DRIVEN PILES IN CENTRAL TEXAS EXPANSIVE SOILS

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Abstract: Expansive soils cause more damage to structures annually than a combination of other major natural disasters. Because of the cost to our society, all means and methods need to be fully explored to mitigate the problems associated with expansive soils. This study will present a foundation design approach that is underutilized in this application, driven piles. The main objective of the study is to present pile test results and analysis from four driven pile project sites in three types of expansive soils found in central Texas: Del Rio formation, Taylor/Navarro formation, and expansive alluvium. High strain dynamic pile tests were conducted on each of the four studies with rigorous signal matching analysis from the CAse Pile Wave Analysis Program (CAPWAP). Ultimate pile capacities ranged from 73 to 311 kips with an average of 61% of the total capacity coming from the pile shaft and were two to six times the structural capacity needed. Allowable loads calculated from Modified Gates dynamic formula best modeled allowable test results. Average unit skin frictions ranged from 0.50 to 4.71 ksf. Restrike pile tests of 1 to 17 days after initial driving reported 30 to 100% shaft capacity gain. All open-ended pipe piles driven produced soil plugs ranging from 4 to 14 feet thick. Small diameter, thick-walled, open-ended pipe piles reached penetration of twice the depth of designated zone of seasonal moisture change without problem. The observed production rate of the driven piles was on average 8 minutes, which implied daily production of 15 to 40 piles. Predrills or augered holes should be specified for underground obstructions found in soil investigation.

Introduction

Expansive soils cause more damage to structures annually than a combination of other major natural disasters. Because of the cost to our society, all means and methods need to be fully explored to mitigate the problems associated with expansive soils. This study will present a foundation design approach that is underutilized in this application, driven piles.

The motivation for this study stemmed from the lack of exposure central Texas engineers had to design of driven pile-supported foundations in expansive soils to overcome uplift of seasonal moisture variance. The principal objective of this report is to present the pile driving practice of central Texas and tested pile capacities with the hope that these results will be the basis for future design.

High strain dynamic pile tests were conducted on four projects involving driven piles in central Texas expansive soils. Every dynamic pile test had rigorous signal matching analysis from the CAse Pile Wave Analysis Program (CAPWAP). The results from these tests, along with pile driving observations, are documented and analyzed in this report.

Expansive Soil and Active Zone

Expansive soils are found in fine-grained cohesive soils such as clay and shale. Clays come in several different groups that are categorized by their mineral makeup. Expansive soils are associated with the clay group smectite. The smectite particles are thin sheets with a very high specific surface (surface area per unit mass) and a negative charge. The combination of the high specific surfaces and negative charges lead to significant interaction between the clay particles and ions in water, causing great volumetric change when water is added or removed (Millot, 1979 and Mitchell, 2001).

The zone of seasonal moisture change, also known as the active zone, is caused by evapo-transpiration from plants and sun, along with seasonal heating and cooling cycles and ground water changes. Regions with significant problems due to expansive soils tend to be semi-arid or arid in nature and therefore subjected to large seasonal differences of moisture content in the soil. The active zone is defined by the depth of wetting (Nelson, et al. 2001).

Central Texas Geology and Weather

The geology in central Texas ages as one goes east to west. From the east heading towards the Gulf of Mexico, the Black Prairies are composed of Taylor and Navarro clay formations which are comprised of highly expansive smectite clay. Further west, the Balcones escarpment is reached. This escarpment was the result of a crustal movement that caused approximately one thousand feet of uplift to occur during the Cretaceous Era and left behind fault lines throughout the area. Within the Balcones escarpment, there are alternating layers of clay or shale with limestone. Some of these clays are of interest because they are also smectite clays: the Del Rio and the Eagle Ford formations. Further west, older, harder limestone formations are found until the Marble Falls area, which has Precambrian granite formations. Within a hundred miles there are three changing types of geology. Rivers and streams crisscross all these formations. Over the years anything from cobbles to gravel to sand and silt, along with clay and limestone have deposited from weathering and transportation by water of the adjacent landscape (Flawn, 1970).

Texas weather is severe. When looking at the Palmer Drought Severity Index published by the Texas Water Development Board over the last century, cyclical drought and moist seasons can be seen (TWDB, 2011). Rain is infrequent; however, when storms come, they bring large amounts of precipitation. One of the biggest factors in these extreme storms is the Balcones escarpment, which has enough elevation gain from the Black Prairies to cause moist air from the Gulf to rise up and condense producing rainfall (Harmel et al., 2003). For instance, the Medina, Texas storm event of 1978 produced over 20 inches in 24 hours, and the Alvin, Texas storm in 1979 produced nearly 40 inches of rain within 24 hours (Slade, 1986). With an average annual rainfall in central Texas of 33 inches, the climate is semi-arid causing the clayey soils to dry out and to produce deep, surface cracks that allow water to further penetrate this relatively impermeable soil.

Engineering Approaches in Expansive Clay

As expansive clay interacts with water, the swell of the soil is not uniform. This non-uniformity leads to differential movements from one location to the next causing surface cracks. With the differential movement, slab failure is directly associated to aesthetics and operational use of the structure caused by substantial cracks in concrete. The need for engineering in expansive soils comes from an extensive history of foundation failures and litigation, leading to design based on an acceptable movement that is clearly communicated to the owner of the structure. Before design, site investigation gathers information needed for establishing the seasonal zone of moisture change and designing a deep foundation. The depth of the borings is based off local knowledge, and the number of borings varies by structure type. Many types of soil characterization tests are available with soil strength and swell potential tests being the most important. A range of engineering approaches use the soil characterization data for design to limit structure movement caused by the soil. The most conservative design approach is a voided structural slab supported by a deep foundation which is traditionally a straight or belled drilled shaft depending on the engineer (Department of the U.S. Army, 1983).

