IDENTIFICATION AND QUANTIFICATION OF PILE RELAXATION

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ABTRACT

Driven piles are widely used for foundation support. In the vast majority of cases, an increase in pile capacity occurs with time which is referred to as "set-up." However, in a very limited number of cases, a decrease in capacity with time can occur. This phenomenon is referred to as "relaxation." This paper will provide a brief review of the relaxation phenomenon, as well as pile driving information and dynamic test data on cases where relaxation has occurred.

The pile types in the presented relaxation cases include H-piles, open-end and closed-end pipe piles, prestressed concrete piles, and prestressed concrete piles with H-pile stingers. The soil conditions where relaxation occurred includes dense to very dense fine sands, heavily over consolidated clays, and weak laminated rocks. Relaxation information from 36 piles at 26 sites is assessed. The collected data will assist foundation designers by identifying the subsurface conditions where relaxation has occurred, as well as by documenting the corresponding relaxation magnitude.

One method of addressing relaxation when it is encountered is to drive the foundation piles to a greater end of initial driving capacity than required long-term. Hence, the identification and quantification of the relaxation magnitude will be beneficial to foundation designers installing similar pile types in similar subsurface conditions. The paper will also present pertinent observations and recommendations on mitigating relaxation when encountered.

INTRODUCTION

One of the earliest observations of a decrease in redriving resistance following initial driving was reported by Miller (1937). He reported that piles driven into a saturated, coarse grained soil could lose 40 to 50% of their resistance when redriven 24 hours after initial driving. Yang (1956) similarly reported that after a break in driving, a decrease in driving resistance could be encountered for piles driven in dense fine sand, inorganic silt, or stiff fissured clay. Parsons (1966) in his paper that described piling difficulties in the New York City metropolitan area termed this phenomenon "relaxation". In this time period, pile capacity was most often evaluated from a pile driving formula frequently checked by a static load test. Hence, the true magnitude of the capacity decrease on a given pile was often poorly quantified. Furthermore, dynamic measurements that could determine what influence driving system performance variation had on the observed redriving resistance were not yet readily available.

The consequences of relaxation can be significant if not identified and addressed during construction. As a result of relaxation, the installed piles will have less than the desired ultimate capacity or nominal resistance yielding a lower factor of safety than targeted. In severe relaxation cases, foundation performance can be compromised and excessive settlements may occur. Morgano and White (2004) presented two case histories where relaxation occurred, one where the piles were driven into shale, and the other where the piles were terminated in sand. Test results indicated a reduction in ultimate capacity of 29% at the shale site and of 30% at the sand site. Both cases required the piles to be driven deeper to mitigate relaxation.

RELAXATION MECHNAISMS

Thompson and Thompson (1985) reviewed end of driving and restrike blow count data from eight sites where the decrease in the observed blow count at the beginning of restrike, relative to the end of driving blow count, suggested relaxation had occurred. Fortunately, these sites also had dynamic measurement data which allowed the authors to differentiate between real relaxation, which occurred at only two of the eight sites, and apparent relaxation, where the decrease in restrike blow count was due to improved hammer performance. The pile types in the two real relaxation cases were an H-pile and a closed-end pipe pile. The two projects were located near each other. In both cases, the bearing layer was a weathered shale bedrock with thin limestone layers and clay seams. The H-piles typically penetrated two to four feet into the shale whereas the heavier loaded closed-end pipe piles penetrated up to 10 feet into the shale and were restruck as many as five times. Possible relaxation mechanisms were shale softening from water migration to the pile toe in the peripheral opening created by the pile driving shoe or closure plate, as well as shale shattering beneath the toe of an already driven pile caused by driving adjacent piles to a deeper final penetration depth. Relaxation of locked-in horizontal rock stresses following pile driving was theorized as another potential relaxation mechanism.

York et al., (1994) reported on a case at JFK International Airport in New York where both setup along the pile shaft and relaxation at the pile toe were encountered in a medium dense, medium to fine glacial sand. Relaxation happened in some larger displacement pile groups when densification of the sand occurred as a result of pile driving. The dense sand dilated and developed negative pore pressures as later piles in the group were driven into the dense material. The negative pore pressure resulted in temporarily elevated effective stresses that required a few hours or several days to return to normal. Once the effective stresses had normalized, piles could be driven past the densified sand layer.

Herrera (2015) reviewed pile relaxation cases in Florida granular soils. He recommended that potential relaxation zones, identified by Standard Penetration Test (SPT) resistance values, be assessed using cone penetration tests with shoulder pore pressure measurements, CPT-u. SPT N values in silty and shelly sands greater than 24 with an automatic hammer, or greater than 30 with a safety hammer were of particular concern. Soil layers where negative pore pressures are encountered in the CPT-u data should then be evaluated with pore pressure dissipation tests. He recommended these results be used as a guide in identifying soils where restrike tests should be performed to check for relaxation.

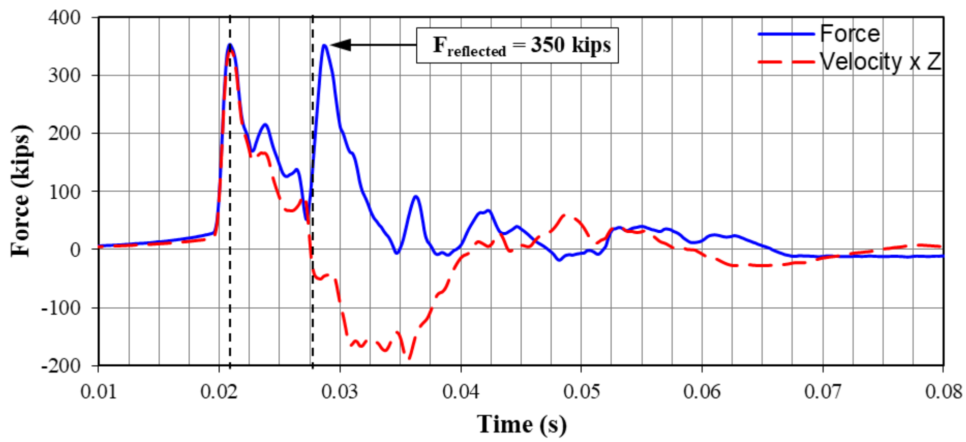
DYNAMIC MEASUREMENT RECORDS

Figure 1 presents dynamic measurement data for a typical relaxation case. The data is from an H-pile driven into a shale bedrock with a single acting diesel hammer. Force and velocity records versus time are presented for the end of initial driving (EOID) in the top half of the figure and at the beginning of restrike (BOR) in the bottom half. The time of impact is indicated by the dotted vertical line on the left, and the toe response, time 2L/c, is indicated by the dotted vertical line on the right. Selected Case Method output quantities are identified by their three letter code along the left margin.

