Time-related increase in bearing resistance of friction piles

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ABSTRACT

The bearing resistance of friction piles usually increases over time after installation. Recently many studies have been made about time-related increase in bearing resistance of friction piles in several countries. Though, in Finnish soil conditions there are little knowledge of time related increase in bearing resistance. The aim of study was to examine the time dependency of friction pile's bearing resistance in Finnish soil conditions. The study consist of experimental and literature parts.

In the experimental part the test results of two test piling sites are introduced and analyzed. Test pilings were conducted at bridge sites which were located in railway from Liminka to Oulu. Soil conditions at the test sites are challenging as the bedrock surface is located in 60–140 m depth based on investigations by Geological Survey of Finland (Breilin & Putkinen 2012). The test piles were close ended steel pipe piles and driven precast concrete piles. The test piles were investigated by dynamic load testing in four phases: the first phase at the end of driving (EOD), the second phase about 24 hours after the EOD, the third phase about 14 days after the EOD and the last phase about 28 days after the EOD. Signal matching for measured signals was also performed.

The study shows that the bearing resistance of friction piles showed significant increase with time. The test results indicate that the major increase happens in two weeks, but also after two weeks the increase is noticeable. According to the literature study the bearing resistance increase will continue over 100 days after EOD.

Keywords: Friction pile, test piling, dynamic load test

1 INTRODUCTION

Railway level crossings are removed and replaced with bridges on railway from Liminka to Oulu. Soil conditions at the bridge sites are challenging as the bedrock surface is located at 60-140 m depth and the layers above the bedrock are loose sands or silts.

Due to the challenging ground condition it is not cost effective to drive piles to the bedrock. Bridge construction piles are designed to work as friction piles and the required pile resistance could not be achieved right after pile driving when major part of the pile resistance comes from the pile toe which is in loose sand or silt layer.

However, piles' bearing resistance usually increases over time. Based on the literature study the time-dependent increase in bearing resistance of frictions piles can be divided into two main causes; stress relaxation (Chow et al., 1996) and soil ageing (Schmertmann, 1991).
In Finnish soil conditions there are little knowledge of time related increase in bearing resistance of friction piles. In Finland end bearing piles are usually used instead of friction piles. The main reason for scarce use of friction piles in Finland is that usually the bearing bottom layer is achieved with relatively short pile length (10–20 m). One reason might also be the lack of knowledge of friction piles and their working principles. In Finland there are also favourable areas for the use of friction piles. Friction piles could be used at the esker margins and regions where thick and loose non-cohesive soil layers exist.
The aim of study was to examine the time dependency of friction pile's bearing resistance in Finnish soil condition. The study consist of experimental and literature parts. In the experimental part the test piles were investigated by dynamic load testing and signal matching.

2 TEST SITES

Test sites are located in Finland, Northern Ostrobothnia. Zatelliitti test site is located in Kempele and Tuuliharju test site is located in Liminka.
Both test sites are at the area of Muhos transition zone which was formed during the last glaciation. At the Muhos transition zone the bedrock surface is located in 60-140 m depth based on investigations by Geological Survey of Finland (Breilin & Putkinen, 2012). Comprehensive geotechnical investigations have been performed at both test sites. Geotechnical investigations included soundings and soil samples.

2.1 Zatelliitti

In Zatelliitti test site the soil surface is at the level of +6,5 and the ground at the test site is flat arable land. Ground water surface is at level +5,5. Working platform is made from 0,5 m thick crushed aggregate.
The soil is sand and sandy silt at the top 2 m. Underneath the top layer is about 6 m thick layer of clay and silt. From the level -1 begins about 30 m thick layer of silt. This layer is partially sandy and partially clayey. Under the silt layer exists about 25 m thick dense layer of sand. The sand layer is uniformly graded.
Underneath the sand layer is extremely dense layer of sandy moraine which alternates to sandy moraine at the level -60. Soil sampling was ended at the level of -60. Sounding diagrams from Zatelliitti test site are presented in figure 1.
At the test site dynamic probing ended at the level -30. The deepest static-dynamic penetration test reached level -56. The static-dynamic penetration test is same as dynamic probing but the sounding rod is rotated during the driving. Rotating the rods during driving ensures that the rod goes straight and same time rod penetrates the soil more easily.