Driven Piles in Stiff, Over-Consolidated Clay

Driven piles in stiff clays have been tested since the 1950s which results Tomlinson used to develop the alpha method to statically calculate pile capacities given undrained shear strength. In his 1970 paper, Tomlinson observed pile set up in stiff clay that increases the capacity ranging from 20 to 200% over weeks after driving. He also observed surface cracks during pile driving in stiff clay caused by pile movement and soil heave which could allow water to travel. Tomlinson was concerned with the potential of water softening the soil, but in expansive soil, this could cause additional uplift loads from the volumetric change of the soil (Tomlinson 1970).

Literature rarely references driven piles as an engineering approach in expansive soils. Work is currently being done in China to numerically model soil-pile interaction and field research was conducted in the USSR from the 1960s to the 1980s. The numerical modeling recommends pile penetration of 2.5 times the depth of seasonal moisture change with a maximum pile diameter of 4.5% the embedment depth (Xiao et. al., 2011). The field studies report 5 pile diameters of penetration into inactive soil reduce uplift forces by 50% (Doroshkevich and Boim 1967).

Current central Texas design practice has several examples of a driven pile deep foundation approach. This is motivated by high water tables and site access resulting in reduced costs for the engineer's client. Majority of these examples did not have geotechnical report recommendations for driven piles.

Central Texas Pile Driving

Central Texas pile driving practice consists of a lattice boom crane supporting a box set of leads guiding a Pile Master 36-3000, a relatively small air hammer. Experience has led to the use of thick-walled, small diameter, open-ended pipe due to availability and economics. However, HP 10x42 and 8 inch square prestressed, precast concrete piles have been driven. Piles are driven in less than 20 minutes with an 8 minute averaged installation time implying daily production rate of 15 to 40 piles installed. Predrills are useful if underground obstructions are found in soil investigation. The following four case histories will expound on driving piles in expansive clay.

Manor

The first project location was east of Manor, Texas on Highway 290 where the test piles support a commercial development's entrance sign. Currently there are three buildings constructed in the development, one having a structural slab supported by drilled shafts and the other two having a structural slab supported by driven piles with allowable pile design compression loads of 20 to 110 kips. The dynamic pile tests were solely for demonstration purposes sponsored by Signor Enterprises (now TX Pile) and conducted by Frank Rausche of GRL Engineers in July 2009.

The geologic formation in the area was Taylor formation. Soil investigation consisted of six borings that were 25 feet deep. The upper stratum was dark gray and olive-brown, very stiff fat clay having a thickness of 4 to 8 feet, moisture content of 28 to 35%, liquid limit (LL) of 94 to 100% and a plasticity index (PI) of 63 to 74. The lower stratum was light yellowish brown (tan) and light to medium olive-brown, hard fat clay having a thickness of 17 to 21 feet, pocket penetration (PP) test result of over 4.5 tsf, moisture content of 18 to 25%, liquid limit (LL) of 76 to 92, and plasticity index (PI) of 56 to 67. Given the PI of this clay, the potential vertical rise (PVR) was 7 inches and the geotechnical engineer recommended 11 feet of soil removal and replacement of select fill to have a PVR below 1 inch.

Two schedule 80 (0.432 inch wall) 6-5/8 inch open-ended pipe piles (MN1 and MN2) were tested along with a HP 10x42 pile (MN3). During driving of the production piles for one of the adjacent buildings, an experiment on pile tips was conducted to determine if there was any difference in speed of driving or final set for a plated end of ¾ inch steel, steel conical tip, or an open end. The experiment found that all three piles had approximately the same final set and the same driving time of 5 to 8 minutes. For the dynamic test, the open-ended pipe pile was chosen for driving because of cost and time savings. As a rule of thumb, a plated end cap for a pipe pile costs about the same as one to two linear feet of the pile itself which can increase with the diameter of pipe. A conical tip is four to six times the cost of a linear foot of pile. Soil plugs ranging from 5 to 9 feet were measured for the two buildings.

Soil setup was of interest for this test, so one of the 6-5/8 inch pipe piles was driven 6 days before the test to be restruck. The other pipe pile and H-pile were tested at the time of driving. Driving time for the test piles ranged from 8 to 10 minutes. The final set for the piles ranged from ½ inch to 1/10 inch.

Tarrytown

A pile supported slab was designed by the structural engineer for a two-story residence located in Tarrytown, Austin, Texas. Although driven piles were not recommended by the geotechnical engineer, the structural engineer adopted them because of concerns about variability in the subsurface and the cost of cased drilled piers compared to driven pipe piles. The final foundation design had 79 piles with a maximum allowable design load of 55 kips per pile.