In the end of initial driving records, the impact force, FT1, is 353 kips. This is also the maximum force at the gage location, FMX. The force reflected from the pile toe is 350 kips, slightly less than the impact force. The velocity record at that time is negative resulting in a mobilized resistance of 400 kips as quantified by RX9, the Maximum Case Method resistance with a damping factor of 0.90. The mobilized resistance exceeds the impact force. The ratio of the resistance over the impact force, ROF, is 1.13. For the record presented, the calculated hammer stroke, STK, is 7.03 ft and the energy transferred to the pile head is 9.9 ft-kips. CSX, the calculated compression stress at the gage location, is 22.8 ksi and CSB, the calculated compression stress at the pile toe, is 27.8 ksi. The equivalent penetration resistance over the final inch of driving was 28 blows per inch as the pile terminated in a hard shale bedrock.

Records from the beginning of restrike, clearly indicate a weaker toe resistance and relaxation. The impact force, FT1, is 372 kips, the calculated hammer stroke, STK, is 7.31 ft, and the energy transferred to the pile head is 10.9 ft-kips. However, the reflected maximum force from the pile toe of 253 kips is significantly less than the 350 at the end of initial driving. The penetration resistance over the first inch of restrike was 8 blows per inch. The pile was restruck with an equivalent to slightly greater impact force and transferred energy, and moved substantially easier due to the lower toe resistance. This is also indicted by the lower ratio of the resistance over the impact force, ROF, of 0.73.

End of Initial Driving		
FT1	353	kips
FMX	353	kips
RX9	400	kips
ROF	1.13	%
STK	7.03	ft
EMX	9.9	ft-kips
CSX	22.8	ksi
CSB	27.8	ksi



Beginning of Restrike		
FT1	372	kips
FMX	372	kips
RX9	271	kips
ROF	0.73	%
STK	7.31	ft
EMX	10.9	ft-kips
CSX	24.0	ksi
CSB	20.9	ksi

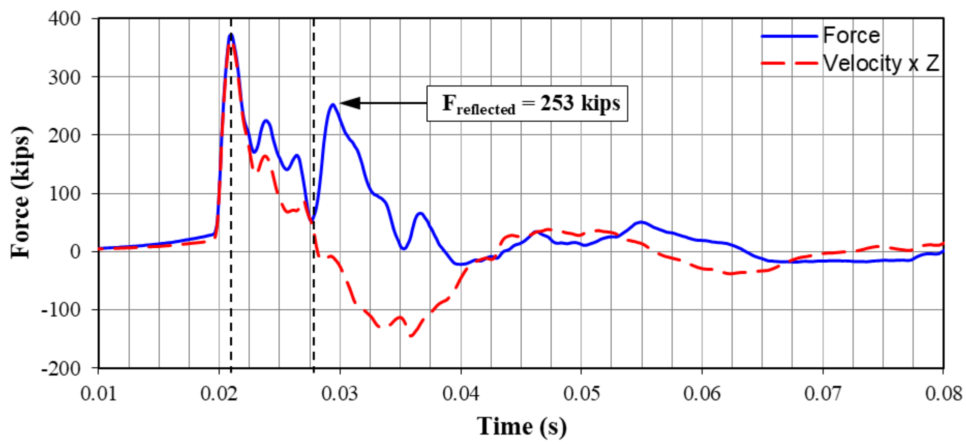


Fig. 1. Dynamic measurement data for a pile driven into a shale bedrock

The ratio of the mobilized resistance over the impact force is very useful in assessing relaxation particularly when the hammer performance often increases from blow to blow such as with a single acting diesel hammer. If the ROF ratio is the same or greater than it was at the end of driving and yet the mobilized resistance is less, this is an indication of the limited impact force mobilizing less capacity. If the ROF ratio is less at the beginning of restrike yet the impact force is equivalent or greater than it was at the end of driving, relaxation has likely occurred. However, an increase in the dynamic soil resistance between the end of driving and beginning of restrike could also be a contributing factor.

RELAXATION DATABASE

Relaxation data from 36 piles at 26 sites were compiled and reviewed. The pile types included 23 H-piles, two open-end pipe piles, five closed-end pipe piles, four prestressed concrete piles, and two prestressed concrete piles with H-pile stingers. The soil or rock materials at the pile toe included 13 cases in dense to very dense sands, one case in heavily over consolidated silty clay till, and 21 cases in weak laminated rocks. The rock cases consisted of 18 cases in shale, three cases in limestone, and one case in siltstone.

Table 1 presents a summary of the pile type and end bearing material for the 36 cases. This table includes the magnitude of the toe resistance loss as well as the percentage loss in toe resistance. The table also summarizes the restrike pile penetration required to re-obtain the geotechnical resistances equivalent to the design load, the ultimate capacity, and the signal matching determined end of initial driving capacity.

Relaxation is quantified by the decrease in capacity that occurs between the beginning of restrike and the end of initial driving. In some cases, an increase in shaft resistance may occur at the same time that a decrease in toe resistance occurs. When this happens, the soil setup will mask the full magnitude of the toe relaxation. Therefore, signal matching analysis is the best dynamic analysis method to assess the change in soil resistance and its distribution on the pile. The dynamic measurements reported in this paper were analyzed with the CAPWAP signal matching program, Rausche at al., (2010).

A key item in determination of the relaxation magnitude is selecting the appropriate hammer blow for signal matching analysis to evaluate the shaft and toe resistance. In general, the dynamic measurements acquired near the end of initial driving are similar from one blow to another. Hence, impact force and data quality are the key factors in selecting the blow for signal matching analysis. For restrike situations, the blow selected for signal matching analysis is again chosen based on the data quality, impact force, as well as the ratio of the mobilized resistance divided by the impact force, ROF. In cases where the ROF value decreases over the initial restrike blows and then increases, the blow with or close to the lowest ROF is generally the blow for signal matching analysis contingent upon data quality and impact force magnitude.

Figure 2 presents a summary of several calculated quantities from each hammer blow over the first inch of restrike. The lowest ROF ratio of 0.85 occurred for blow 2 in conjunction with a somewhat lower impact force, FT1, of 775 kips. Blow 3 had a marginally higher ROF ratio of 0.87, with an impact force of 866 kips. This impact force is also closer to the impact force at the end of initial driving of 880 kips.

