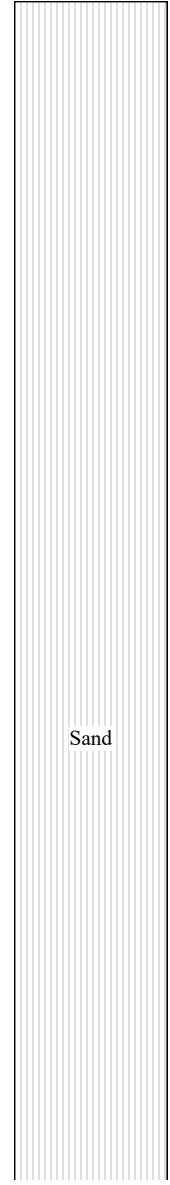
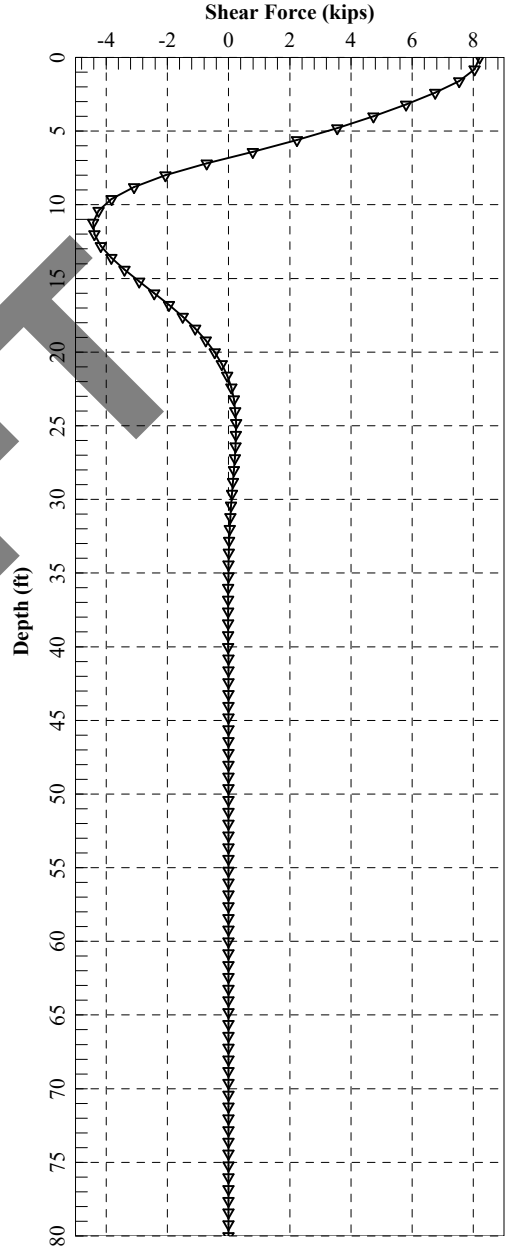
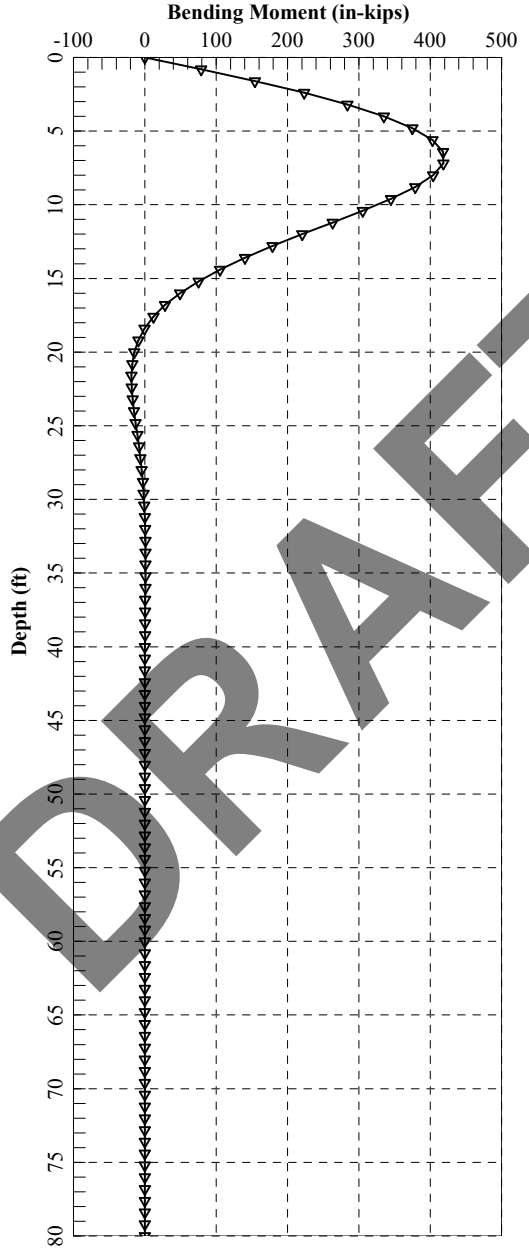
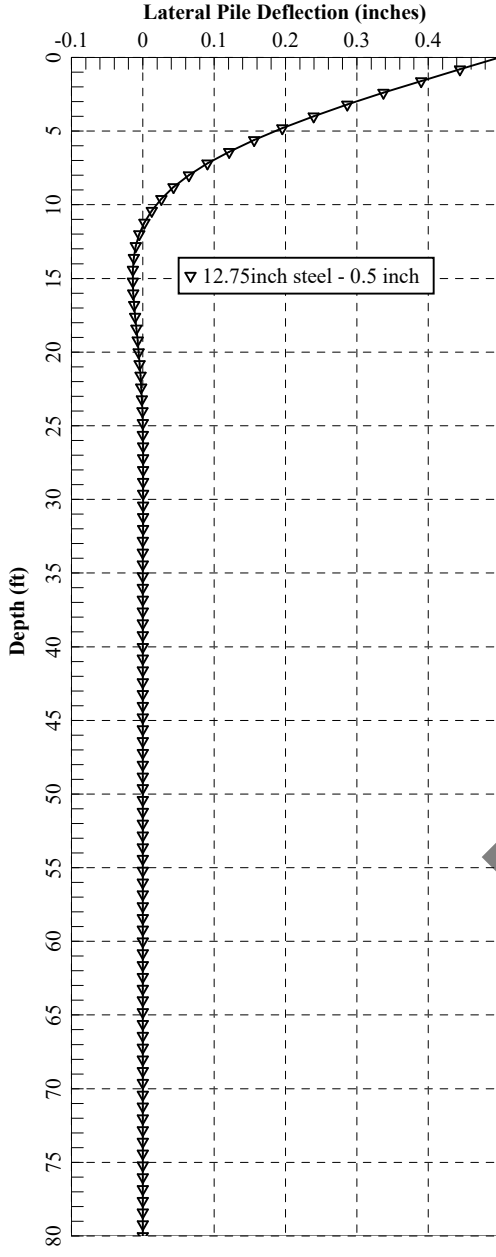
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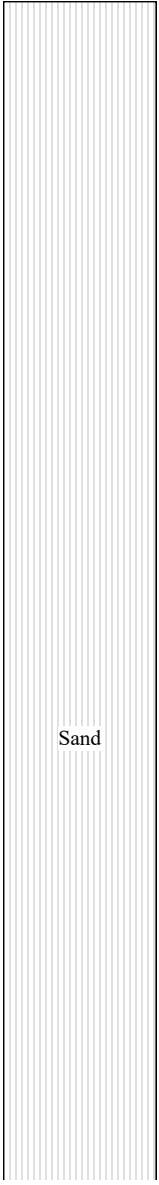
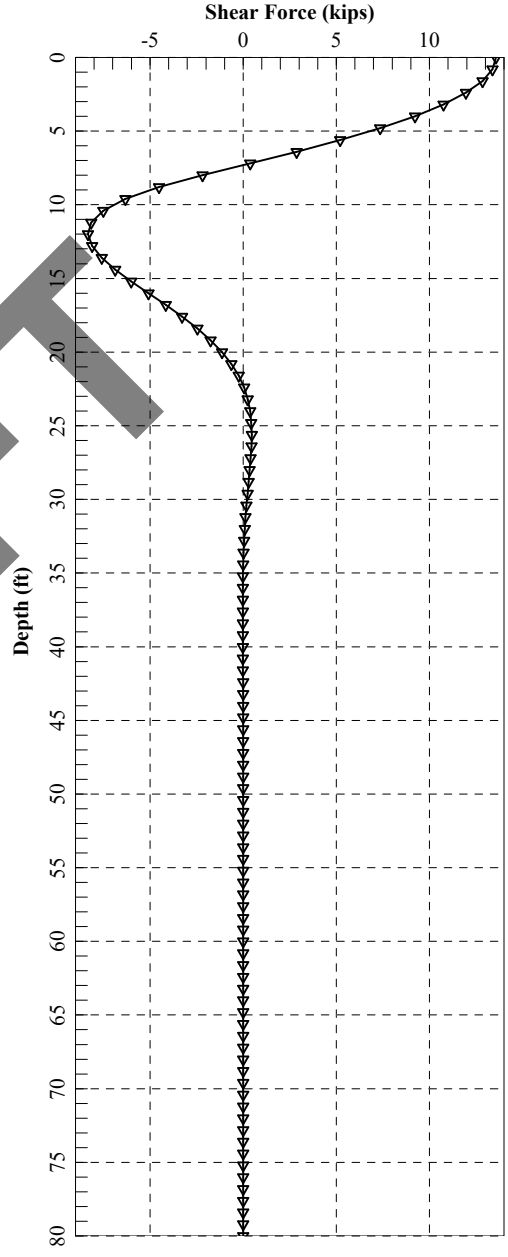
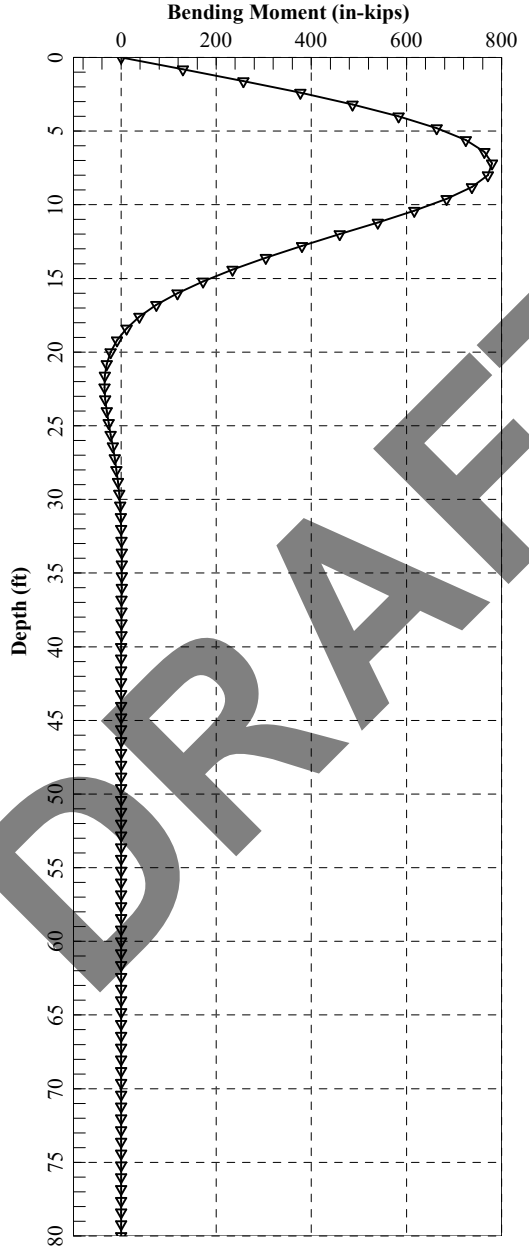
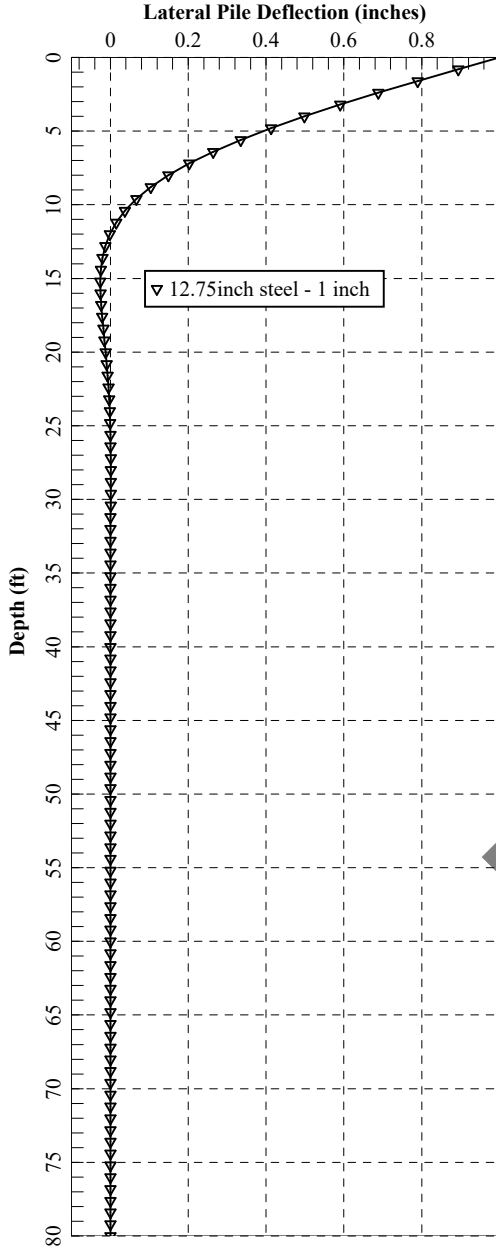
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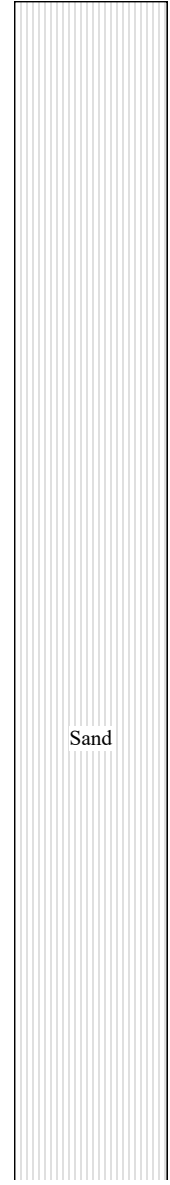
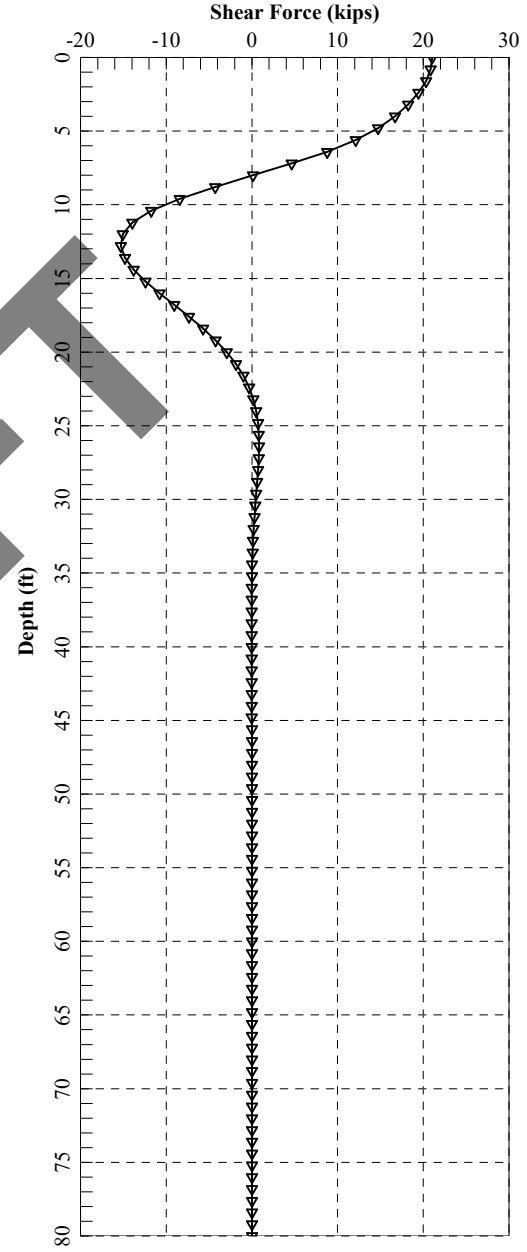
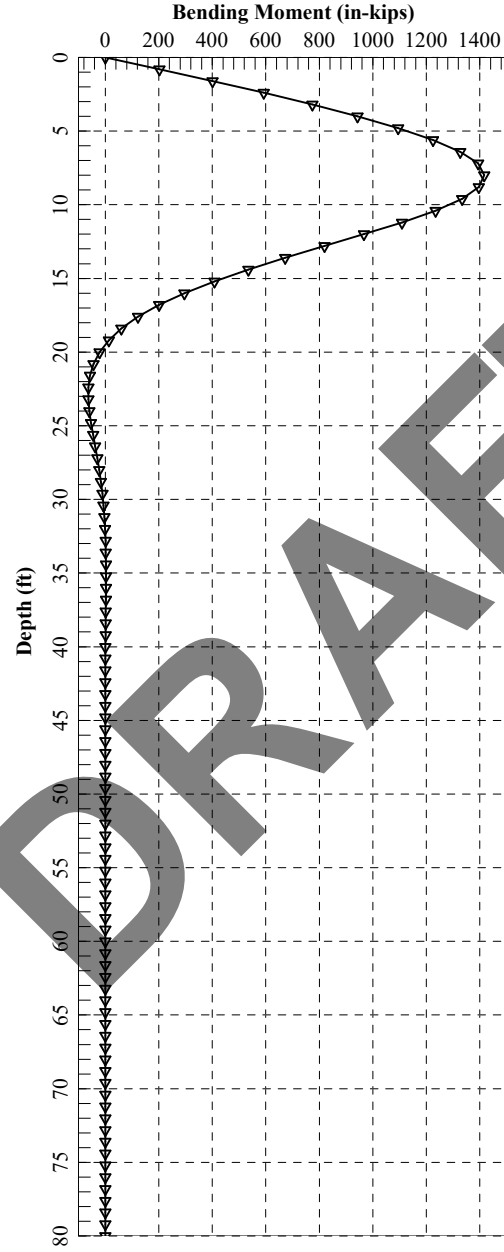
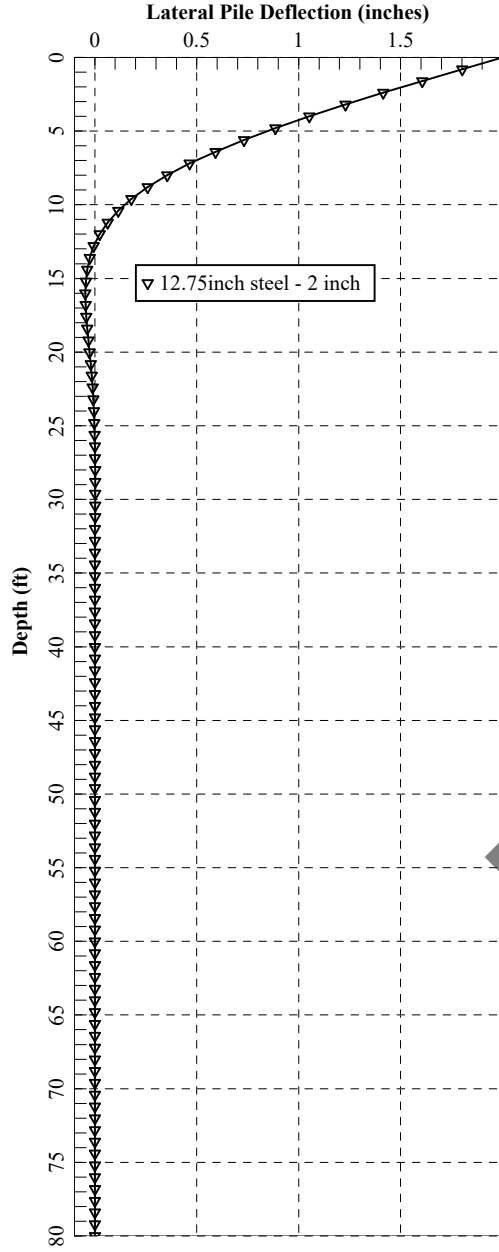
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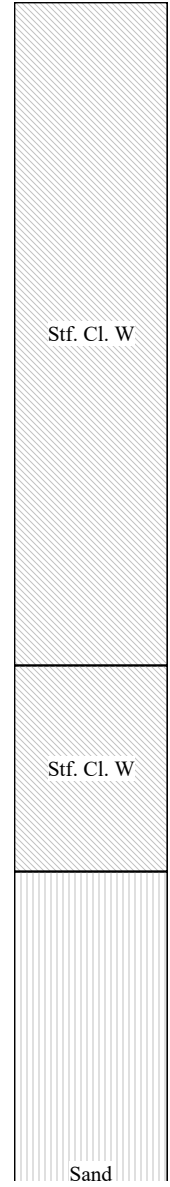
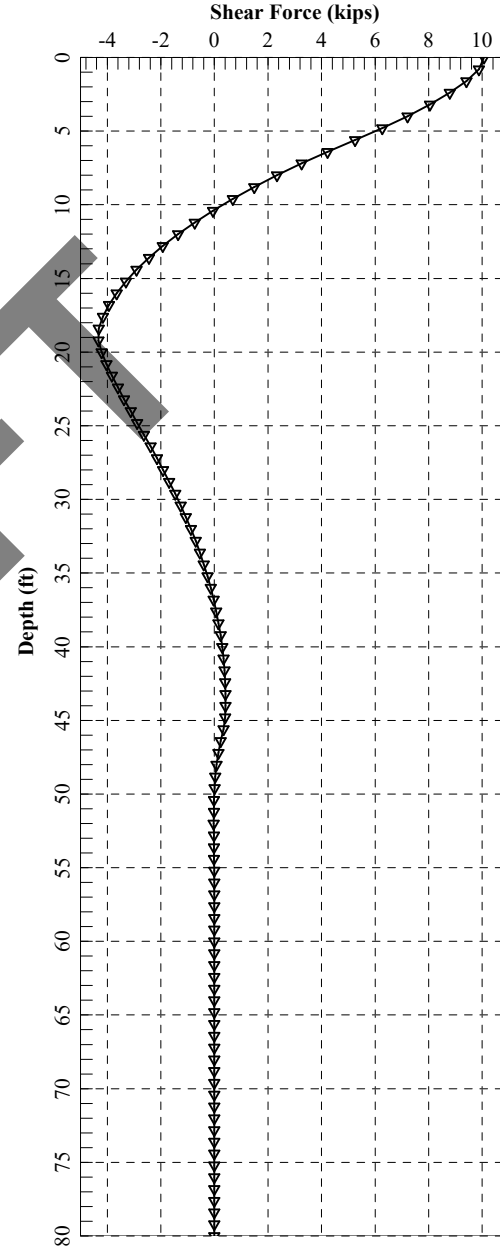
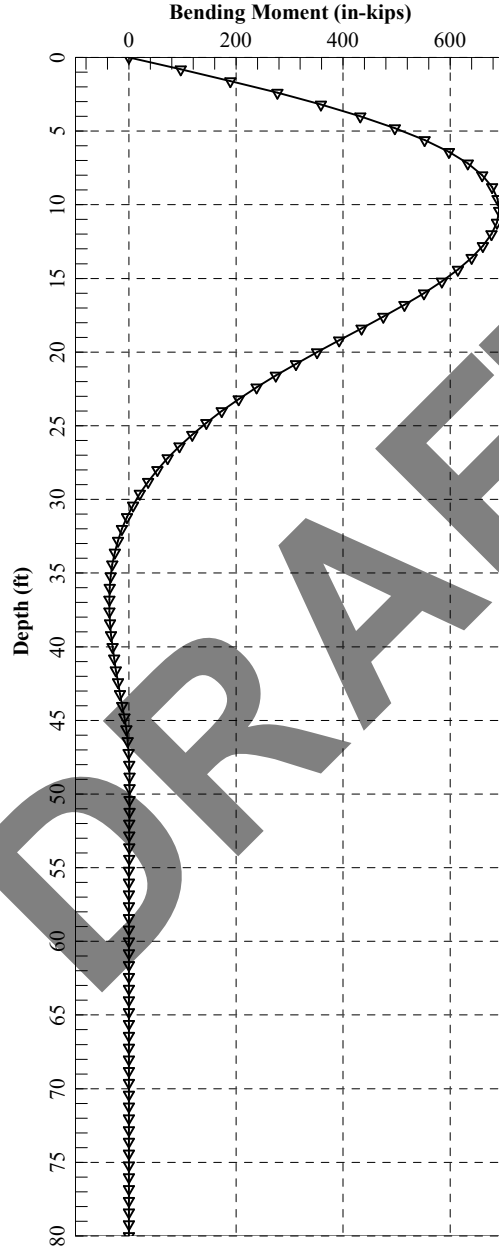
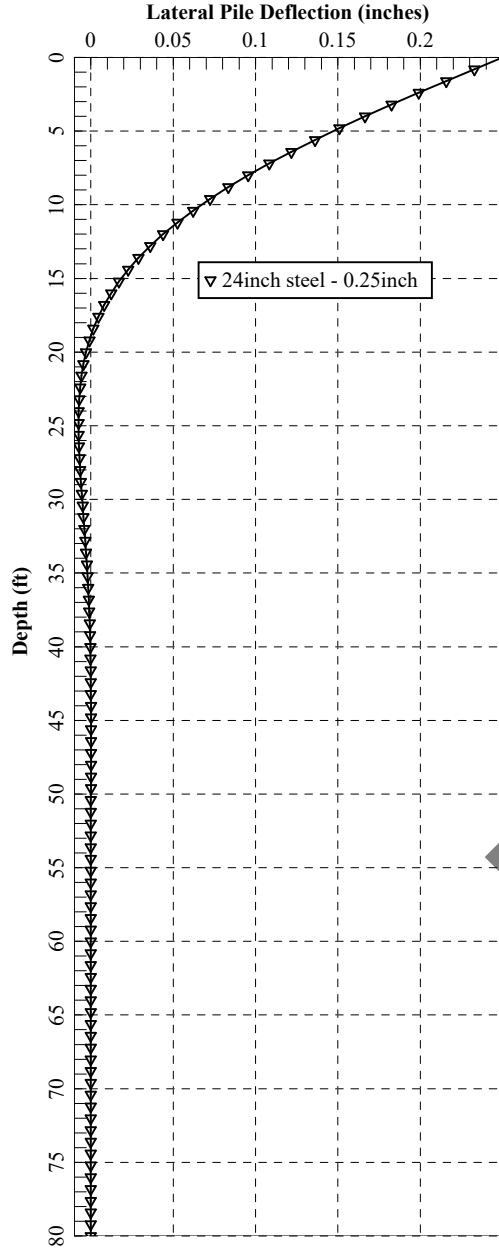