This project was located in a formation of Upper Colorado River terrace (Qucr) deposits which range from silt to clay soils to sand and gravel and underlain by the Del Rio formation (Kdr) of clay. Four, 30 foot borings were drilled to investigate the site. Three strata were observed in the borings. An upper stratum of brown, silty, medium stiff to stiff fat clay of Qucr having a thickness of 5 to 7.5 feet, moisture content of 21 to 27%, LL of 58 to 69% and a PI of 38 to 49. A middle stratum of reddish brown and tan, medium stiff clayey sand with small to medium sized gravel of Qucr having a thickness of 3.5 to 11.5 feet, moisture content of 13 to 15%, LL of 50 to 55% and a PI of 31 to 35. A lower stratum of greenish tan and gray, jointed, stiff fat clay of Del Rio formation (Kdr) having a thickness of 12 to 18 feet, moisture content of 23 to 25%, LL of 62 to 65% and a PI of 42 to 45. Water was found on half of the site from 12 to 14 feet and was physically observed in approximately 20% of the predrills and associated to TT4 and TT5. Figure 1 shows the SPT N-values of the soil with depth from the four borings. The upper strata were weaker at 10 blows per foot versus the deeper soils that averaged around 25 blows per foot. On boring 1 at 15 feet, the values went up to 39 blows which could be due to a dense layer of sand and/or gravel there.

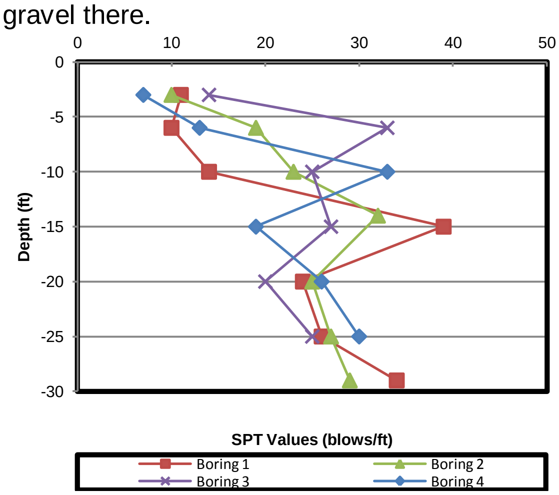

Figure 1: Tarrytown SPT N-Values with Depth.

Webberville

Piles driven in Webberville were utilized to support lightly loaded energy collectors and transformers for a solar farm. The individual point loads for the piles were 4,000 pounds each, however the main concern was soil uplift. The original design was for to 12 to 15 inch diameter pipe. From the previous case histories of Manor and Tarrytown the recommendation to use smaller diameter pipe was made. All piles for these structures were designed for a 20 foot penetration with axial capacity confirmed by dynamic pile tests. An additional 12-3/4 inch diameter pipe with 0.250 inch wall was tested to compare any size affect.

The project site was investigated with more than 30 borings to delineate where the transition was from lean to fat clays due to the active zone differences and the necessary pile embedment depth. The lean clay was found to be up to 12 feet in depth and layered with alluvium and sand deposits designated with an active zone of 7 feet and a PI of 19 to 31 in this stratum. Test piles WB 1 and WB2 were located in the middle of the lean clay area. The fat clay was found to have an active zone of 10 feet with a PI of 47 to 56. The majority of the borings close to test piles WB3, WB4, and WB5 were completely comprised of fat clay, however the closest borings showed layering of lean clay and silty sand along with the fat clay.

For all the test piles and the 152 production piles driven on the site, the typical driving time was 3 to 5 minutes and approximately 8 minutes from the start of driving one pile to the start of driving the next. This production rate led to completing 8 piles in a little over an hour for each area. The site consisted of 19 different areas over the 400 plus acres. Due to the distance between areas, 32 to 40 piles were driven per day.

At the time of driving, the soil was very dry, leading to additional cracking as the piles were driven into the ground. This experience is similar to that described in Tomlinson's papers, where a pile goes down through stiff clay; it breaks up the clay and has localized heaving. However, with the open-ended pipe the heaving was kept to a minimum.

When comparing the two sizes of pipe tested, the main difference was the amount of plugging that was observed. There was a 16 foot plug for the 12-3/4 inch pipe versus an 8 foot plug for the 7-5/8 inch pipe. The plug for the larger diameter pile traveled far enough to be visible. Another observation was the belling of the 12-3/4 inch pipe caused by its wall thickness being too thin.

San Marcos

This pile test was conducted to compare small-sized precast concrete piles with pipe driven in previous cases. At the time of driving, there were two concrete piles, to be driven to 25-foot embedment and 35-foot embedment, and then two 7-5/8 inch by 0.375 inch wall pipe piles driven to 25-foot embedment and 12-foot embedment.

There were three main strata from the four borings of 25 and 50 feet in depth. The first stratum was very thin of 1 to 4.5 feet of lean clay. The PI was 47, and moisture content was 23 to 30 percent. The second stratum, presumably the Taylor or Navarro Formation, was fat clay that ended at 45-feet with the moisture content leveling out to 15 percent about 10 or 12 feet. The deepest stratum was gray shale that had a PI of 34. The unconfined compression strength for the boring averaged 5 ksf for the first 20 feet, then 10 ksf to 30 feet, where it increased to harder clay and shale shown in Figure 2.

The 31 and 22 foot pipe piles were driven in 8 and 3 minutes, respectively. In the longer pipe pile, a plug of 10 feet was recorded. Also, an inch and a half gap between the soil and pile was observed at the surface caused by leaning the pile in the leads for leveling purposes.