Driving Status	Blow Number	Impact Force FTI (kips)	Signal Matching Capacity (kips)	Case Method Capacity (kips)	Transferred Energy EMX (ft-kips)	Hammer Stroke STK (ft)	Resistance / Force ROF (%)
EOID	125	880	963	959	57.9	8.88	1.09
BOR	1	529	- - -	445	21.9	Trip Blow	0.88
	2	775	- - -	627	46.9	7.72	0.85
	3	866	713	717	55.4	8.74	0.87
	4	872	- - -	751	57.3	8.75	0.91
	5	880	- - -	791	57.2	8.91	0.95
	6	884	- - -	808	57.5	8.97	0.96
	7	903	- - -	842	60.0	9.26	0.98
	8	911	- - -	863	60.0	9.30	1.00

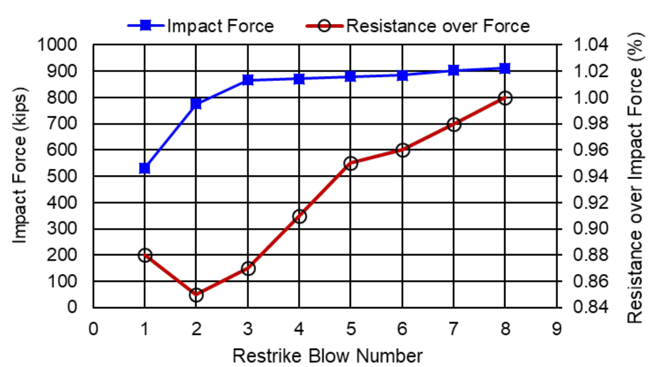


Fig. 2. Example of the restrike blow selection process for relaxation assessment

Blow 3 was therefore chosen for signal matching analysis and assessment of the relaxation magnitude. The increase in ROF ratio with subsequent blows indicates the pile was being redriven into the shale to regain the additional capacity noted for those subsequent blows. This process was used for the restrike cases in Table 1 to select the blow to analyze for the toe relaxation magnitude.

Figure 3 presents the loss in toe resistance between the end of initial driving and the beginning of restrike for the piles driven into dense granular soils. As noted in Table 1, the Unified Soil Classification System designations for the materials at the pile toe included SP, SM, SP-SM, SM-SC, and SW-SM. In all cases, these sand materials were dense to very dense with uncorrected Standard Penetration Test (SPT) resistance values ranging from 31 blows per foot with an unreported SPT hammer type to 50 blows / 0.1 inches with an automatic SPT hammer. For the 13 sand cases, the restrike toe resistance ranged from 12% to 89% of the toe resistance at the end of driving and averaged 62%. The magnitude of relaxation for the two low displacement H-piles was less than that encountered by the 11 displacement piles in the dense granular materials.

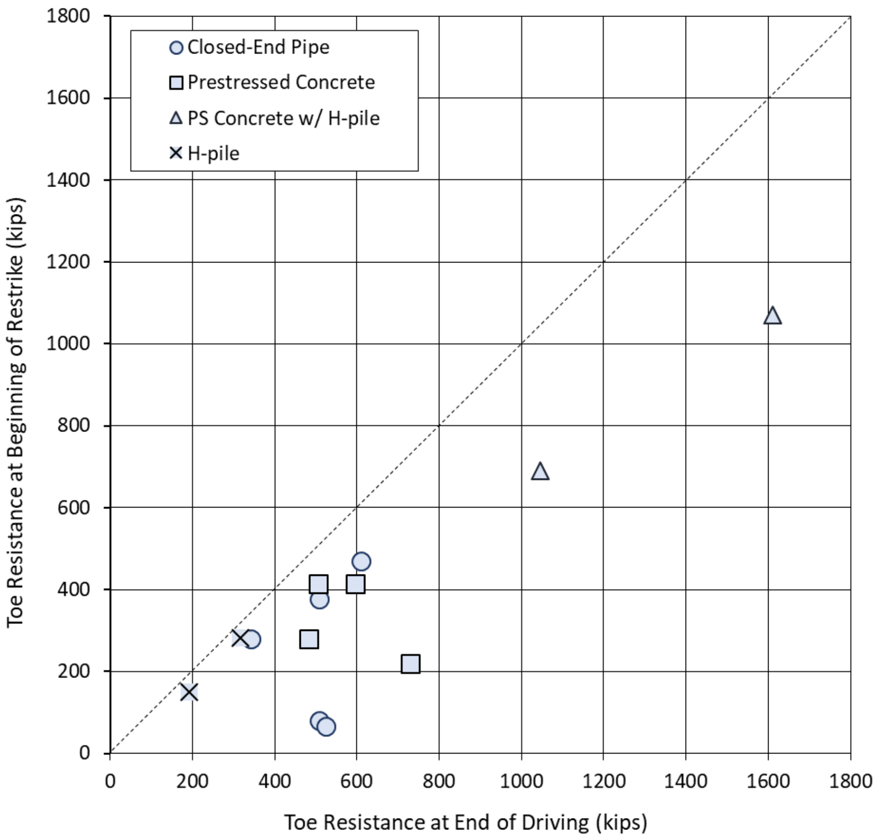


Fig. 3. Change in Toe Resistance in Sands

Table 1 includes the restrike pile penetration required to re-obtain the geotechnical resistances equivalent to the design load, the ultimate capacity, and the signal matching determined end of initial driving capacity. The pile penetration required to re-acquire these resistances was determined from the number of hammer blows applied before re-acquiring that resistance and the associated pile movement from the recorded blows per inch or fraction thereof. Piles having a restrike geotechnical resistance less than the design load would experience settlement under that applied load. Therefore, the pile penetration required to re-achieve the design load is presented solely as an indicator of the maximum potential settlement magnitude if relaxation had not been identified and addressed.

For the restrike event, 0.0 to 0.5 inches of movement was needed to re-achieve the geotechnical resistance equal to the design load with an overall average of 0.05 inches. The penetration required to regain the required ultimate capacity varied from no movement in four cases to in excess of 22 inches in three cases. One pile was driven 104 feet without achieving the ultimate capacity. Piles that required no movement to regain the ultimate capacity had initially been overdriven to satisfy minimum penetration requirements.

The loss in toe resistance between the end of driving and the beginning of restrike for piles driven into shale bedrock is presented in Figure 4. The piles in 16 of the cases were H-piles with driving shoes and in the other two cases the piles were open-end pipe piles with driving shoes. The shale material properties were characterized by either Rock Quality Designation (RQD) values or SPT resistance values. Where the shale was cored, the RQD values ranged from 0 to 90%. Many sites had an RQD of 0% so the average RQD was only 14%. Where the shale was sampled using a split-spoon sampler, uncorrected SPT resistance values ranged from 50 for 5 inches to 100 for 0.1 inches, both with an unreported SPT hammer type. For the 18 shale cases, the restrike toe resistance ranged from 27% to 82% of the toe resistance at the end of driving and averaged 65%.