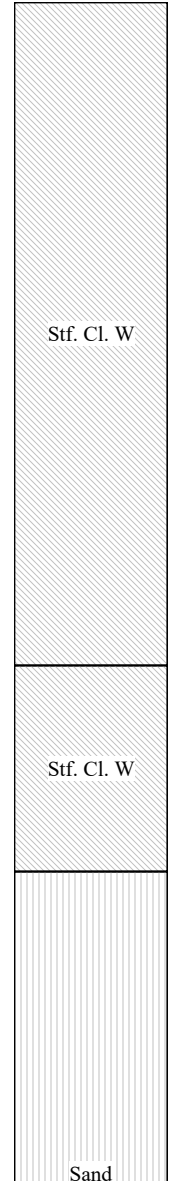
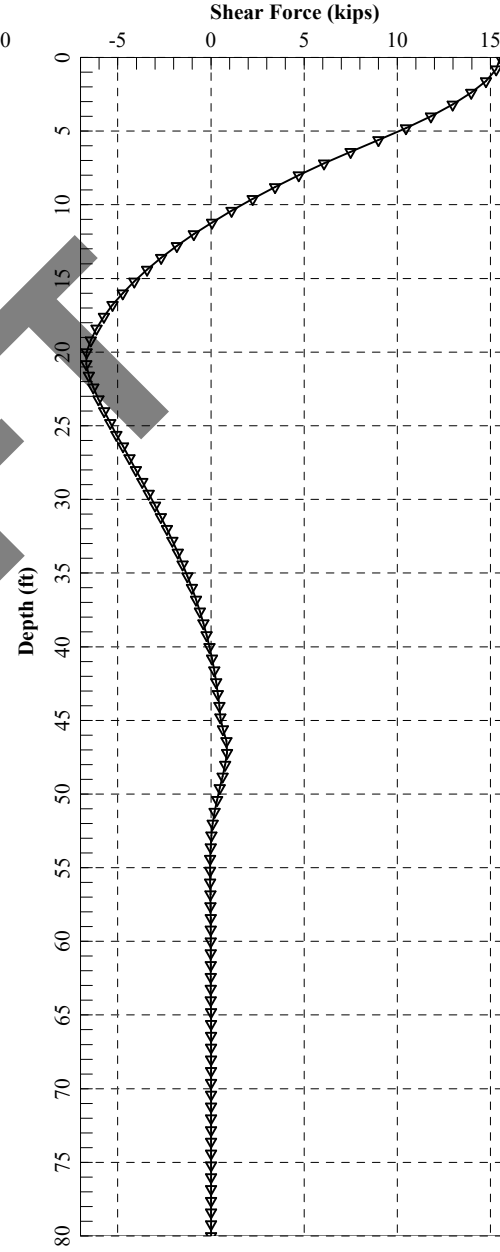
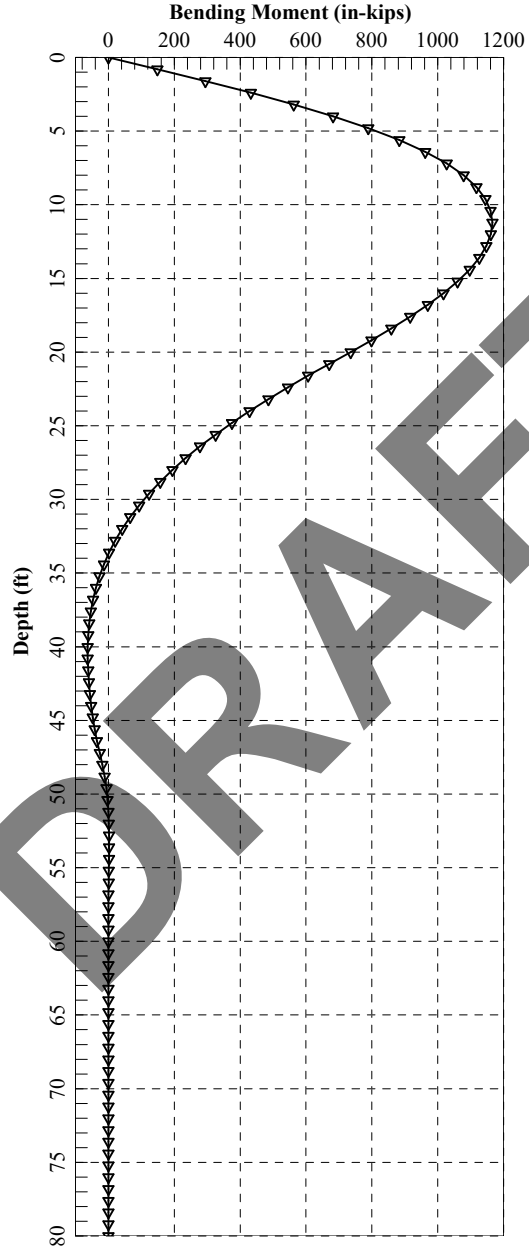
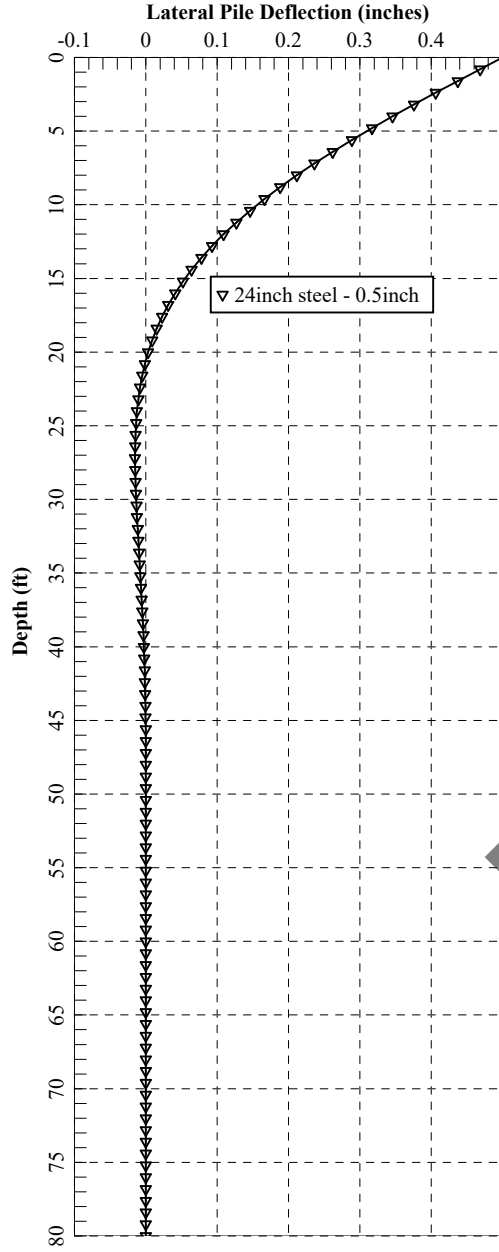
24-INCH STEEL PIPE