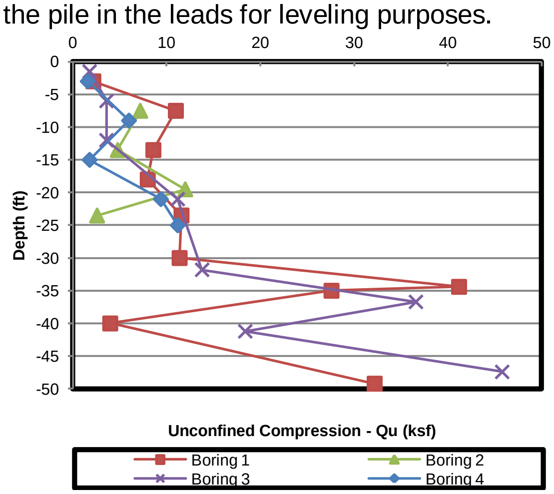

Figure 2: San Marcos Unconfined Compression Values with Depth.

The 40 and 30-foot long concrete piles were driven in 40 and 20 minutes, respectively. The 40 foot concrete pile only reached 23'8" of penetration before the pile cushion failed and the head of the pile fractured. Smoke was notably coming from the plywood cushion before failure. Also, the unsupported length of the 40 foot pile showed a lot of flex during driving. However, the 30-foot concrete pile drove to its tip penetration depth of 25 feet without problem. The assumed reason for this failure is that the hammer was too small resulting in hard driving of more than 80 blows per foot for a long duration of time. This hard driving fatigued the 6 inch thick plywood cushion and the concrete failed once it struck the steel strike plate of the hammer.

Result Comparison

The results of the dynamic pile tests are presented in Table 1. Overall, the axial pile capacity during driving ranged from 108 to 222 kips in the Taylor Formation, 120 to 311 kips in the Del Rio formation, and at the Webberville site ranged from 73 to 153 kips. The tested values were 2 to 6 times the needed allowable capacities for the structures.

The shaft friction was on average 61% of the total capacity, and ranged between 60 and 70% for the majority of the test piles range. The piles driven in Manor, Texas in the Taylor formation were averaging 90%. The average toe bearing pressure for all of the piles was approximately 178 ksf, with a range from 42 ksf for the 12-3/4-inch diameter Webberville piles to 230 ksf for the close-ended pipe piles that were driven in the Del Rio clay. All test results can be found in Table 1.

Both total stress analysis, or α-method, and effective stress analysis, or β-method, were used to analyze the pile capacities for the given soil information. For the α-method, if unconfined compression strengths were provided, undrained shear strength (ksf) was determined by dividing by two; if SPT N-values were provided, then undrained shear strength (ksf) was determined for each value by dividing by 8.

The adhesion factor, or α, was estimated using the undrained shear strength and an adhesion factor graph with values that ranged from 0.27 to 1.0. For the β-method, the internal friction angle was estimated at 30 degrees for all stiff clay found at all sites; therefore the β-factor was 0.4 and N_t was 30. Overall, both the α-method and the β-method under-predicted ultimate pile capacities, on average, by 50%. Comparatively the average side shear values for the α-method only under-predicted by 15%.