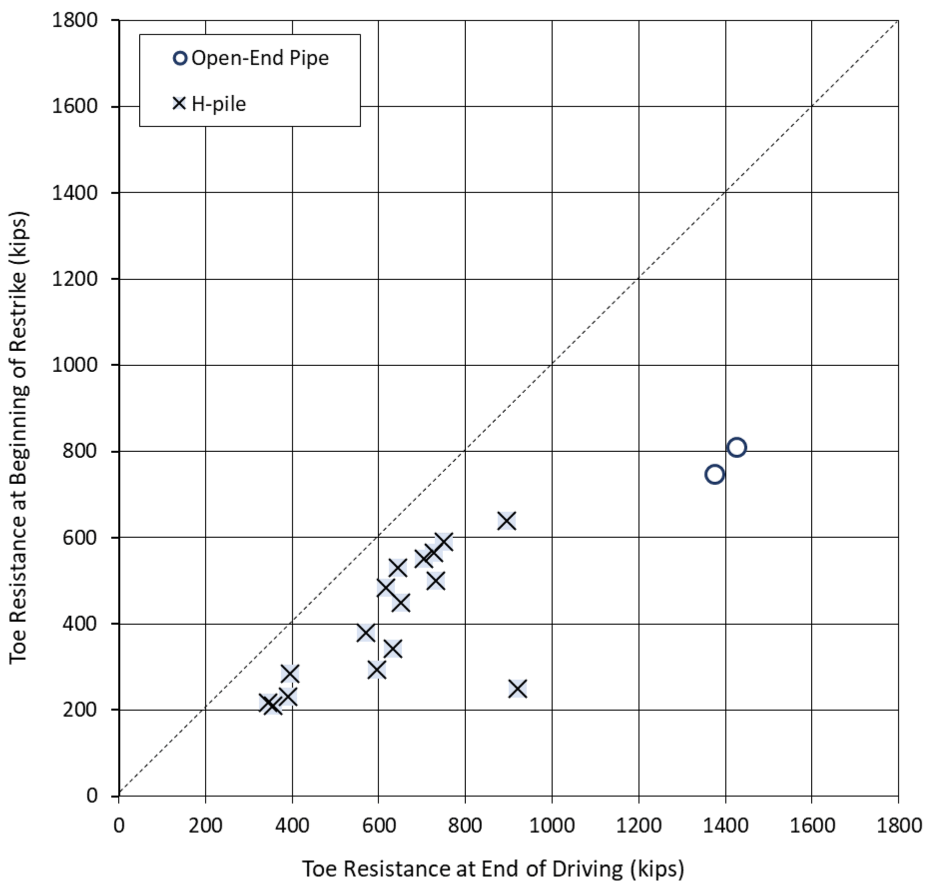


Fig. 4. Change in Toe Resistance in Shales

When piles driven into shale exhibit relaxation, a common question is whether they will relax again after being redriven. In the shale data sets presented herein, the same pile was redriven and restruck a second time in five cases, and redriven and restruck a third time in two cases. The reduction in toe resistance for the five, second restrike cases ranged from 14% to 71% and averaged 36%. The toe resistance for the two, third restrike cases ranged from 32% to 46%. The second and third restrike data is not presented in Figure 4 but it is included in Table 1.

For the first restrike event, no movement to 7.0 inches of movement was needed to re-achieve the geotechnical resistance equal to the design load with an overall average of 0.47 inches. It should be noted that 17 of the 18 cases required 0.33 inches of penetration or less to achieve the design load. To regain the ultimate capacity, no movement to 84 inches of penetration were required with five cases needing 1.3 inches or greater penetration. Piles that required no movement to regain the ultimate capacity had initially been driven to a capacity greater than the ultimate capacity because relaxation had been anticipated.

For piles subject to a second restrike, four of the five piles required no movement to re-achieve the design load with the fifth pile requiring 1.33 inches of movement. To regain the required ultimate capacity in the second restrike, 0.44 to 7.75 inches of additional pile penetration was required.

Figure 5 summarizes the loss in toe resistance between the end of driving and the beginning of restrike for piles driven in other materials including limestone, siltstone, and silty clay till. In all five cases, the pile type was an H-pile with a driving shoe. The database in these materials is too limited for further analysis. However, the change in toe resistance for the H-piles driven into the limestone and siltstone materials is not dissimilar to the shale bedrock. In the limestone cases, the rock description included the terms “severely fractured”, “containing a steep fracture plane”, or “containing thin shale seams”. These features likely explain why a limestone bearing material which is not generally associated with relaxation problems exhibited relaxation. Heave was also not reported for this pile which, if observed, would be another reason for the reduced capacity upon restrike.

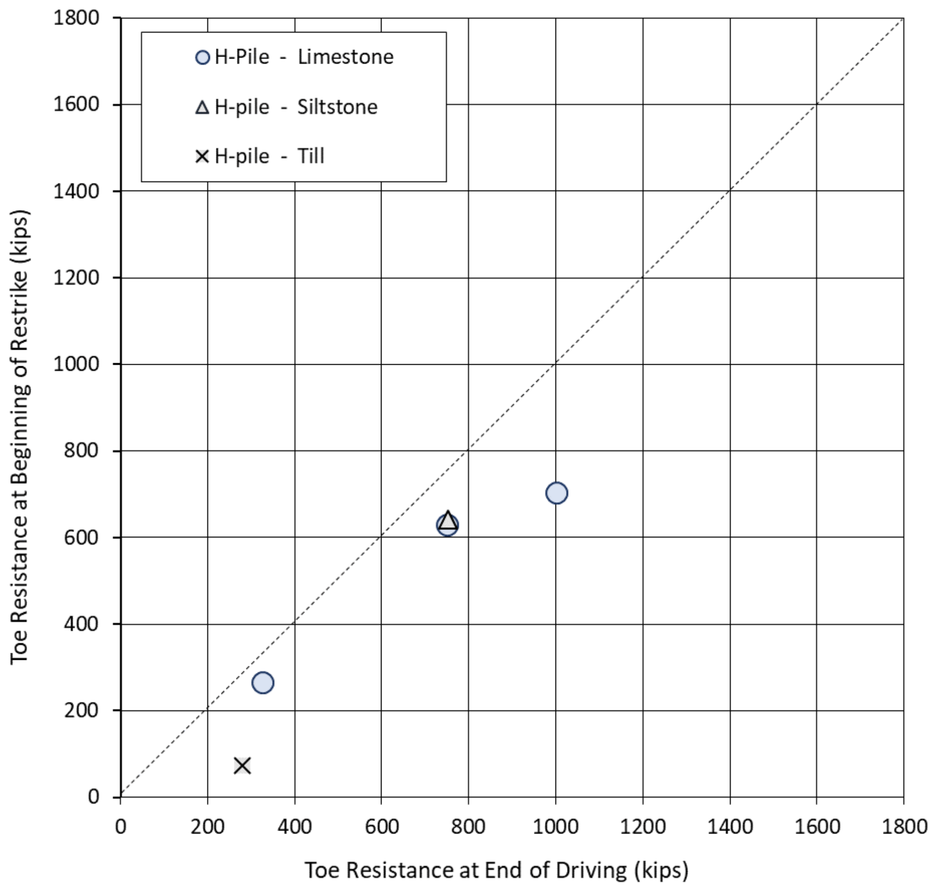


Fig. 5. Change in Toe Resistance in Limestone, Siltstone, and Silty Clay Till

RELAXATION VERSUS TIME

The change in toe resistance was also evaluated as a function of the elapsed time since initial driving with consideration of the end bearing material and pile type. Plots of the percentage toe resistance loss as a function of log time are presented in Figure 6 for sands and shale bearing materials. Only data for the initial restrike is presented in the figure. It should not be construed that long term restrikes are required in sands, as the same magnitude of relaxation would like have been apparent if the initial restrike had been performed at an earlier time. Shales however have been known to exhibit a greater magnitude of relaxation with time. Regardless, no clear trend in relaxation magnitude as a function of time is indicated by the sand or shale data other than restrike tests should be performed in materials prone to relaxation.