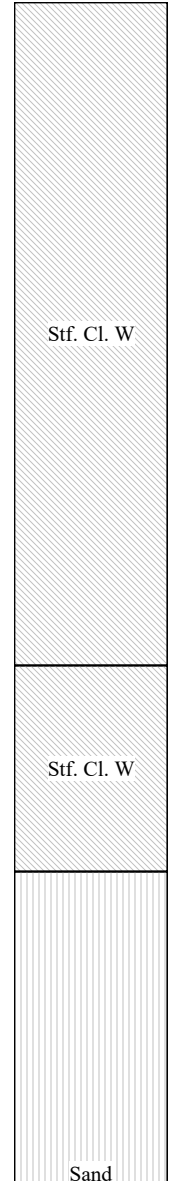
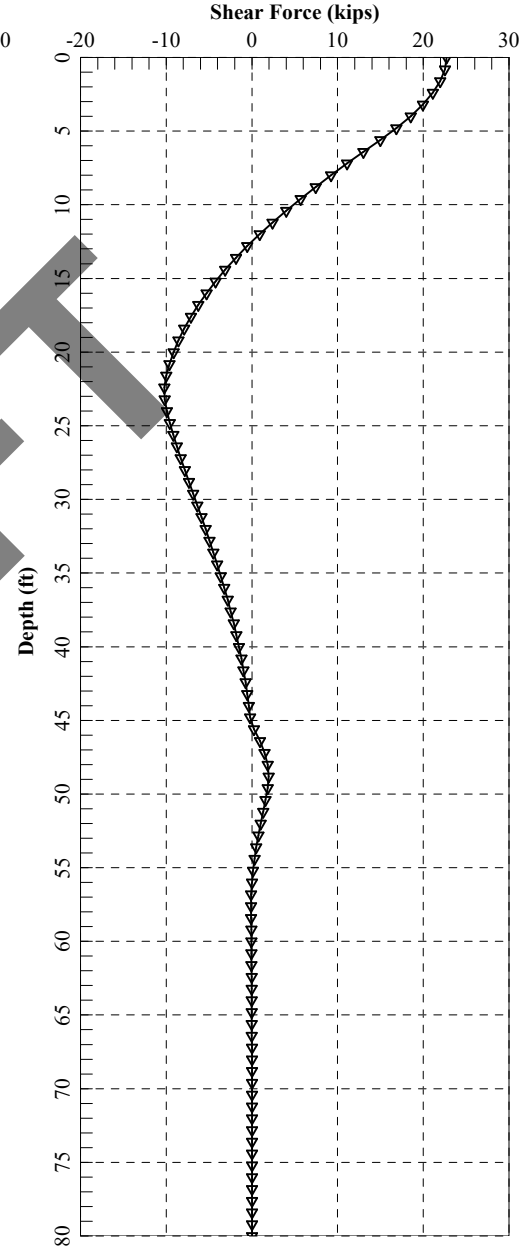
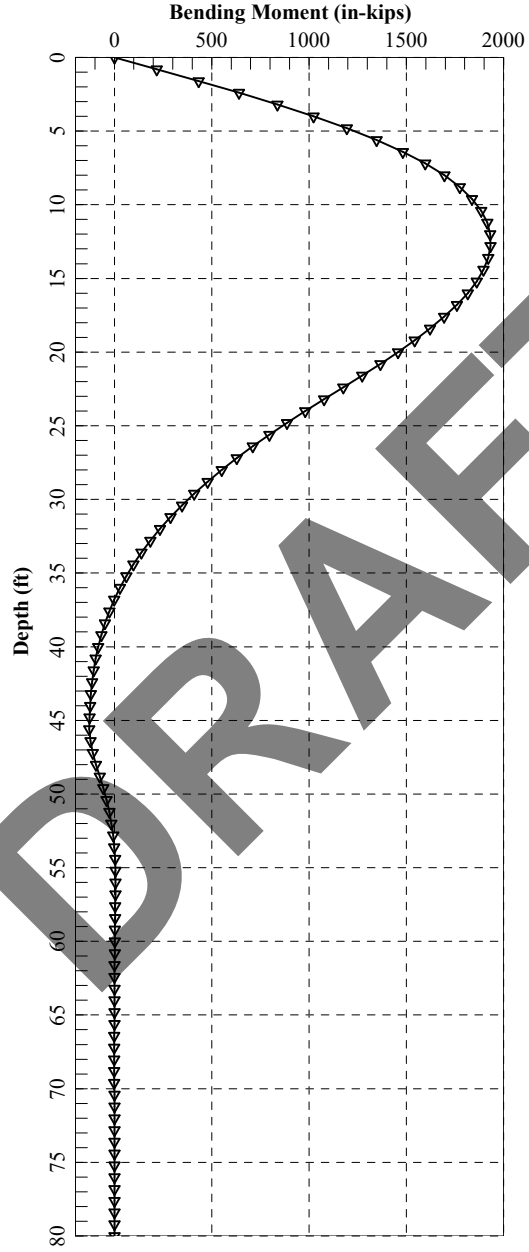
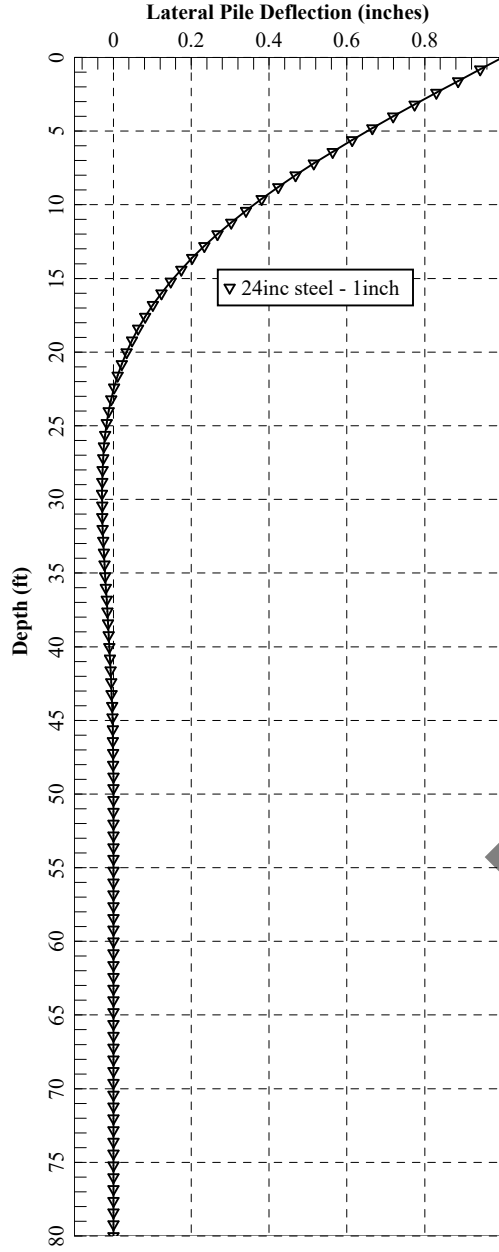
CLEAR CREEK