Table 1: Summarized Site and Test Pile Information

Test Pile Description			Soil Information			Pile Information			Driving Information								CAPWAP Results			Result Breakdown				
																				Average Unit Skin Friction				
Test Location	Test Pile Label	Day of Test	Soil Formation	Estimated Depth of Active Zone	Pre-Drill Depth	Pile Type	Pile Size	Pile Wall	Test Type	Pile Penetration	Blows per Inch (max= 100)	Observed Hammer Energy	Measured Hammer Energy	Hammer Efficiency	Max Compression Stress	Max Tension Stress	Total	Shaft	Toe	Total Shaft	Active Zone	Inactive Zone	Average Toe Bearing Pressure	% Skin Friction
				(ft)	(ft)			(in)		(ft)		(ft-lbs)	(ft-lbs)	(%)	(ksi)	(ksi)	(kips)	(kips)	(kips)	(ksf)	(ksf)	(ksf)	(ksf)	(%)
Manor	MN1	0	Taylor	12	NA	O. Pipe	6-5/8"	0.432	EOD	22	2.0	9000	6200	69%	26.1	-3.22	108	98	10	2.57	3.55	1.4	42	91%
Manor	MN2	6	Taylor	12	NA	O. Pipe	6-5/8"	0.432	EOR	22	10.0	9000	6600	73%	39.1	-3.18	215	180	35	4.74	4.04	5.58	147	84%
Manor	MN3	0	Taylor	12	NA	H Pile	10x42	NA	EOD	22	9.0	9000	6500	72%	27.4	-1.46	222	201	21	2.76	2.41	3.18	30	91%
Tarrytown	TT1	0.1	Del Rio	NA	15	C. Pipe	7-5/8"	0.375	BOR	35	40.0	9000	3700	41%	25.6	-5.29	218	192	26	2.74	1.58	4.30	82	88%
Tarrytown	TT1	5	Del Rio	NA	15	C. Pipe	7-5/8"	0.375	BOR	35	40.0	9000	6500	72%	33.7	-7.75	280	180	100	2.57	1.11	4.53	315	64%
Tarrytown	TT1	10	Del Rio	NA	15	C. Pipe	7-5/8"	0.375	BOR	35	100.0	9000	5700	63%	33.2	-7.59	311	261	50	3.73	1.30	6.97	158	84%
Tarrytown	TT2	0	Del Rio	NA	20	C. Pipe	7-5/8"	0.375	EOR	35	5.0	9000	4600	51%	20.0	-3.72	120	50	70	0.72	0.12	1.16	221	42%
Tarrytown	TT2	5	Del Rio	NA	20	C. Pipe	7-5/8"	0.375	BOR	35	10.0	9000	5000	56%	23.5	-6.75	180	90	90	1.29	0.20	2.10	284	50%
Tarrytown	TT2	10	Del Rio	NA	20	C. Pipe	7-5/8"	0.375	BOR	35	20.0	9000	5900	66%	27.8	-6.16	217	130	87	1.86	0.31	3.02	274	60%
Tarrytown	TT3	0.1	Del Rio	NA	15	C. Pipe	7-5/8"	0.375	BOR	35	76.9	9000	4200	47%	26.7	-5.44	220	150	70	2.14	1.11	3.52	221	68%
Tarrytown	TT3	5	Del Rio	NA	15	C. Pipe	7-5/8"	0.375	BOR	35	100.0	9000	4700	52%	31.0	-6.45	230	130	100	1.86	1.12	2.84	315	57%
Tarrytown	TT3	10	Del Rio	NA	15	C. Pipe	7-5/8"	0.375	BOR	35	100.0	9000	5400	60%	30.9	-7.13	249	178	70	2.55	1.43	4.04	221	72%
Tarrytown	TT4	0	Del Rio	NA	20	C. Pipe	7-5/8"	0.375	EOD	35	5.0	9000	5200	58%	21.5	-3.10	123	38	85	0.55	0.09	0.90	268	31%
Tarrytown	TT4	5	Del Rio	NA	20	C. Pipe	7-5/8"	0.375	BOR	35	13.3	9000	6000	67%	27.4	-6.03	209	124	85	1.78	0.14	3.01	268	59%
Tarrytown	TT4	10	Del Rio	NA	20	C. Pipe	7-5/8"	0.375	BOR	35	40.0	9000	5400	60%	29.0	-6.81	224	139	85	1.99	0.18	3.35	267	62%
Tarrytown	TT5	0	Del Rio	15	NA	C. Pipe	7-5/8"	0.375	EOD	32	4.2	9000	6500	72%	24.3	-1.97	138	88	50	1.38	0.64	2.03	158	64%
Tarrytown	TT5	5	Del Rio	15	NA	C. Pipe	7-5/8"	0.375	BOR	32	10.0	9000	5000	56%	27.4	-2.07	200	150	50	2.35	0.91	3.62	158	75%
Webberville	WB1	0.1	CL	7	NA	O. Pipe	7-5/8"	0.375	EOD	20	1.8	7500	5100	68%	18.9	-1.21	73	20	53	0.50	0.20	0.66	167	27%
Webberville	WB2	0.1	CL	7	NA	O. Pipe	7-5/8"	0.375	EOD	20	2.8	7500	4200	56%	17.6	-1.48	87	22	65	0.55	0.21	0.73	205	25%
Webberville	WB3	3	CH	10	NA	O. Pipe	7-5/8"	0.375	BOR	20	2.8	7500	4900	65%	21.1	-1.14	87	30	57	0.75	0.64	0.84	180	34%
Webberville	WB3	17	CH	10	NA	O. Pipe	7-5/8"	0.375	BOR	20	3.2	9000	4200	47%	19.7	-1.01	91	43	48	1.07	1.16	0.98	151	47%
Webberville	WB4	1	CH	10	NA	O. Pipe	7-5/8"	0.375	BOR	20	2.7	7500	4700	63%	19.7	-1.03	85	35	50	0.89	0.52	1.25	158	41%
Webberville	WB4	17	CH	10	NA	O. Pipe	7-5/8"	0.375	BOR	20	3.2	9000	4200	47%	21.1	-1.41	100	49	51	1.22	0.83	1.60	161	49%
Webberville	WB5	0.1	CH	10	NA	O. Pipe	12-3/4"	0.250	EOD	20	6.3	7500	4800	64%	25.3	-2.87	123	86	37	1.28	1.17	1.39	42	70%
Webberville	WB5	7	CH	10	NA	O. Pipe	12-3/4"	0.250	BOR	20	10.0	9000	4900	54%	31.6	-2.25	153	114	39	1.70	1.54	1.85	44	75%
San Marcos	SM1	0	Taylor	12	NA	O. Pipe	7-5/8"	0.375	EOD	24.75	2.8	9000	6900	77%	24.1	-1.60	122	103	19	2.09	1.37	2.74	60	84%
San Marcos	SM2	0	Taylor	12	NA	O. Pipe	7-5/8"	0.375	EOD	12.67	2.3	9000	8100	90%	25.6	-1.55	118	67	51	2.81	2.81	NA	161	57%
San Marcos	SM3	0	Taylor	12	NA	SQ Con	8"	NA	EOD	23.33	7.5	9000	3900	43%	3.11	-0.27	173	99	74	1.58	0.97	2.24	212	57%
San Marcos	SM4	0	Taylor	12	NA	SQ Con	8"	NA	EOD	25.00	6.7	9000	3900	43%	3.27	-0.16	169	102	67	1.53	0.71	2.29	192	60%

CL = Lean Clay; CH = Fat Clay

Pre-Drilled Depth was used for Active Zone calculations when applicable.

O. Pipe = Open-Ended Pipe Pile; C. Pipe = Closed-Ended Pipe Pile; SQ Con = Square Precast, Prestressed Concrete Pile

EOD = End of Drive; BOR = Beginning of Restrike; EOR = End of Restrike

Blows per Inch pertain to reciprocal of final set determined by averaging final three blows or number of blows for last 6 inches of penetration.