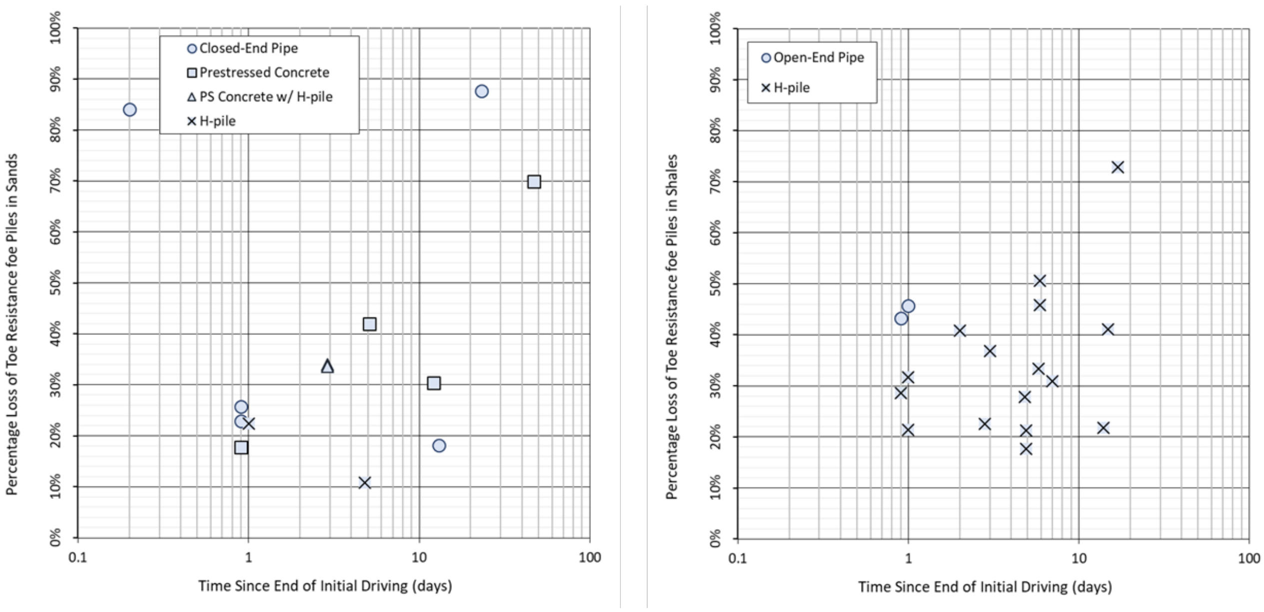


Fig. 6. Percentage Toe Resistance Loss in Sands (left) and Shales (right) Versus Time

DISCUSSION

Many of the piles in the database were driven to bedrock. In these situations, a low set per blow is often encountered at final driving and dynamic test methods can underestimate the ultimate capacity. Several potential data sets were not included in this database, such as the example data presented in Figure 1, due to high end of drive or restrike blow counts. However, a few of the end of driving cases with high final blow counts were retained (ID-7, ID-10, ID-11, and ID-18) if the restrike blow count was less than 11 blows per inch. A similarly high end of driving blow count occurred for ID-6 which was terminated in sand. In actuality, the loss in toe resistance for data sets with high final driving resistance including the above noted cases would be underestimated due to the end of initial driving toe resistance being underestimated.

In many of the cases, relaxation was anticipated based on the end bearing material and those piles were driven in excess of the required ultimate capacity to accommodate some later magnitude of relaxation. In other cases, satisfying minimum penetration requirements resulted in overdriving. For the database piles in sand, the piles were driven on average to a capacity 59% greater than the required ultimate capacity with a range of 6% to 202%. Database piles driven into rock were driven on average to a capacity 28% greater than the required ultimate capacity with a range of 2% to 86%. This may have resulted in the rock piles terminating in less weathered material.

While pile penetration into the rock is not quantified in this paper, experience has indicated that piles terminating in more weathered materials generally tend to have a greater magnitude of relaxation than pile driven deeper into less weathered material. Hence, the toe resistance loss for the rock database piles may have been greater if pile driving had been terminated at only the required ultimate capacity.

In two cases, the pile was hammer changed or the pile was concrete filled between the end of driving and beginning of restrike. This caused inconsistency in the ROF values between the end of drive and the beginning of restrike noted for ID-2, and ID-6 in Table 1.

CONCLUSIONS AND RECOMENDATIONS

Piles driven into dense to very dense sands with Unified Soil Classification System designations of SM, SP, SP-SM, SM-SC, and SW-SM can have a significant loss in toe resistance over time. The limited number of cases presented herein suggest the average magnitude of toe relaxation loss is 38% in these materials and is greater for the 11 displacement piles (33%) than for the two low-displacement H-piles (17%). Piles driven into shales had an average toe relaxation loss of 35%. The average toe resistance loss was greater for the two open-end pipe piles (44%) than for the 16 H-piles (34%). Additional data is needed for all pile types in these end bearing materials.

The data sets indicate that overdriving is beneficial to limit the detrimental effects of relaxation. Piles were overdriven to some magnitude to accommodate future relaxation in all but three of the 36 data sets. The average toe resistance loss was 32% for the overdriven pile data sets compared to an average toe resistance loss of 46% for the three data sets not overdriven. Additional data is again needed to bolster this conclusion.

Based on the data sets reviewed, driving to a capacity in excess of the required ultimate capacity is recommended when piles are terminated in relaxation prone materials. The average magnitude of toe resistance loss in sands was 244 kips or 38% of the end of initial driving toe resistance. In shales, the average magnitude of toe resistance loss was 258 kips or 35% of the end of initial driving toe resistance. For fractured limestones, the average magnitude of toe resistance loss was 160 kips or 22% of the end of initial driving toe resistance, and for the one case in siltstone, the magnitude of toe resistance loss was 114 kips or 14% of the end of initial driving toe resistance. All of these relaxation magnitudes are based on the pile sizes noted in Table 1.