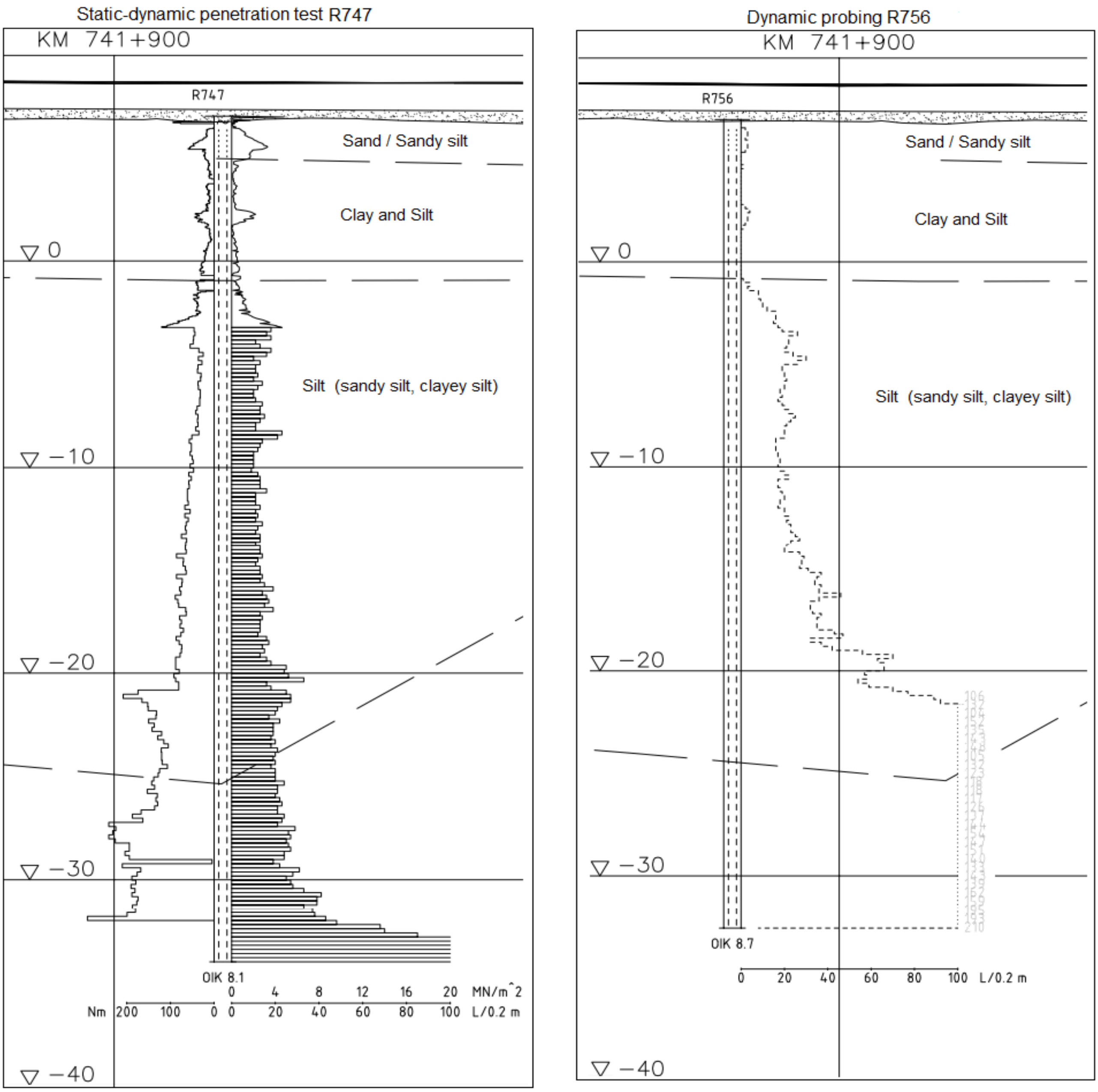

Figure 1 Sounding diagrams from Zatelliitti test site. Distance between investigation points is 10 m.