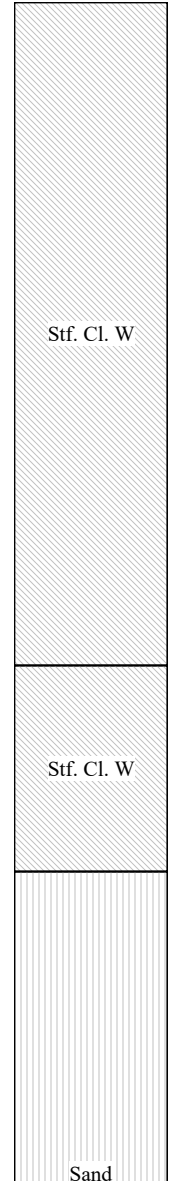
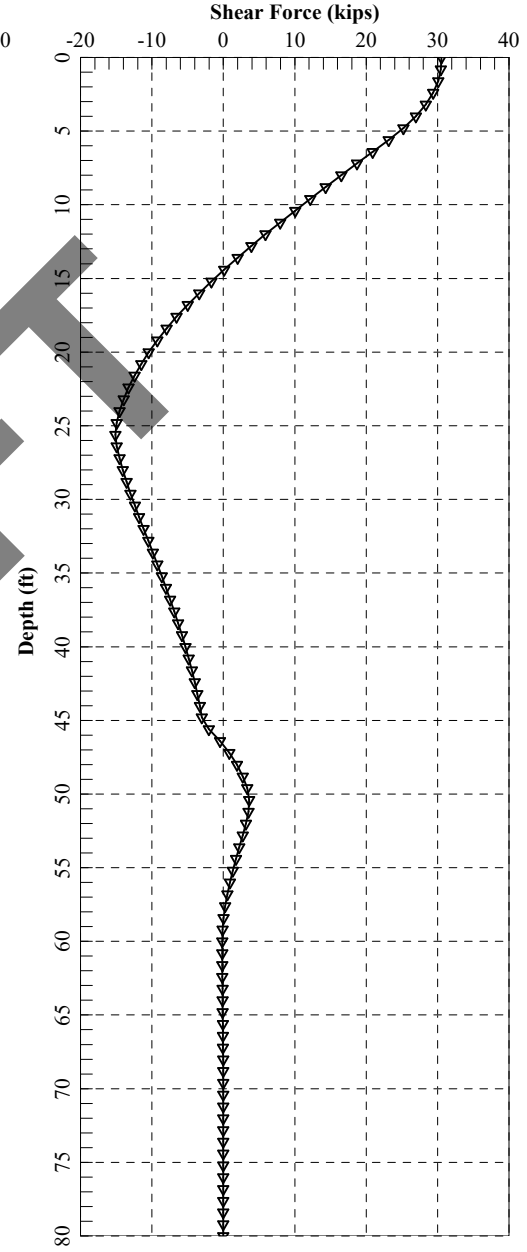
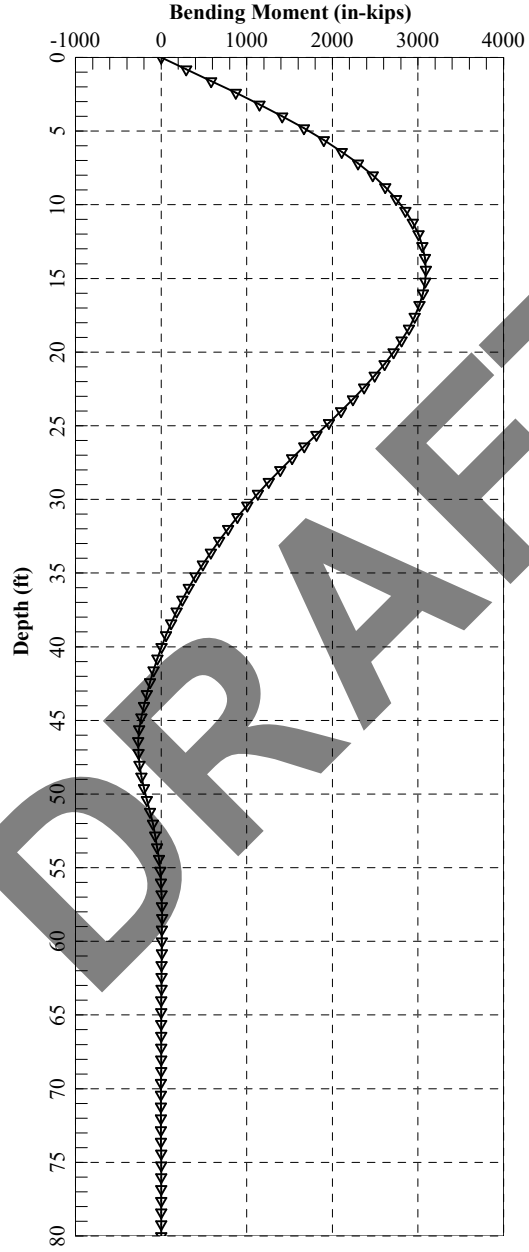
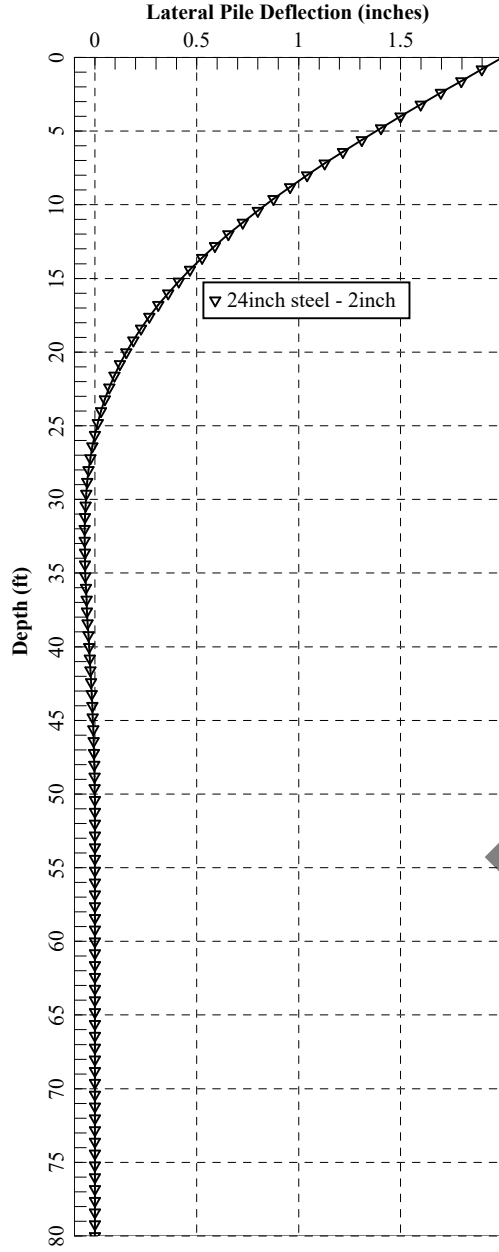
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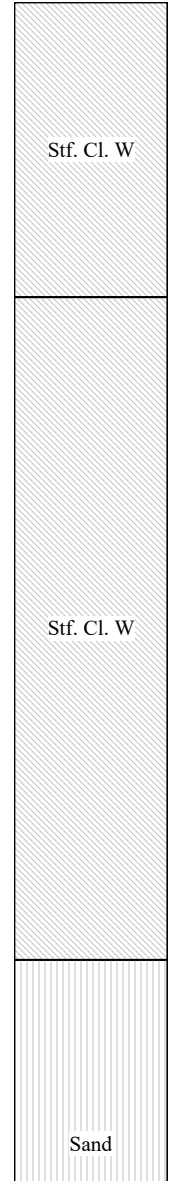
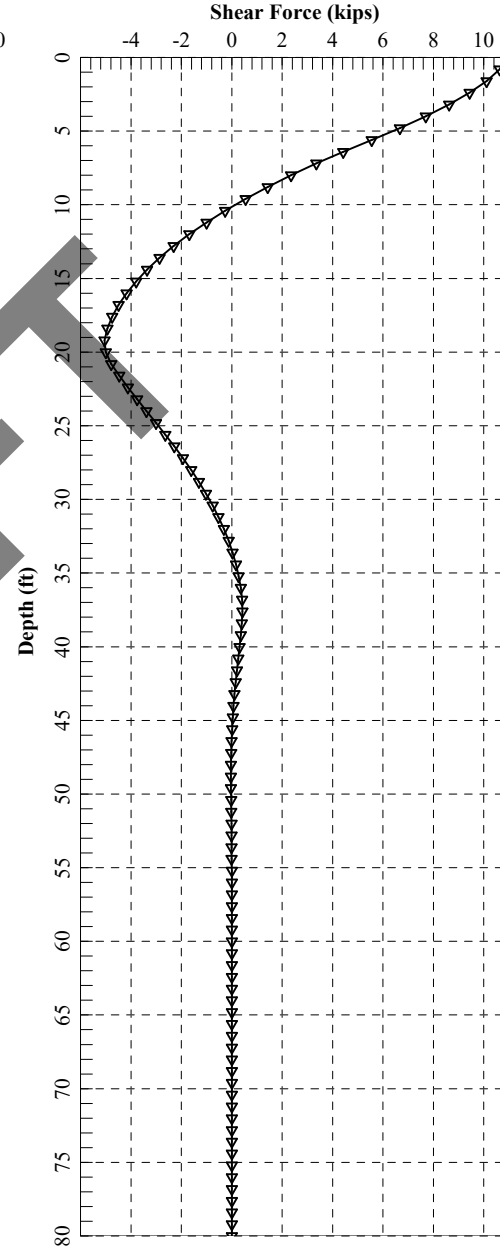
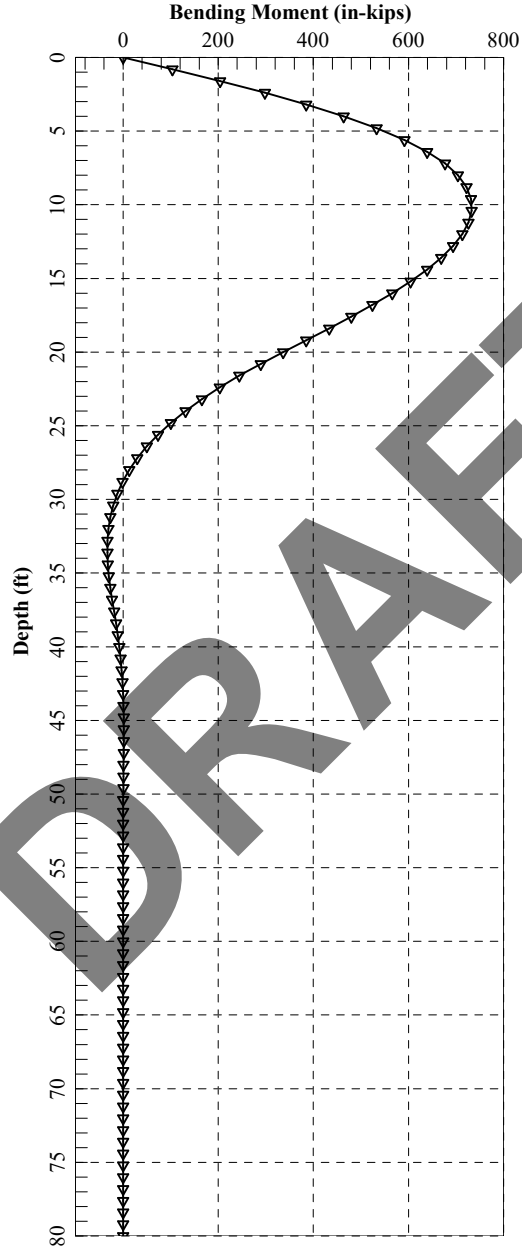
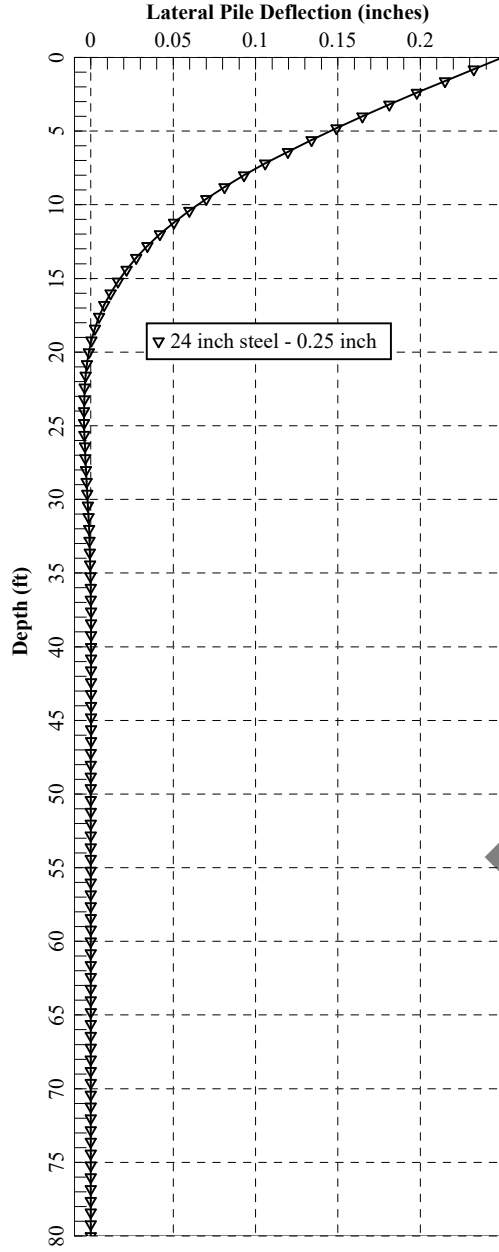