The averaged end bearing for the β-method also under-predicted by only 15%. Therefore, a combination of the shaft capacity of the α-method was coupled with the toe values for the β-method. The averaged ultimate capacities for the combination analysis under predicted by 33%. Overall, static analysis leads to overly conservative values that are prone to scatter for larger capacities.

GRLWEAP was used to perform wave equation analysis on the four case studies. Three different type of soil modeling were used depending on the information provided by the geotechnical report: simple soils method (ST), SPT-N method (SA), and API based method using unconfined compression strengths (API). No cone penetration tests were provided, so the CPT method was not used.

The main focus of this analysis was driveability and Inspector's Chart. Driveability analysis was run on all test piles by using the default setting at first. When changing quake and damping soil parameters to the values found with CAPWAP, there was no change to results. However when the "Resistance Gain/Loss Factors" were modified, there were great changes in capacities. What modeled CAPWAP results most precisely in the driveability analysis was modifying the "Resistance Gain/Loss Factors" for a shaft value of 2.0 for initial driving and a value of 4.0 for restrikes. The exception for this was a value of 6.0 used for the restruck pile in Manor and the Day 5 piles in Tarrytown using a value of 3.0. GRLWEAP consistently under-predicted in a linear trend.

The Inspector's Chart analysis consisted of entering in ultimate capacity reported by CAPWAP analysis to produce the set at the end of driving for the largest hammer stroke. The driveability set had high scatter and generally over-predicted the driveability of the pile; however the Inspector's Chart produced reliable modeling of the actual set. Therefore driveability should be used to produce a capacity to be entered in the Inspector's Chart analysis to produce a realistic final set.

The Modified Gates dynamic formula is recommended by the Federal Highway Administration (FHWA) for small projects to determine the ultimate capacity of piles. In central Texas, this method is predominantly used for projects with only a handful of jobs actually using a full CAPWAP analysis for the piles driven. Almost all values are being over-predicted by the Modified Gates formula. Figure 3 compares all design methods multiplied by FHWA recommends safety factor of 2.25 for CAPWAP, 3.5 for static and dynamics formulas, and 2.75 for wave equation analysis (Hannigan et al., 2006).

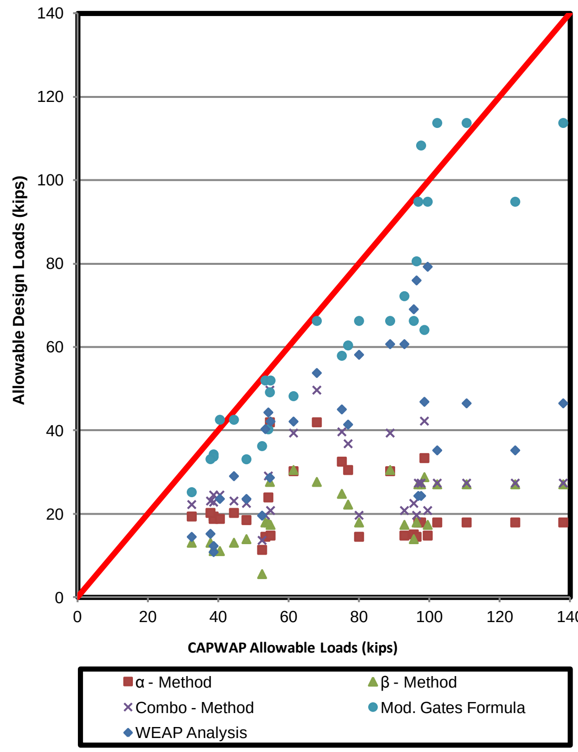

Figure 3: CAPWAP Allowable Loads versus Allowable Loads of Various Design Methods

Unit skin friction versus depth is plotted for all cases in Figure 4 through Figure 8. Figure 4 compares all test piles driven in Manor, 6-5/8 inch diameter pipe piles at initial driving (MN1) and at day 6 (MN2) and a HP 10x42 (MN3). It is notable that the averaged unit skin friction for the pipe pile and H-pile where similar at initial driving, 2.57 and 2.74 ksf, respectively.

For Tarrytown, all test piles on the initial day and the averaged unit skin friction for all test piles on days 0, 5, and 10 were plotted. Figure 5 shows that test piles TT1 – TT4 with predrills exhibited the same behavior as test pile TT5 without a predrill. It demonstrates that the first 10 feet are negligible for skin friction, and only at 15 feet of depth skin friction increased the most. When comparing skin friction trends to the soil borings, this was about the depth when the Upper Colorado River terrace transitioned into the Del

Rio formation. Comparing the averaged skin by day in Figure 6 demonstrates how these unit skin frictions are increasing with time.