2.2 Tuuliharju

Soil surface level at Tuuliharju test site is between +4,9–5,6 and ground water level is about 1 m below the soil surface. Working platform is 0,5 m thick. The test site is surrounded by flat arable and forest areas. The soil consists of a thin layer of topsoil at the top. Underneath the topsoil is 1,5 m thick layer of crust. Under the crust is a layer which contains silt, clayey silt and lean clay. This layer grows thinner from the level -20 to level -8 when going towards north parallel to track.

Under the variable thick layer exists layer of sandy silt, silty sand and fine sand. Density of the layer varies from middle dense to extremely dense. Layer from level -25 to level -45 varies from sand moraine to fine sand and fine silt. Sounding resistances from Tuuliharju test site are presented in figure 2. At the test site deepest sounding reached the level -34,5, but typically soundings ended at the level -25.

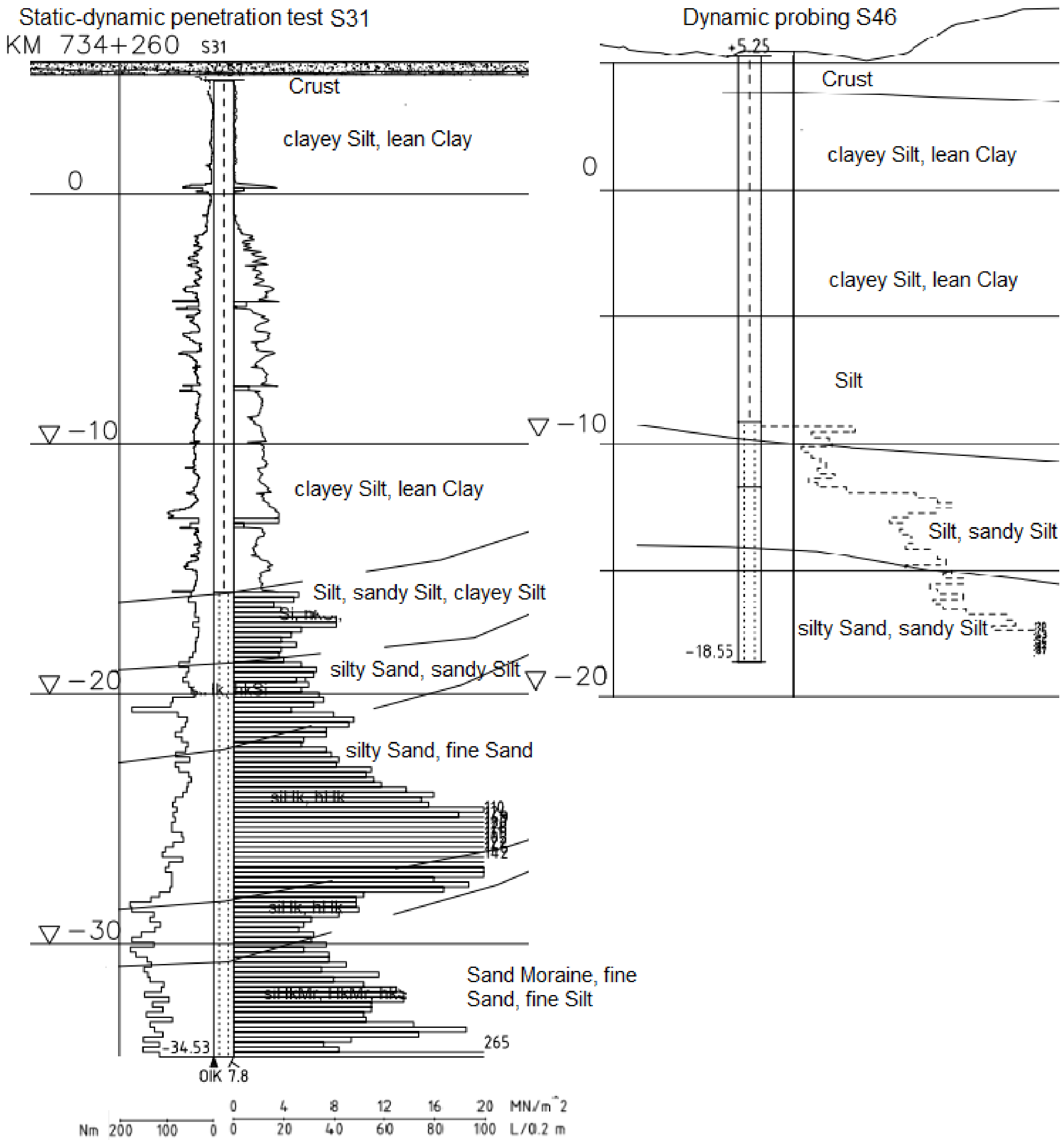

Figure 2 Sounding diagrams from Tuuliharju test site. Distance between investigation points is 32 m.

3 TEST PILING

Test piles were close ended steel pipe piles (diameter 323,9 mm and wall thickness 10 mm, steel grade S440J2H) and driven precast concrete piles (TB300b, 300x300 mm²). There were six steel pipe piles and four precast concrete piles at the Zatelliitti test site and three steel pipe piles and two precast concrete piles at Tuuliharju test site.

At Zatelliitti test site the test piles are divided into north and south groups, both including five piles. Distance between piles inside the group is 5 m and distance between north and south group is 42 meters at shortest. Test piles were driven using Junttan HHK 5A hydraulic hammer (hammer weight 5000 kg, maximum drop height 1,2 m). Information of the test piles is presented in table 1. Pile number starting with letter Z means Zatelliiti test site piles and letter T means Tuuliharju test site pile.

Table 1 Test pile information.

Pile number	Pile type	Element length [m]	Tip level	End of drive criteria s/10 [mm]
ZET1	Steel pipe pile D323,9/10 mm S440J2H	16+16	-23	151