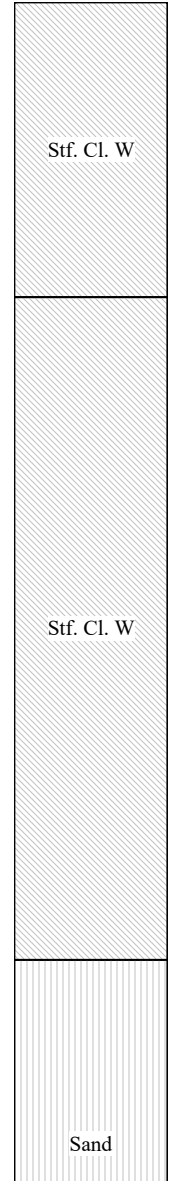
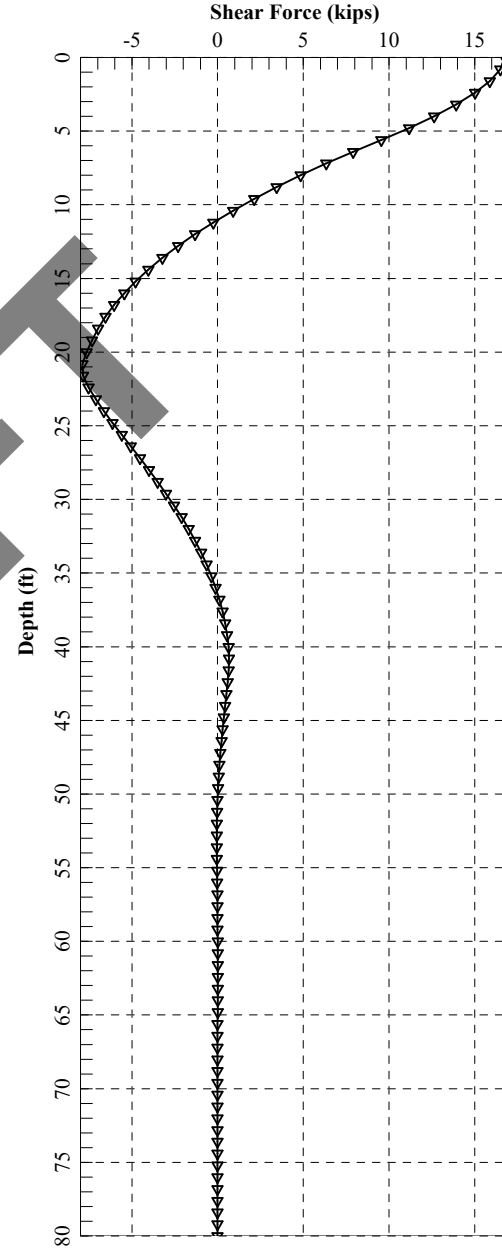
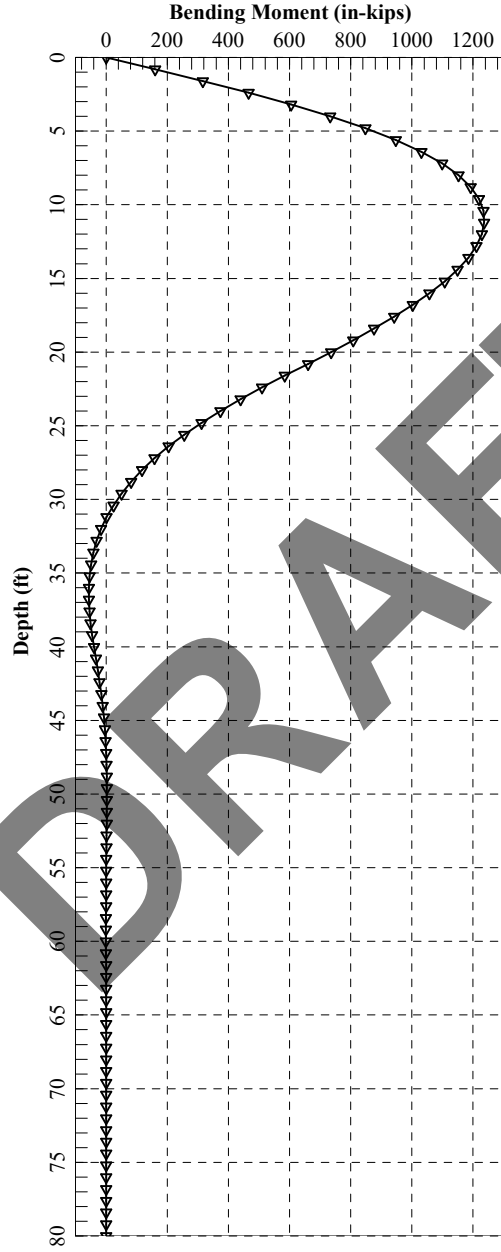
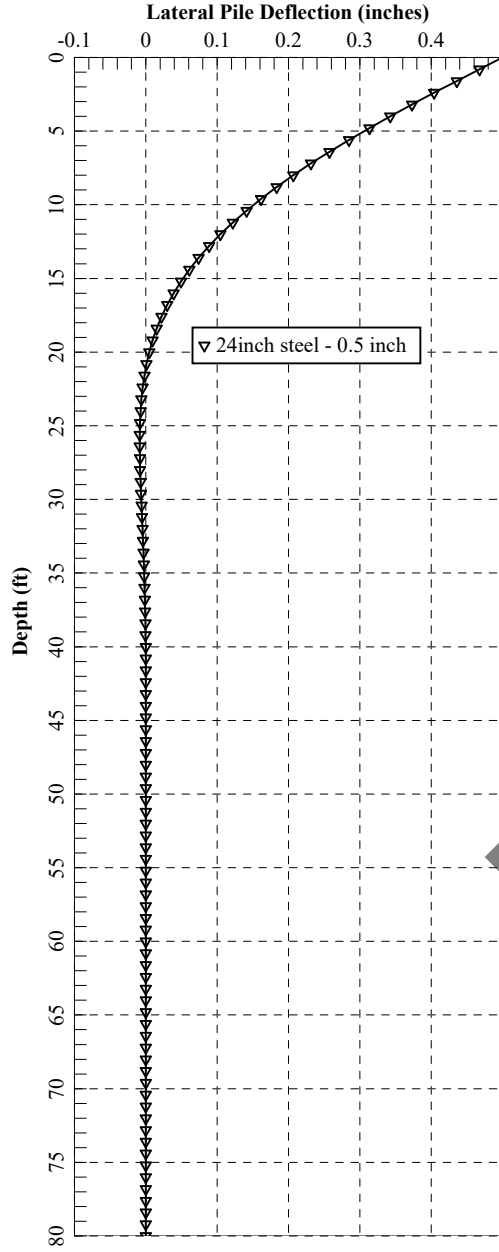
24-INCH STEEL PIPE