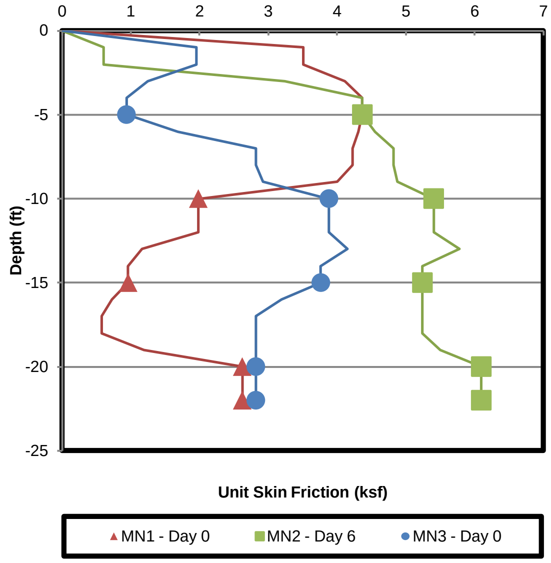

Figure 4: Manor Unit Skin Friction Values with Depth

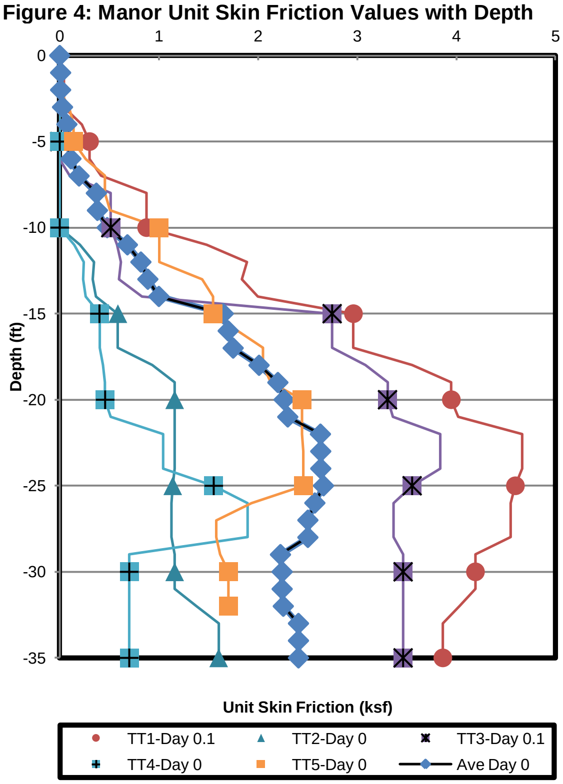

Figure 5: Tarrytown Unit Skin Friction with Depth for Initial Test Date.

The Webberville case had the smallest unit skin friction values. All values were on average of 0.53 ksf for the lean clay piles in Table 1 (WB1 and WB2), 0.82 ksf for the fat clay 7-5/8 inch pipe (WB3 and WB4) and 1.28 ksf for the 12-3/4 inch pipe (WB5) in Figure 7.

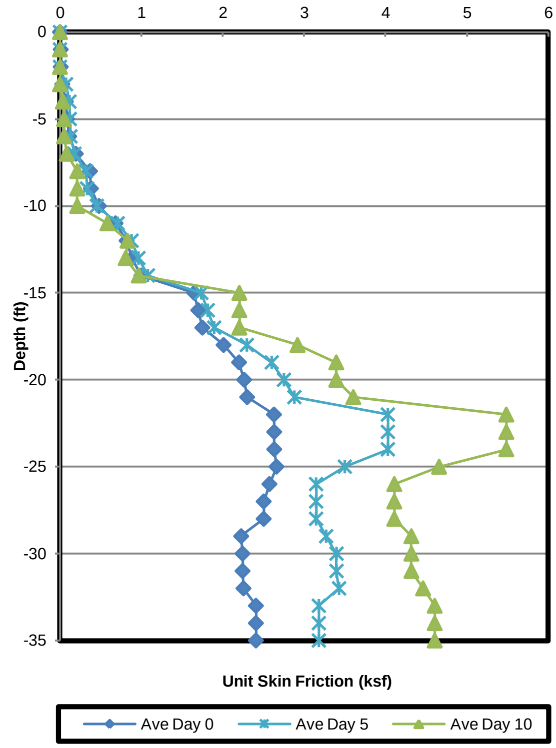

Figure 6: Tarrytown Averaged Unit Skin Frictions with Depth for Day 0, 5 and 10.

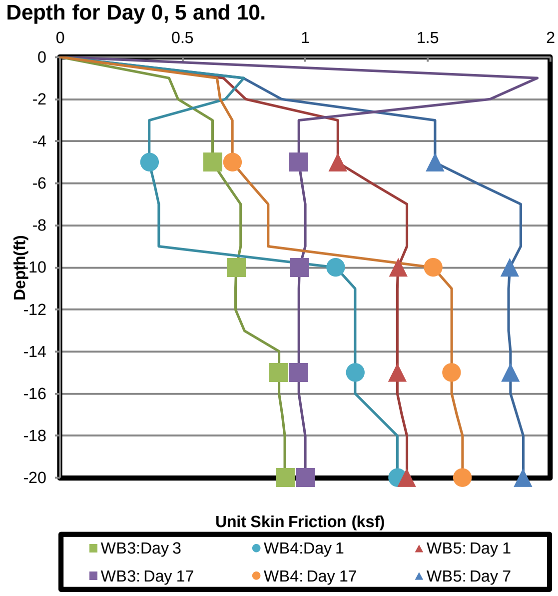

Figure 7: Webberville Unit Skin Friction with Depth for Fat Clay.

Lastly, the San Marcos results in Figure 8 had unusual results of the steel skin friction being 33 to 50% higher than that for the concrete. Typically concrete adheres better to soil than steel does. Another oddity was test pile SM2, the short pipe. The skin friction was very high in the upper soil and had a high end bearing value. There could have been a localized stratum of soil that was stiffer at a shallower depth causing higher skin friction values to be recorded. Also, with having a shorter depth, less pile whip would have occurred. So, there would have been less of a gap at the end of driving, causing higher skin friction.