Pile designs and wave equation drivability analyses should consider the potential magnitude of relaxation and the pile-hammer system be designed accordingly. Depending on the end bearing material and pile type, most of the data presented herein suggests that driving to an ultimate capacity that includes a 20 to 45% loss in toe resistance would compensate for the relaxation magnitude in 83% of the database cases. However, the database indicates a greater loss in toe resistance in six cases. Therefore, restrike dynamic load tests with signal matching analysis or static load tests should also be performed to quantify relaxation magnitudes on site specific materials and pile types where relaxation is a design concern.

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Table 1. Summary of Pile Type, Bearing Material, and Resistance

Case ID	Pile Type	Boring Distance to Pile (ft)	Toe Bearing Material Description	Toe Bearing Material Classification	Toe Bearing Material Strength	Design Load or *Factored Load* (kips)	Ultimate Capacity or *Nominal Resistance* (kips)	Test Status	Pen (ft)	Time Since EOID (days)	Reported Blows per Inch	Impact Force (kips)	Blow No. to EOID or From BOR	Signal Matching Analysis (SM) Total Capacity (kips)	Signal Matching Analysis (SM) Shaft Resistance (kips)	Signal Matching Analysis (SM) Toe Resistance (kips)	Total Capacity / Impact Force (%)	Signal Matching Toe Resistance Loss (kips)	Signal Matching Toe Resistance Loss (%)	Signal Matching Total Resistance Loss (kips)	Signal Matching Total Resistance Loss (%)	Penetration Required to Regain Prior SM Capacity (in)	Penetration Required to Regain Design Load (in)	Penetration Required to Regain Ultimate Capacity (in)
1	14 in O.D. x 0.50 in CEP	10	M Dense to V Dense Fine Sand,	SP, SP-SM	N=40[a]	TBD	TBD	EOID	64.7	- - -	3	747	4	644	137	507	0.86	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Trace Silt			"	"	BOR	64.7	0.2	1	580	2	211	130	81	0.36	426	84%	433	67%	> 841 in	n.a.	n.a.
2	14 in O.D. x 0.50 in CEP	44	M Dense to V Dense Fine Sand,	SP, SP-SM	N=79[s]	TBD	TBD	EOID	65.7	- - -	3	747	2	737	212	525	0.99	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Trace Silt			"	"	BOR	65.7	23.1	4	334*	2	425	360	65	1.27	460	88%	312	42%	n.a.	n.a.	n.a.
3	HP 12x53 w/shoe	50	V Dense Sand and Gravel	- - -	N=50/5 in[u]	*270*	*386*	EOID	53.3	- - -	4	446	5	447	131	316	1.00	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			with Cobbles			"	"	BOR	53.3	4.8	3	432	4	350	68	282	0.81	34	11%	97	22%	> 1244 in	None	> 1244 in
4	14 in O.D. x 0.375 in CEP	135	Medium Dense Sand	SP-SM	N=31[u]	180	360	EOID	25.1	- - -	3	654	4	568	60	508	0.87	- - -	- - -	- - -	- - -	- - -	- - -	- - -
						"	"	BOR	25.1	0.9	4	614	3	504	127	377	0.82	131	26%	64	11%	6 in	None	0.25 in
5	HP 12 x74 w/shoe	16	Loam (Sandy Silty Clay)	CL-ML	N=87/11 in[u]	*265*	*379*	EOID	34.0	- - -	2	738	4	493	214	279	0.67	- - -	- - -	- - -	- - -	- - -	- - -	- - -
						"	"	BOR	34.0	2.9	3	646	3	350	276	74	0.54	205	73%	143	29%	> 6 in	None	5.5 in
6	14 in O.D. x 0.375 in CEP	unknown	M Dense to V Dense Sand	SM	N=75/10 in[u]	180	360	EOID	30.6	- - -	15	604	5	458	117	341	0.76	- - -	- - -	- - -	- - -	- - -	- - -	- - -
								BOR	30.6	13.0	11	204*	10	402	123	279	1.97	62	18%	56	12%	n.a.	n.a.	n.a.
7	HP 12x53 w/shoe	unknown	Gray Shale	Shale	No RQD	160	320	EOID	30.1	- - -	28	360	4	388	33	355	1.08	- - -	- - -	- - -	- - -	- - -	- - -	- - -
						"	"	BOR	30.1	2.0	8	373	2	271	61	210	0.73	145	41%	117	30%	1.25 in	0.25 in	0.50 in
8	HP 14x117	unknown	Shale	Shale	No RQD	744	1488	EOID	30.1	- - -	7	1490	3	1140	220	920	0.77	- - -	- - -	- - -	- - -	- - -	- - -	- - -
						"	"	BOR #1	30.1	16.9	3	1165	3	503	253	250	0.43	670	73%	637	56%	85 in	7.0 in	84 in
						"	"	EOR #1	37.2	- - -	9	1463	4	1335	335	1000	0.91	- - -	- - -	- - -	- - -	- - -		- - -
8	HP 14x117	unknown	Shale	Shale	No RQD	744	1488	BOR #2	37.2	37.7	3	1204	2	635	347	288	0.53	712	71%	700	52%	9 in	1.33 in	7.75 in
						"	"	EOR #2	37.95	- - -	14	1446	2	1530	400	1130	1.06	- - -	- - -	- - -	- - -	9 in	1.33 in	7.75 in
9	HP 12x63 w/shoe	unknown	Red Shale	Shale	No RQD	200	400	EOID	22.6	- - -	13	383	1	600	210	390	1.57	- - -	- - -	- - -	- - -	- - -	- - -	- - -
						"	"	BOR	22.6	14.8	6	361	3	470	240	230	1.30	160	41%	130	22%	3.3 in	None	0.33 in
10	HP 14x89 w/shoe	60	Weathered Gray Shale,	Shale	RQD = 0	450	900	EOID	81.2	- - -	13	880	4	963	67	896	1.09	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			broken, wthrd joints			"	"	BOR #1	81.2	0.9	8	867	3	713	73	640	0.82	256	29%	250	26%	1.8 in	0.25 in	1.3 in
10	HP 14x89 w/shoe	60	Weathered Gray Shale,	Shale	RQD = 0	450	900	EOR #1	81.5	- - -	20	860	2	855	75	780	0.99	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			broken, wthrd joints			"	"	BOR #2	81.5	13.7	10	893	2	754	94	660	0.84	120	15%	101	12%	0.75 in	None	0.80 in
11	HP 14x89 w/shoe	120	Highly Weathered and	Shale	RQD = 35	450	900	EOID	95.33	- - -	28	880	2	819	87	732	0.93	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Broken Gray Shale			"	"	BOR #1	95.33	1.0	11	796	4	668	168	500	0.84	232	32%	151	18%	1.8 in	0.27 in	2.1 in
11	HP 14x89 w/shoe	120	Highly Weathered and	Shale	RQD = 35	450	900	EOR #1	95.58	- - -	23	821	1	910	190	720	1.11	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Broken Gray Shale			"	"	BOR #2	95.58	13.2	43 / .5	795	2	805	185	620	1.01	100	14%	105	12%	0.18 in.	None	1.1 in
12	24 in x 0.5 in OEP w/shoe	65	Thickly Bedded, Weathered	Shale	RQD = 90	600	1200	EOID	100.25	- - -	9	1289	4	1522	95	1427	1.18	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Sound Gray Shale			"	"	BOR #1	100.25	0.9	6	1267	2	958	148	810	0.76	617	43%	564	37%	> 2 in	0.17 in	0.67 in
12	24 in x 0.5 in OEP w/shoe	65	Thickly Bedded, Weathered	Shale	RQD = 90	600	1200	EOR #1	100.41	0.9	14	1367	4	1445	125	1320	1.06	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			to Sound Gray Shale			"	"	BOR #2	100.41	13.1	16	1277	1	804	331	473	0.63	847	64%	641	44%	1.25 in	None	0.69 in
12	24 in x 0.5 in OEP w/shoe	65	Thickly Bedded, Weathered	Shale	RQD = 90	600	1200	EOR #2	100.54	13.1	10 / 0.5	1373	2	1511	191	1320	1.10	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			to Sound Gray Shale			"	"	BOR #3	100.54	23.9	6	1296	1	911	291	620	0.70	700	53%	600	40%	> 1 in	None	0.67 in
13	24 in x 0.5 in OEP w/shoe	40	Weathered Gray Shale,	Shale	RQD = 0	600	1200	EOID	88.66	- - -	9	1326	2	1456	81	1375	1.10	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Broken, Weathered Joints			"	"	BOR #1	88.66	1.0	6	1113	2	877	131	746	0.79	629	46%	579	40%	> 6 in	0.33 in	3.2 in
13	24 in x 0.5 in OEP w/shoe	40	Weathered Gray Shale,	Shale	RQD = 0	600	1200	EOR #1	89.16	- - -	9	1287	2	1265	125	1140	0.98	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Broken, Weathered Joints			"	"	BOR #2	89.16	13.1	9	1411	2	1123	153	970	0.80	170	15%	142	11%	0.67 in	None	0.44 in
13	24 in x 0.5 in OEP w/shoe	40	Weathered Gray Shale,	Shale	RQD = 0	600	1200	EOR #2	89.33	- - -	9	1410	5	1416	166	1250	1.00	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Broken, Weathered Joints			"	"	BOR #3	89.33	24.1	7	1323	1	1044	194	850	0.79	400	32%	372	26%	> 3 in	None	0.71 in
14	HP 12 x 63 w/shoe	66	Moderately Weathered	Shale	RQD = 0	*300*	*430*	EOID	15.2	- - -	8	579	1	662	67	595	1.14	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Gray Shale, Thinly Bedded			*"*	*Rndr = 505*	BOR #1	15.2	5.9	9	479	1	365	71	294	0.76	301	51%	297	45%	> 4 in	None	0.55 in
						"	*"*	BOR #1	15.2	5.9	9	540	4	525	63	462	0.97	133	22%	137	21%	> 4 in	None	0.55 in