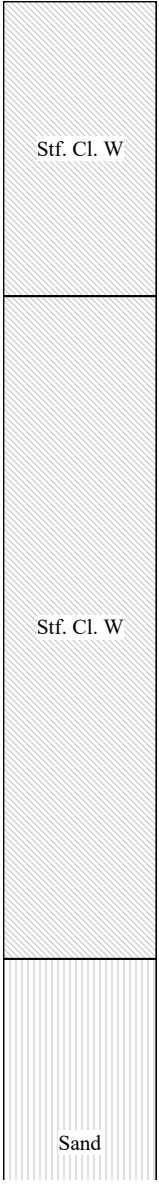
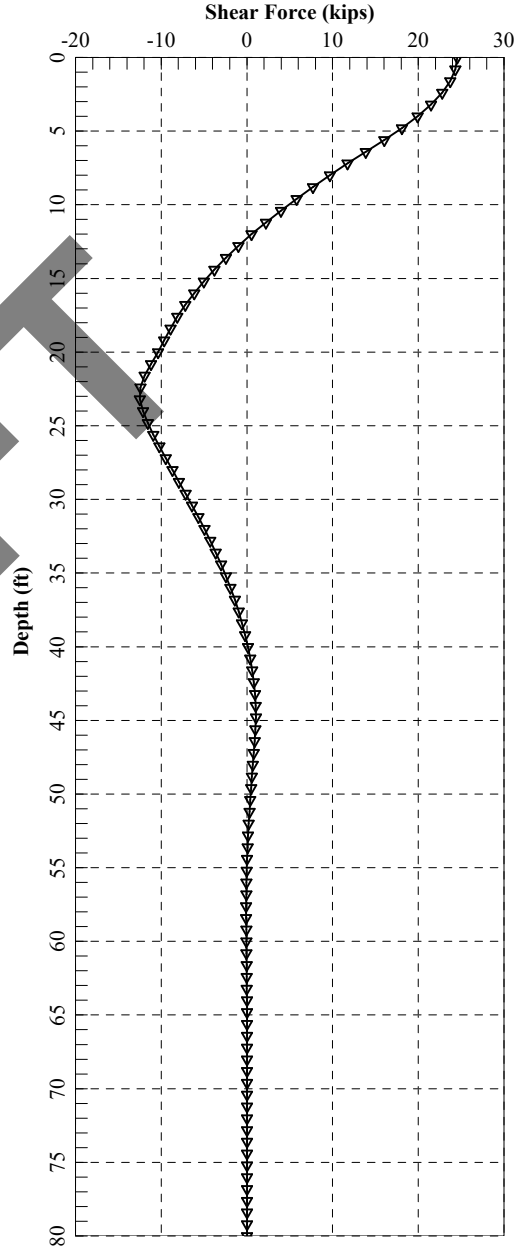
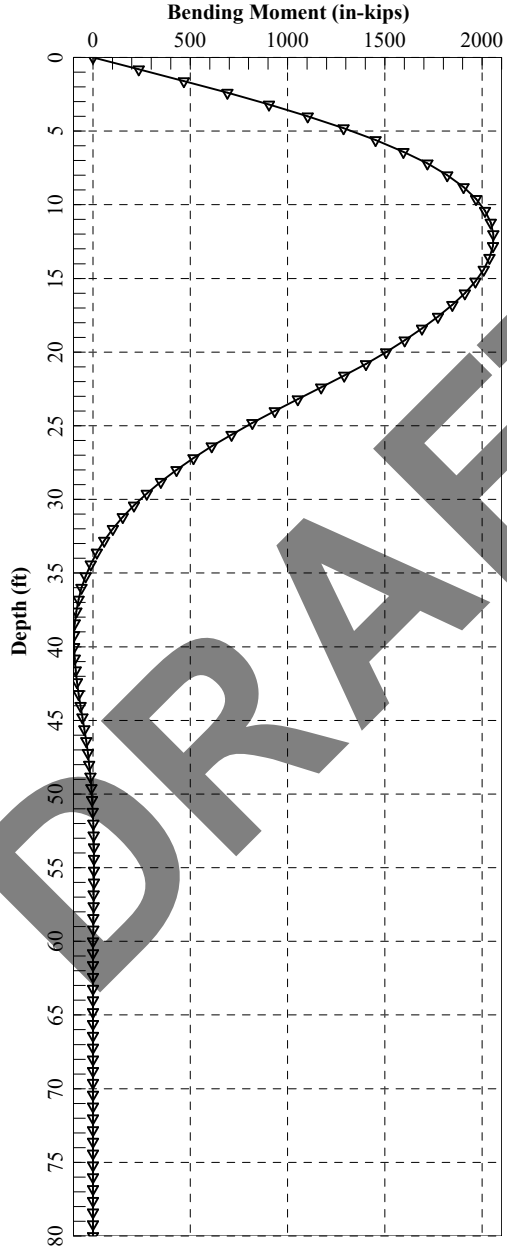
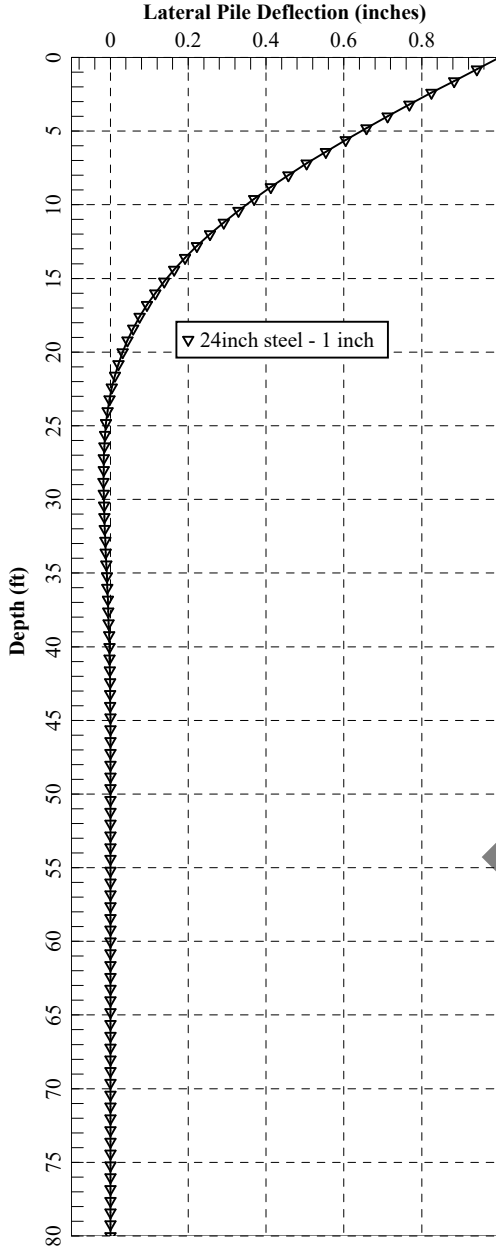
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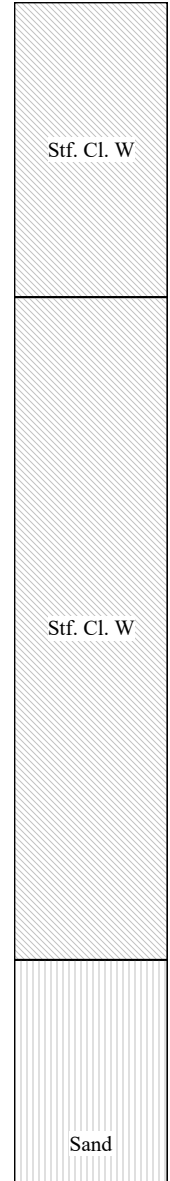
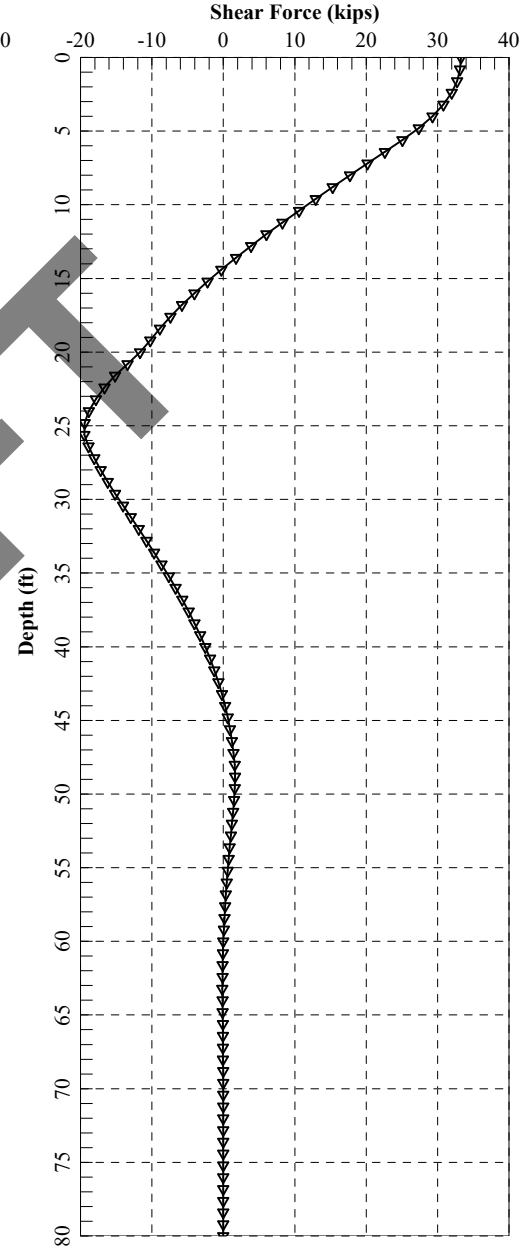
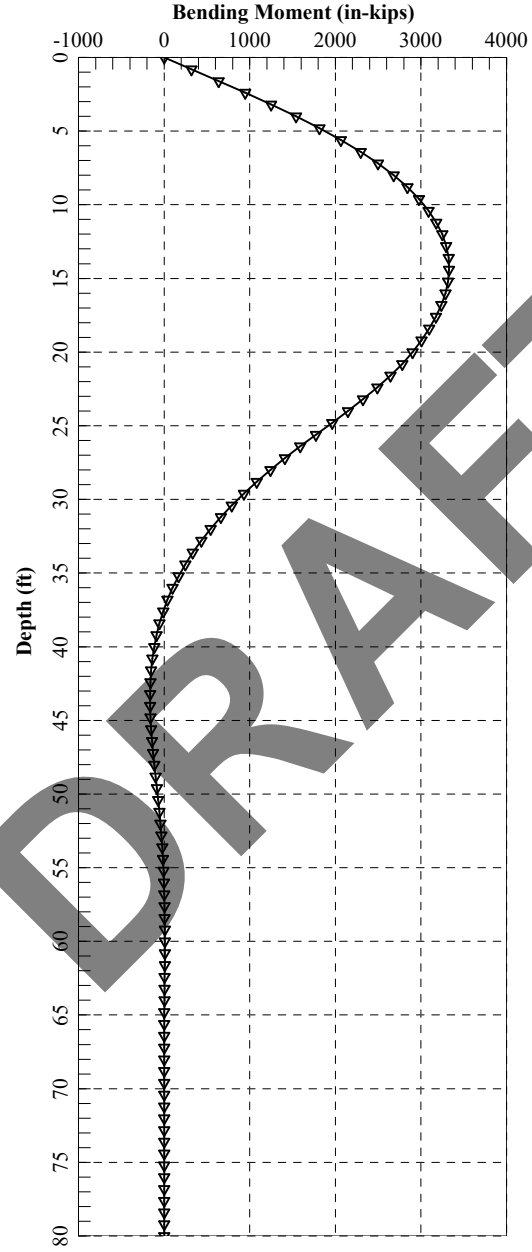
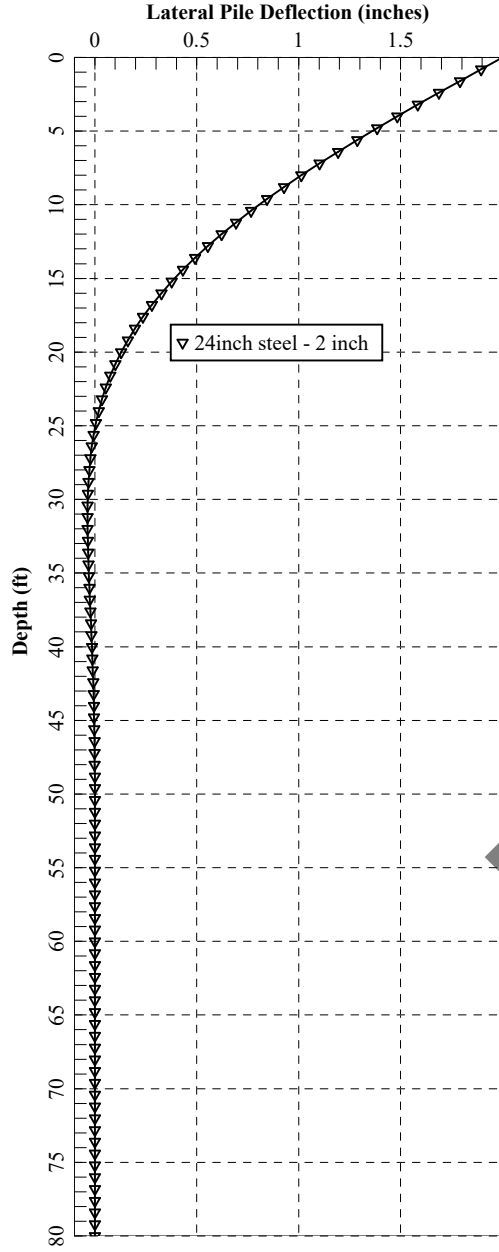
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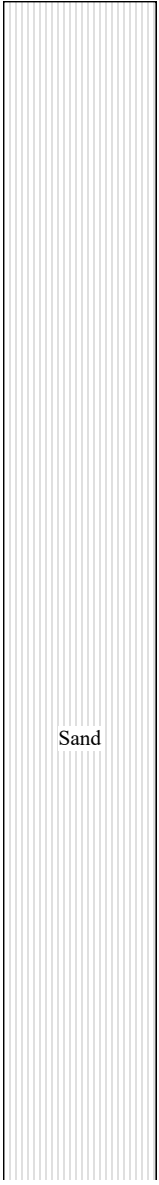
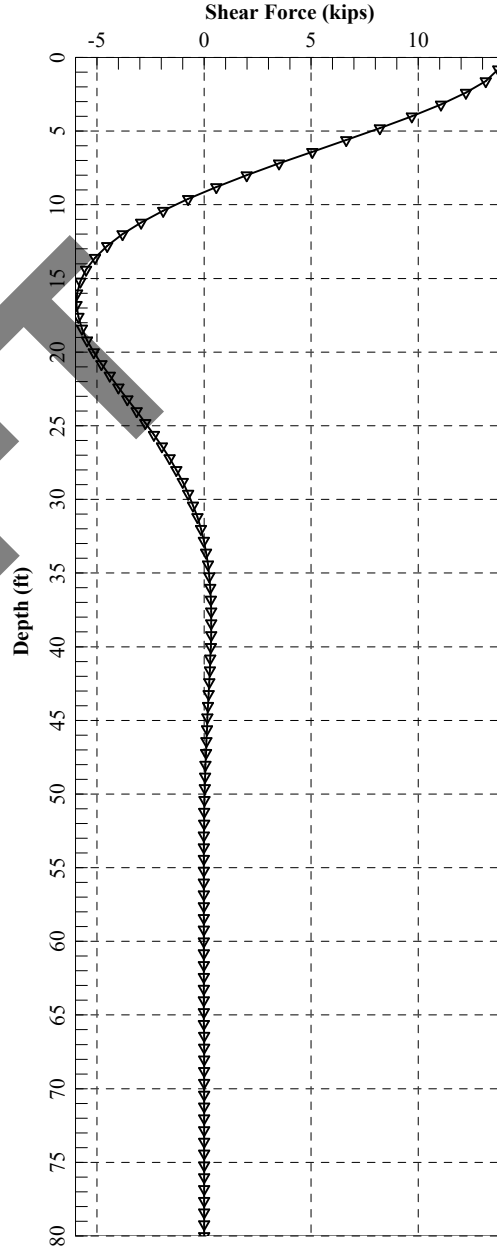
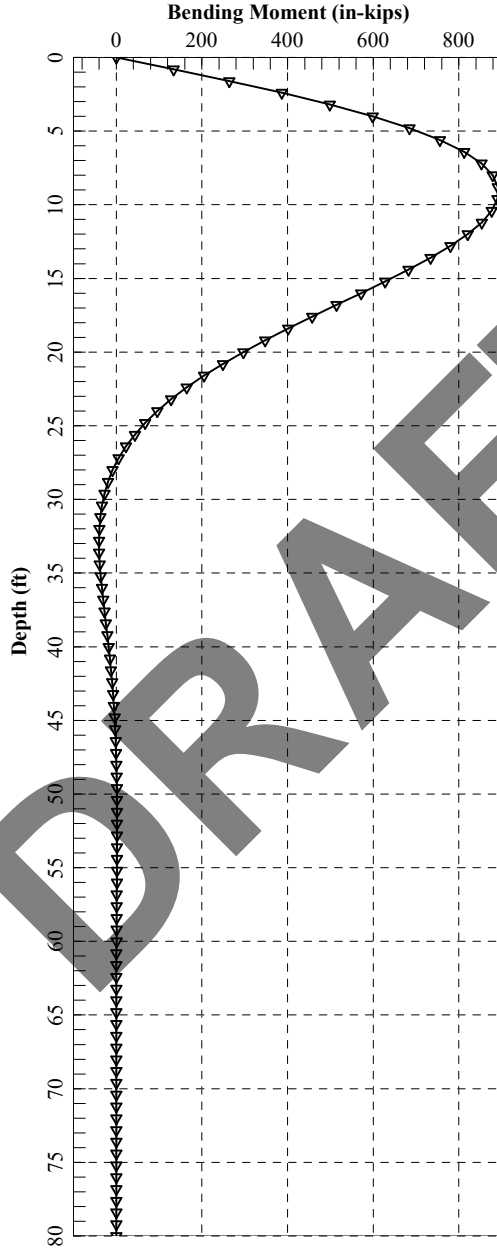
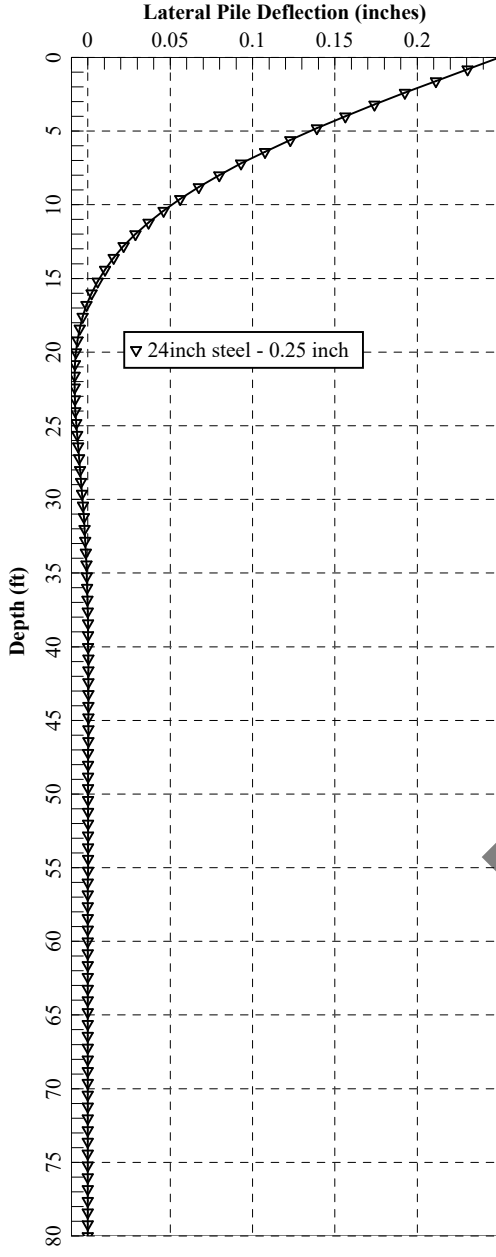