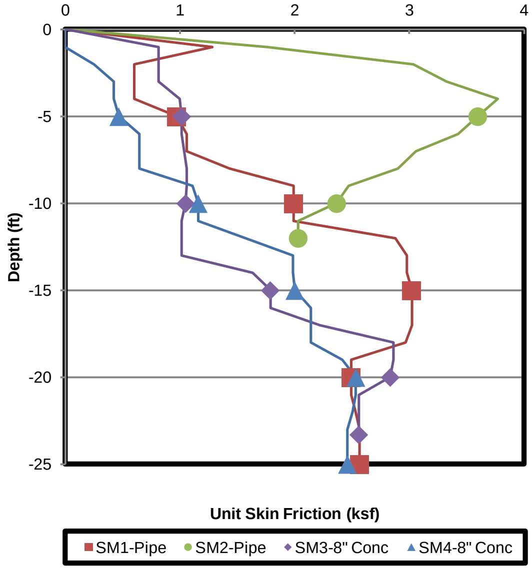

Figure 8: San Marcos Unit Skin Frictions with Depth

Overall, the unit skin friction values found in the zone of seasonal moisture change are less than the values found in the inactive zone except for the Manor 6-5/8 inch diameter pipe pile. A size affect was observed of 50 to 70% increase for the Webberville fat clay piles from the 7-5/8 inch to the 12-3/4 inch diameter open-ended pipe piles. It is also of interest to understand how the soil setup has an impact for each zone of soil. Majorly, the piles increased unit skin friction values with time more in the inactive zone than the active zone. An explanation for this is that soils become denser and more competent with depth. Also, as a pile is driven, deeper soils become more compacted and lock in driving stresses. Another observation is that open-ended pipe cuts through the upper region of soil before a plug is formed. The 6-5/8 inch diameter pipe in the Taylor formation of clay had a higher unit skin friction value in the active zone than the inactive zone. However, when the soil setup was taken into account, there was only a 14% increase in the active zone versus close to a 300% increase in the inactive zone.

Conclusions & Recommendations

Central Texas has a variety of geologic formations with three notable formations that are composed of smectite and are expansive in nature: Taylor/Navarro, Del Rio, and Eagle Ford. These formations span a large distance from south of San Antonio to north of Dallas/Fort Worth. Also Texas experiences extreme droughts followed by extreme rain, partly triggered by the Balconies Escarpment, which exaggerate expansive soil movement. Below are conclusions found from dynamically testing piles and having signal matching CAPWAP analysis performed in two of the three above mentioned clay formations along with an alluvium-based formation within the Lower Colorado River flood basin.

1) Ultimate pile capacities ranged from 73 to 311 kips and are 2 to 6 times the structural capacity needed.
2) Static capacity analysis calculations were overly conservative and scattered, while driveability capacity analysis results closely modeled test capacities. Modified Gates dynamic formula with a factor of safety of 3.5 resembled CAPWAP allowable capacities.
3) Unit skin friction values were on average 2 ksf to 3 ksf for the Taylor formation, 2 ksf to 4 ksf between initial driving and 10 days of soil set up for the Del Rio formation, and 1 ksf for clays intermixed with alluvium. Pile size was found to have an impact on skin friction however steel skin friction was found to be 30 to 50% greater than for concrete.
4) Soil set up ranged from 30 to 100% increase with time.
5) Skin friction was on average 61% of total capacity. Average unit skin friction is less in the zone of seasonal moisture change and exhibits soil set up at a reduced percentage in this zone.
6) Soil plugs stopped moving below zone of seasonal moisture change and plug thickness decreases with increase of pile capacity.

Central Texas pile driving practice consists of a lattice boom crane supporting a box set of leads guiding a minimum of a 3000 pound air hammer. Piles are typically driven in under 20 minutes with an 8 minute averaged installation time implying daily production rate of 15 to 40 piles installed. Central Texas design practice have a dozen examples of a driven pile approach motivated by high water tables and site access resulting in reduced costs for the engineer's client. The majority of these examples did not have geotechnical report recommendations for driven piles. The below items are recommended to be incorporated in future foundation design and specifications:

1) Site investigation should accurately sample a site by drilling at least 10 feet below or 1.5 times the depth of estimated deep foundation for a site. Soil characterization tests should focus on soil strength and swell potential.

2) Surface cracks occur during pile driving in stiff clay caused by pile movement and soil heave that could allow water to travel down and cause expansion. If a concrete slab does not seal these cracks, additional expansive loads should be considered in the design.

3) Predrills should be specified if underground obstructions are found in soil investigation.

4) Small diameter, open-ended pipe pile form a soil plug during driving which eliminates the need for plated ends.

5) Design for dry conditions should consider the worst-case scenario where soil in the active zone completely separates from the driven pile, eliminating all skin friction in that zone. Embedment depth should be based on skin friction and toe bearing pressure necessary to bear all structural loads below active zone.

6) Design for wet conditions should consider the worst-case scenario where soil in the active zone exerts an upward load along the driven pile. Embedment depth should be based on the necessary skin friction below the active zone to overcome upward loads which are reduced by the deadweight of the supported structure.

7) Minimum embedment depth for driven pile design should be based off of the greater depth of the dry and wet conditions. Unit skin friction and end bearing pressure values for all dynamically tested driven piles can be found in Table 1.

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