Notes: EOID = End of Intital Driving, BOR = Beginning of Restrike, TBD = To Be Determined, a = automatic SPT hammer, s = SPT safety hammer, u = unreported SPT hammer type, Pen = embedded depth in ground, * = concrete filled pipe

Table 1. Summary of Pile Type, Bearing Material, and Resistance (continued)

Case ID	Pile Type	Boring Distance to Pile (ft)	Toe Bearing Material Description	Toe Bearing Material Classification	Toe Bearing Material Strength	Design Load or *Factored Load* (kips)	Ultimate Capacity or *Nominal Resistance* (kips)	Test Status	Pen (ft)	Time Since EOID (days)	Reported Blows per Inch	Impact Force (kips)	Blow No. to EOID or From BOR	Signal Matching Analysis (SM) Total Capacity (kips)	Shaft Resistance (kips)	Toe Resistance (kips)	Total Capacity / Impact Force (%)	Signal Matching Toe Resistance Loss (kips)	(%)	Signal Matching Total Resistance Loss (kips)	(%)	Penetration Required to Regain Prior SM Capacity (in)	Design Load (in)	Ultimate Capacity (in)
15	HP 12 x 63 w/shoe	54	Moderately Weathered	Shale	RQD = 0	*300*	*430*	EOID	20.2	- - -	8	582	2	665	33	632	1.14	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Gray Shale, Thinly Bedded			"	*Rndr = 520*	BOR #1	20.2	5.9	9	481	1	407	65	342	0.85	290	46%	258	39%	1.7 in	None	0.46 in
						"	"	BOR #1	20.2	5.9	9	528	4	520	53	467	0.98	165	26%	145	22%	1.7 in	None	0.46 in
16	HP 12 x 63 w/shoe	47	Dark Gray, Slightly	Shale	RQD = 0	*295*	*425*	EOID	13.8	- - -	11	547	3	664	94	570	1.21	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Weathered Shale			"	*Rndr = 500*	BOR #1	13.8	5.8	6	483	2	474	94	380	0.98	190	33%	190	29%	5.2 in	None	0.50 in
17	HP 12 x 63 w/shoe	67	Dark Gray, Slightly	Shale	RQD = 0	*295*	*425*	EOID	17.0	- - -	12	614	3	725	75	650	1.18	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Weathered Shale			"	*Rndr = 500*	BOR #1	17.0	7.0	9	563	2	560	111	449	0.99	201	31%	165	23%	> 4 in	None	0.22 in
18	HP 14 x 73 w/shoe	75	Severly Fractured	Limestone	No RQD	*425*	*703*	EOID	42.5	- - -	15	732	2	853	103	750	1.17	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Light Gray Limestone			"	*Rndr = 751*	BOR #1	42.5	1.0	10	738	2	738	108	630	1.00	120	16%	115	13%	> 2 in	None	0.20 in
19	HP 12 x 74 w/shoe	25	Highly Weathered	Shale	No RQD	*284*	*472*	EOID	54.0	- - -	6	785	3	712	96	616	0.91	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Gray Shale			"	*Rndr = 505*	BOR #1	54.0	1.0	5	850	3	591	107	484	0.70	132	21%	121	17%	> 6 in	None	0.40 in
20	HP 12 x 53 w/shoe	32	Highly Weathered	Shale	N=100/0.10 in	*134*	*191*	EOID	37.6	- - -	10	404	3	409	64	345	1.01	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Black Shale			"	*Rndr = 266*	BOR #1	37.6	3.0	6	401	2	316	98	218	0.79	127	37%	93	23%	6.1 in	None	None
21	HP 12 x 84 w/shoe	46	Weathered Hard Black Shale,	Shale	RQD = 0	*307*	*439*	EOID	22.8	- - -	7	838	1	743	40	703	0.89	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Thinly Bedded			"	*Rndr = 539*	BOR #1	22.8	13.9	7	733	2	597	47	550	0.81	153	22%	146	20%	4.4 in	None	0.28 in
22	HP 12 x 84 w/shoe	11	Weathered Black Shale	Shale	N=50/3 in[a]	*307*	*439*	EOID	22.6	- - -	10	831	3	795	45	750	0.96	- - -	- - -	- - -	- - -	- - -	- - -	- - -
						"	*Rndr = 539*	BOR #1	22.6	4.9	6	885	2	635	45	590	0.72	160	21%	160	20%	1.2 in	None	0.28 in
23	HP 14 x 89 w/shoe	42	Weathered Interbedded	Shale	- - -	*410*	*586*	EOID	71.0	- - -	8	964	4	660	265	395	0.68	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Sandstone and Shale			"	*Rndr = 686*	BOR #1	71.0	4.8	8	852	4	560	275	285	0.66	110	28%	100	15%	0.75 in	0.25 in	> 6.0 in