ZET2	Steel pipe pile D323,9/10 mm S440J2H	16+4+16	-27	123
ZET3	Steel pipe pile D323,9/10 mm S440J2H	16+8+16	-30	30
ZPT4	Steel pipe pile D323,9/10 mm S440J2H	16+7+16	-30	184
ZPT5	Steel pipe pile D323,9/10 mm S440J2H	16+3+16	-26	268
ZPT6	Steel pipe pile D323,9/10 mm S440J2H	16+13	-20	254
ZEB1	Precast concrete pile 300x300 mm²	15+3+14	-23	175
ZEB2	Precast concrete pile 300x300 mm²	15+7+15	-28	139
ZPB3	Precast concrete pile 300x300 mm²	15+14	-20	210
ZPB4	Precast concrete pile 300x300 mm²	15+4+15	-25	147
TU-T1	Steel pipe pile D323,9/10 mm S440J2H	16+12	-21	70
TU-T2	Steel pipe pile D323,9/10 mm S440J2H	16+3+9	-20	35
TU-T3	Steel pipe pile D323,9/10 mm S440J2H	16+6+4	-19	26
TU-B1	Precast concrete pile 300x300 mm²	15+12	-20	30
TU-B2	Precast concrete pile 300x300 mm²	15+10	-18	*

*was not measured

At the Zatelliitti test site the pile driving was easy until the level of -27 was reached and the driving resistance increased significantly. At Tuuliharju test site the driving resistance was low until the level -15 was reached. Hammer drop height at the time when end of drive criteria was measured was 30–50 cm for steel pipe piles and 20–30 cm for precast concrete piles. Steel pipe piles were filled with water to 2 m below the surface level to prevent the buoyancy force effect.

4 DYNAMIC LOAD TESTS AND SIGNAL MATCHING

The test piles were investigated by dynamic load testing in four phases: the first phase at the end of driving (EOD), the second phase about 24 hours after the EOD, the third phase about 14 days after the EOD and the last phase about 28 days after the EOD. Signal matching for measured signals was performed for last three phases.

The meaning of second phase was to determine the short-term resistance increase. Meaning of third and fourth phase was to investigate the long-term resistance increase. Long-term resistance increase is expected to happen due to stress relaxation (Chow et al., 1996) and soil ageing (Schmertmann, 1991). Short-term resistance increase is expected to happen mainly due to dissipation of pore water pressure. Stress relaxation and soil ageing also start right after the pile driving.