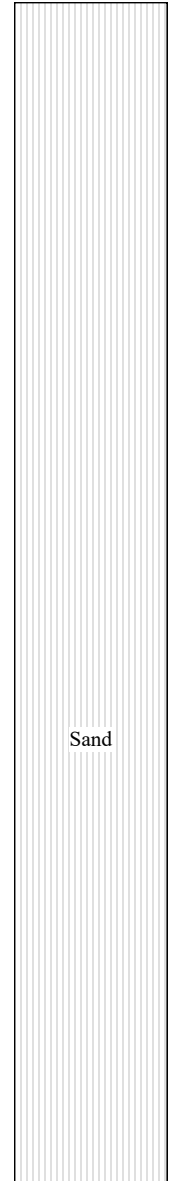
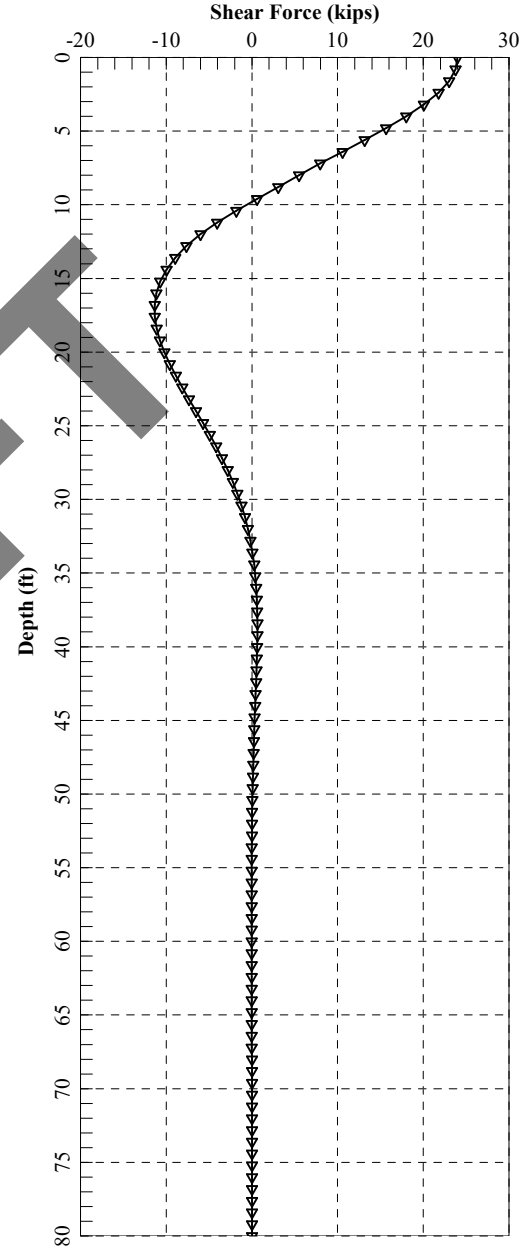
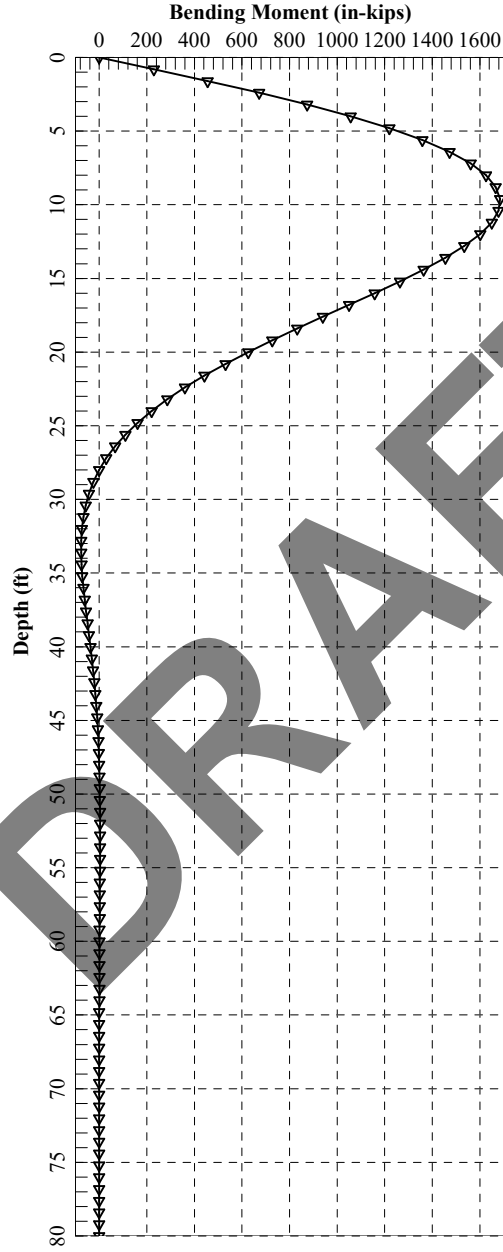
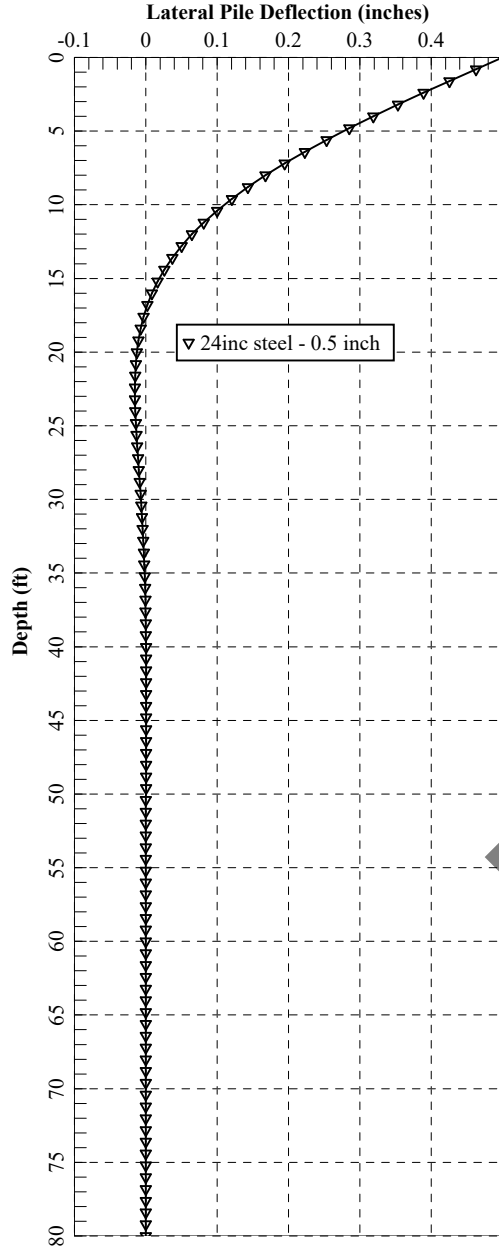
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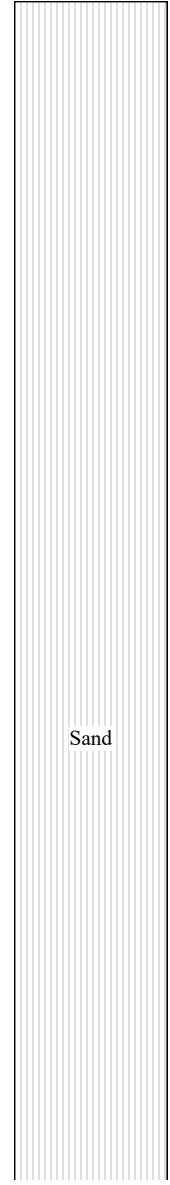
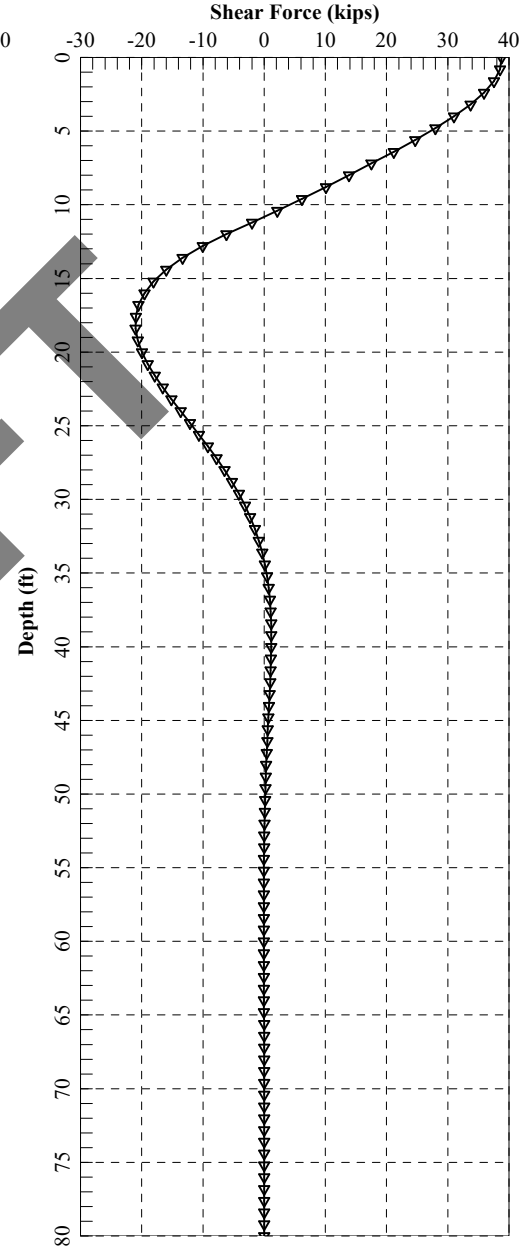
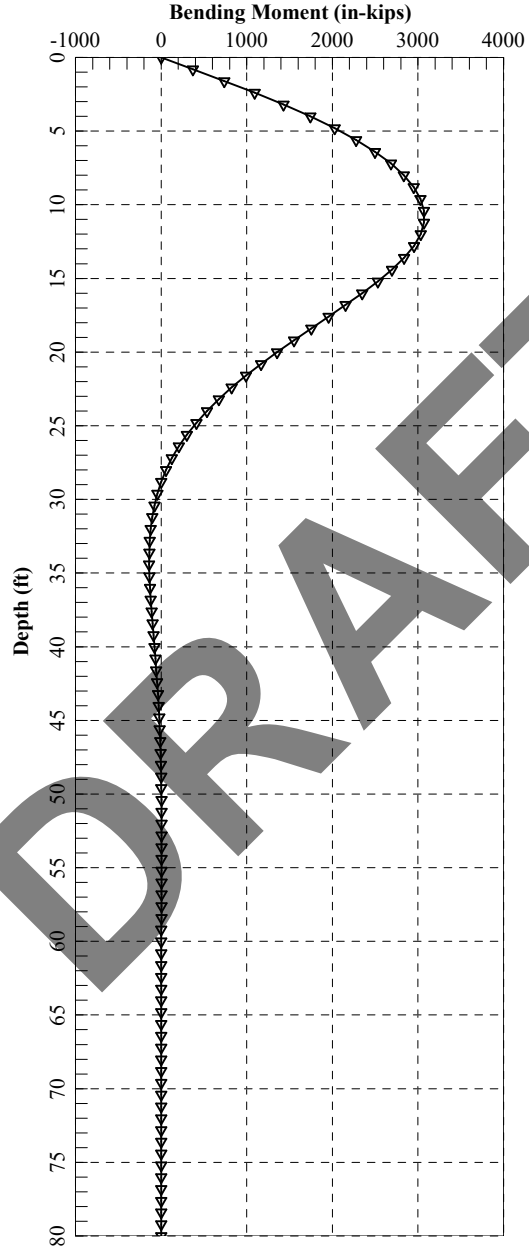
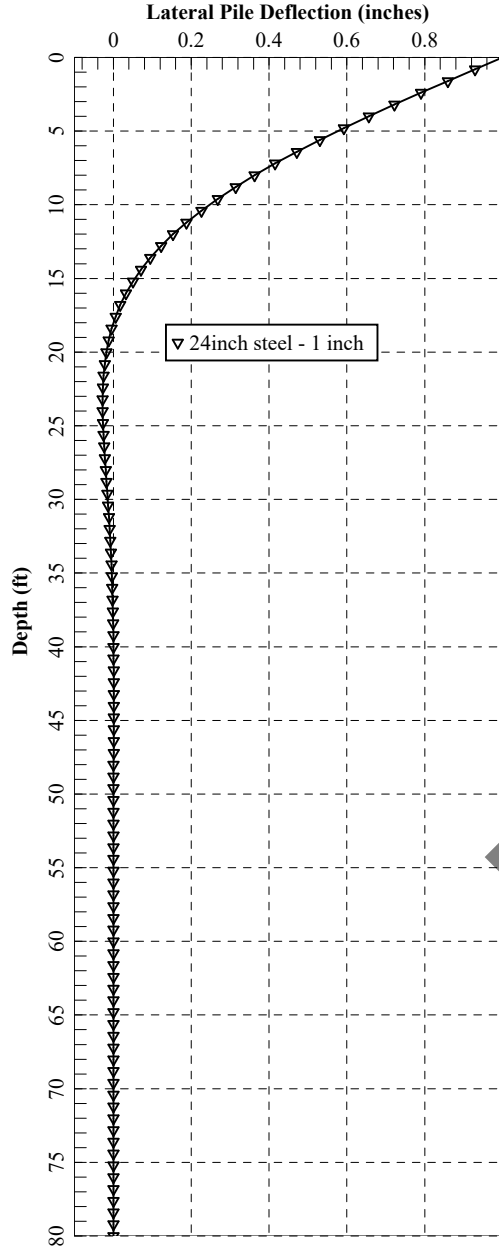
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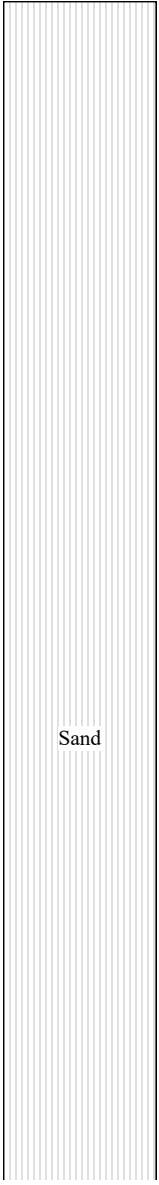
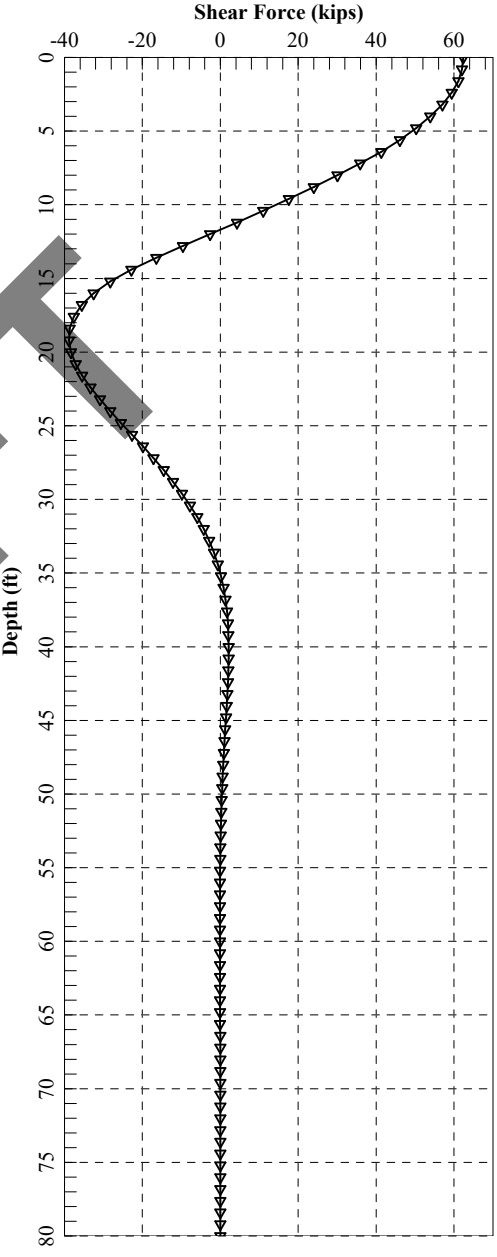
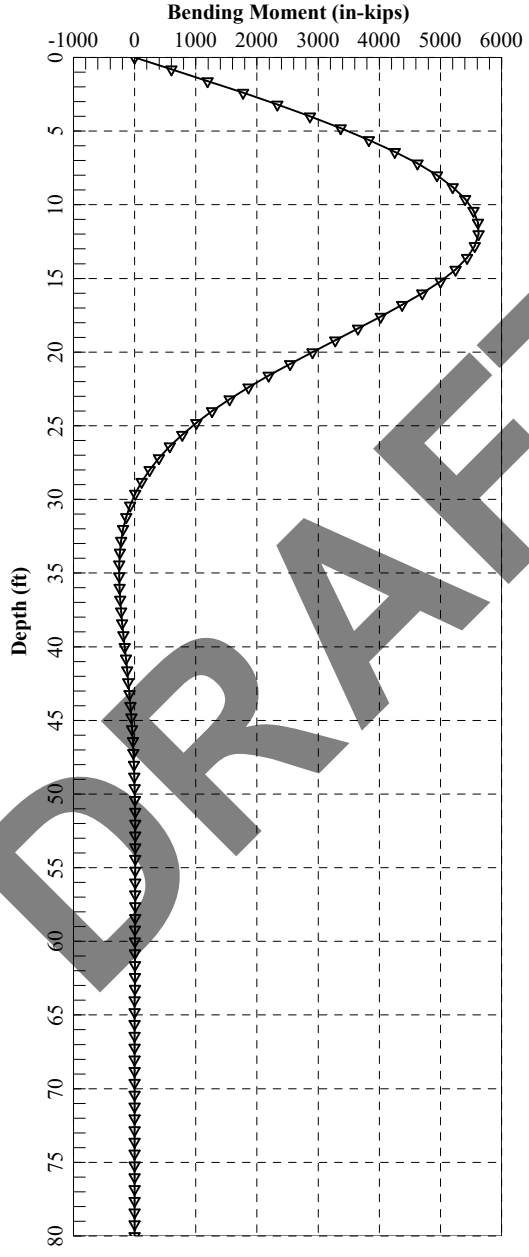
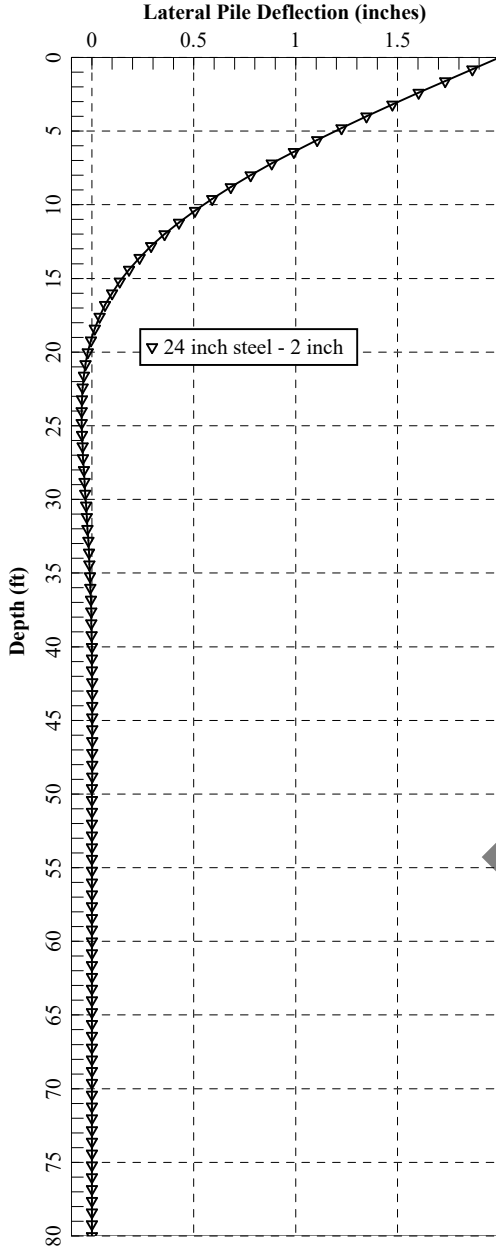
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Appendix C - Clear Creek and Dickinson Bayou Location Plans