24	24 in. PS Concrete	13	Very Dense Clayey	SP-SC	N=48[s]	*280*	*374*	EOID	38.0	- - -	10	787	3	658	176	482	0.84	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Fine Sand			"	"	BOR #1	38.0	5.1	5	1051	1	460	180	280	0.44	202	42%	198	30%	> 3.75 in	None	None
25	18 in. PS Concrete	33	V Dense Fine Sand to	SP, SP-SM	N=52[a]	*200*	*268*	EOID	26.0	- - -	10	1056	3	810	214	596	0.77	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Fine Sand with Silt			"	"	BOR #1	26.0	12.1	10 / 1.5	1159	2	660	245	415	0.57	181	30%	150	19%	> 1.5 in	None	None
26	18 in. PS Concrete	57	V Dense Fine Sand to	SP, SP-SM	N=35[a]	*340*	*454*	EOID	33.0	- - -	7	992	3	640	135	505	0.65	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Fine Sand with Silt			"	"	BOR #1	33.0	0.9	4	1383	2	586	171	415	0.42	90	18%	54	8%	> 1.75 in	None	None
27	24 in. PS Concrete	8	V Dense, Light Gray,	SP	N=56[s]	*320*	*428*	EOID	32.0	- - -	20	1259	2	935	205	730	0.74	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Fine Sand with Trace Clay			"	"	BOR #1	32.0	47.0	3	1317	2	420	200	220	0.32	510	70%	515	55%	> 3.25 in	None	1.6 in.
28	HP 14 x 73 w/shoe	45	Hard Limestone with	Limestone	N=50/1 in[a]	220	480	EOID	33.0	- - -	7	672	2	490	163	327	0.73	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Thin Shale Seams			"	"	BOR #1	33.0	2.8	3	816	2	425	160	265	0.52	62	19%	65	13%	8.3 in	None	8.3 in.
29	HP 14 x 84 w/shoe	13	Weathered Gray Siltstone	Siltstone	N=50/1 in[a]	*435*	*622*	EOID	26.4	- - -	6	902	3	799	45	754	0.89	- - -	- - -	- - -	- - -	- - -	- - -	- - -
						"	*Rndr = 722*	BOR #1	26.4	3.8	5	841	3	686	46	640	0.82	114	15%	113	14%	6.5 in	0.40 in	1.2 in
30	HP 14x89 w/shoe	108	Severely Weathered	Shale	N=50/5 in[u]	280	644	EOID	80.8	- - -	8	921	7	818	90	728	0.89	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Gray Shale			"	"	BOR #1	80.8	2.8	6	926	2	666	102	564	0.72	164	23%	152	19%	> 7.0 in	None	0.17 in
31	HP 12x74 w/shoe	5	Hard, Gray, Sandy Loam Till	SC-SM	N=50/4 in[a]	280	370	EOID	101.0	- - -	6	743	2	415	223	192	0.56	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			with Gravel			"	"	BOR #1	101.0	1.0	4	842	2	385	236	149	0.46	43	22%	30	7%	> 4 in	None	0.50 in
32	HP 12x84 w/shoe	6	Weathered Gray Shale	Shale	N=50/1 in[a]	*210*	*364*	EOID	26.0	- - -	10	979	3	831	187	644	0.85	- - -	- - -	- - -	- - -	- - -	- - -	- - -
						"	*Rndr = 446*	BOR #1	26.0	4.9	10	944	4	727	197	530	0.77	114	18%	104	13%	> 2 in	None	None
33	HP 14x89 w/shoe	8	Hly Wthrd Soft Limestone	Limestone	N=50/5 in[a]	*435*	*625*	EOID	36.5	- - -	10	1036	3	1067	67	1000	1.03	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			w/Steep Fracture Plane			"	"	BOR #1	36.5	1.0	8	908	4	782	79	703	0.86	297	30%	285	27%	> 2 in	None	0.12 in
34	16 in x 0.5 in CEP	6	V Dense Gravelly Sand	SM	N=50/0 in[a]	*318*	*454*	EOID	16.0	- - -	9	840	2	661	51	610	0.79	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			with Cobbles			"	"	BOR #1	16.0	0.9	8	796	3	550	80	470	0.69	140	23%	111	17%	> 4 in	None	0.25 in
35	24 in PSC, HP12x53 Toe	55	V Dense, CMF Sand,	SW-SM	N=86[a]	500	1000	EOID	47.6	- - -	4	1830	2	1110	65	1045	0.61	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Little Gravel, Trace Silt			"	"	BOR #1	47.6	2.9	4	2133	4	875	185	690	0.41	355	34%	235	21%	> 299 in	0.5 in	> 299 in
36	24 in PSC, HP12x53 Toe	141	V Dense, CMF Sand,	SW-SM	N=74[a]	850	1700	EOID	57.8	- - -	11	2029	8	1800	190	1610	0.89	- - -	- - -	- - -	- - -	- - -	- - -	- - -
			Trace Silt			"	"	BOR #1	57.8	2.9	4	2897	5	1490	420	1070	0.51	540	34%	310	17%	> 22.8 in	None	22.5 in

Notes: EOID = End of Intital Driving, BOR = Beginning of Restrike, TBD = To Be Determined, a = automatic SPT hammer, s = SPT safety hammer, u = unreported SPT hammer type, Pen = embedded depth in ground, * = concrete filled pipe