Piles were driven and end of driving dynamic load testing was performed 2.–4.3.2015 for all 15 test piles. Second phase was performed 3.3–5.3. (19–31 hours after EOD). Same accelerated hydraulic hammer Junttan HHK 5A was used in second phase. For all the test piles full geotechnical bearing resistance could not be mobilized in second phase with using the said hydraulic hammer due to insufficient energy.

The third phase was conducted on 18.3. using a heavier Junttan HHK 7A hydraulic hammer (hammer weight 7000kg, maximum drop height 1,2 m). Even with that hammer full geotechnical resistance could not be mobilized for all test piles. Pile TU-T2 was driven 1 m deeper at the time of third load testing phase.

The fourth phase was conducted on 31.3. 10 000 kg drop hammer was used in fourth phase. Precast concrete piles were investigated by dynamic load testing only in first three phases because the structural capacity was achieved at the third phase.

The amount of blows per pile during the restrike tests was limited to 2-4 blows so that the dynamic load testing would disturb the set-up as little as possible. Dynamic load testing results as well as signal matchings from all the four phases are presented in table 2.

Table 2 Test piles' geotechnical capacity with Case Method and CAPWAP.

Pile Number	LE [m]	Time after installation	Geotechnical Capacity, Case Method [kN]	Geotechnical Capacity, CAPWAP [kN]			Settlement per blow [mm]	EMX [kNm]
				Total	Shaft	Toe		
ZET1	30,9	EOD	301	301			75	68,9
		29 hours	1148	1075	821	254	25	60,6
		15 days	2551	2619	1977	642	8	68,1
		28 days	3036	2965	2157	808	22	165
ZET2	35	EOD	1363	1363			18	63,7
		31 hours	1917	1882	1196	686	9	54,3
		15 days	2436	2494	1819	675	2	69,9
		28 days	3327	3386	2314	1072	10	151
ZET3	38,8	EOD	1814	1816	863	953	6	59,7
		30 hours	2078	2116	1477	639	4	61,9
		15 days	2549	2614	1947	667	1	70,2
		28 days	3561	3688	2350	1338	3	122,9
ZEB1	31	EOD	864	864			31	35,3
		23 hours	1537	1490	747	743	12	44
		15 days	2463	2528	1628	900	6	59,4
ZEB2	27,5	EOD	977	977			24	33,3
		24 hours	1356	1070	281	790	17	44,6
		15 days	1954	1626	734	891	10	62,8
ZPT4	37,7	EOD	454	572	378	194	45	63,4
		25 hours	1217	1238	1009	229	19	62,2
		16 days	2555	2450	1943	507	6	72,2
		28 days	3533	3489	2716	773	17	159,7
ZPT5	33,6	EOD	480	480			90	74,2
		23 hours	1366	1244	957	287	23	64,4
		16 days	2726	2676	1849	827	7	71,6
		28 days	3450	3441	2492	949	8	137,8
ZPT6	27,6	EOD	303	303			90	75,4
		21 hours	1253	1270	922	347	29	61,7
		16 days	2738	2755	1698	1057	4	71,9
		28 days	3722	3575	2477	1098	12	136,2
ZPB3	27,5	EOD	283	283			39	21,4
		21 hours	1203	1219	970	249	4	37,7
		16 days	2518	2372	1588	784	6	58,1
ZPB4	32,8	EOD	654	654			55	41,7
		22 hours	1629	1590	1020	570	6	39,9
		16 days	2526	2658	1755	903	5	56,2
TU-T1	30,7	EOD	1711	1751	1027	724	17	64,3
		26 hours	2119	2157	1339	818	11	68,2
		14 days	2560	2599	1771	828	7	77,6
		28 days	3085	3053	2140	913	20	145,1
TU-T2	31,6	EOD	1958	1957			10	62,9
		24 hours	2265	2398	1233	1165	8	66,9
		14 days	2774	2792	1617	1175	5	69,5
		28 days	2830	2810	1613	1197	24	152,3
TU-T3	32,7	EOD	1595	1595			16	68,3
		22 hours	1746	1770	487	1283	13	73
		14 days	2314	2273	962	1311	15	69,9
		28 days	2479	2470	1190	1280	25	136,4
TU-B1	25,9	EOD	1530	1530			8	37,7
		20 hours	1635	1662	953	709	8	46,2
		14 days	1966	2100	1308	792	6	60,8
TU-B2	24,2	EOD	847	847			18	25,9
		29 hours	1079	1265	618	647	10	32,4
		14 days	1605	1610	900	710	11	63,6