C1-- Clear Creek Location Plan with Note Re Rte. 146 ROW

C2 -- - Dickinson Bayou Location Plan



Figure C-1. Clear Creek Location Plan with Note Re Rte. 146 ROW

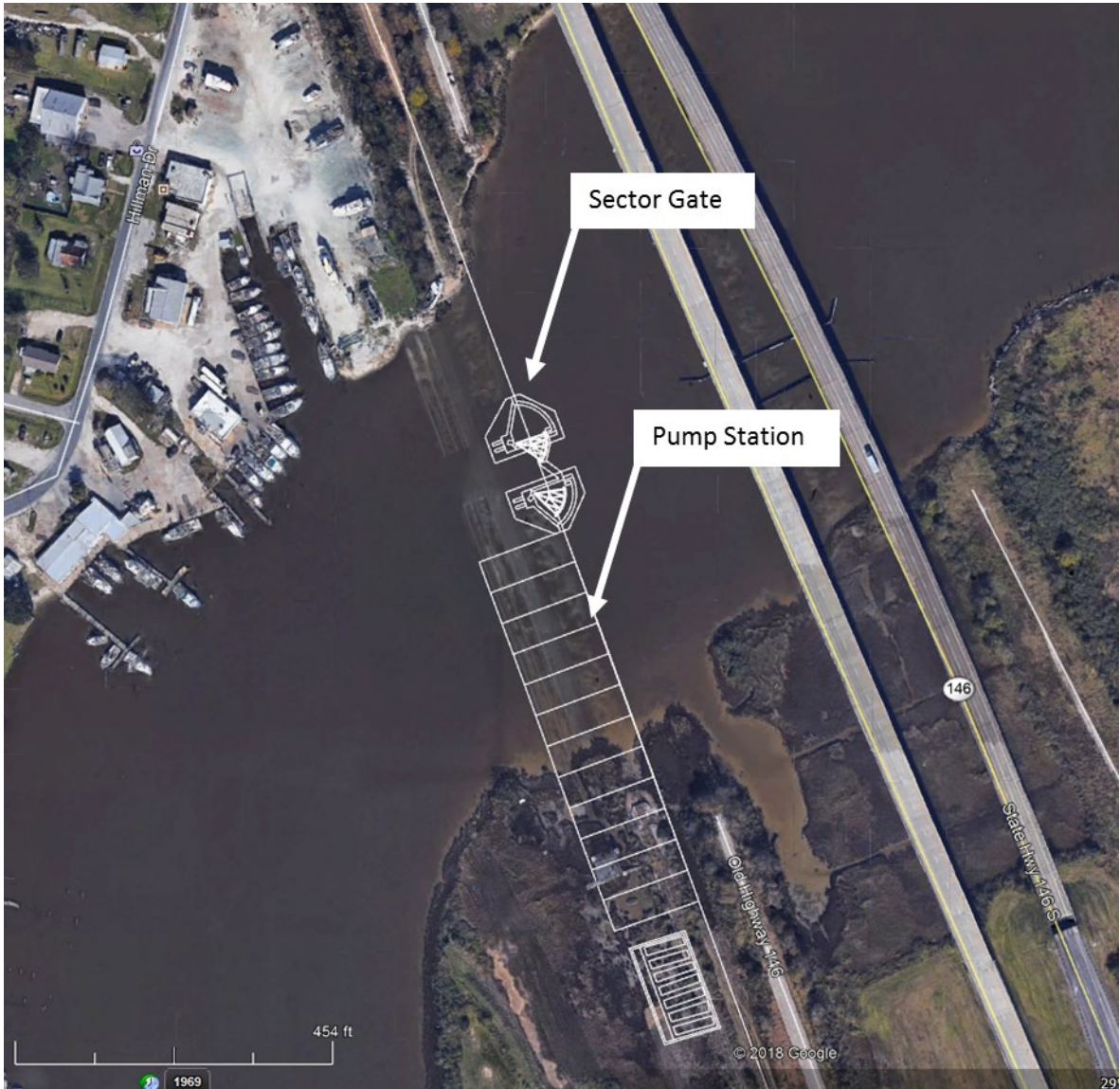


Figure C-2. Dickinson Bayou Location Plan

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Appendix D - Pipelines

D1 -- ROW Pipelines - Clear Creek

D2 -- ROW Pipelines - Dickinson Bayou



Figure D.1. ROW Pipelines - Clear Creek



Figure D.2. ROW Pipelines - Dickinson Bayou