LE is the pile length below gauges. EMX is maximum energy transferred to pile. Case Method capacity is used in CAPWAP total column if CAPWAP-analysis is not done for that particular load test.
Based on the observations during the dynamic load testing, the required settlement per blow to fully mobilize the geotechnical resistance is assumed to be over 10-15 mm. It can be seen from table 2 that the full geotechnical resistance could not be mobilized for every pile in every phase. Zatelliitti test piles' bearing resistance development is presented in figure 3.

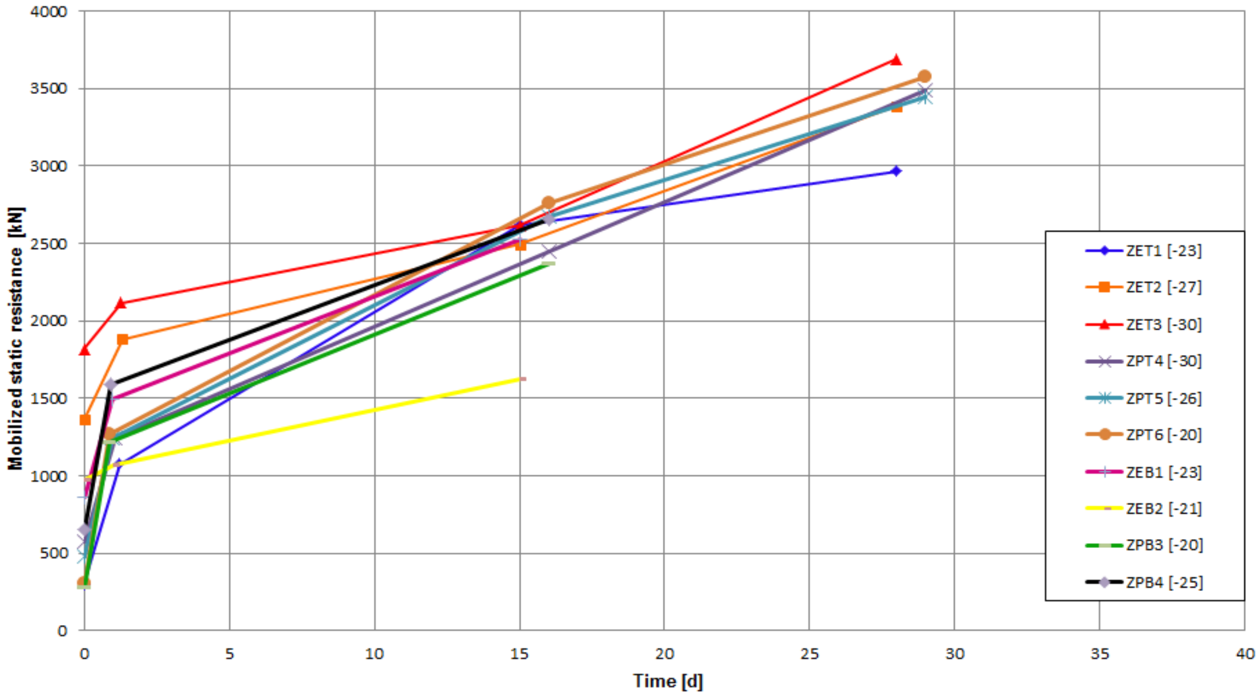

Figure 3 Bearing resistance developments of Zatelliitti test piles.

Shape of the curves for most piles in figure 3 does not truly present the correct time-related bearing capacity behavior because the bearing resistance was not fully mobilized for all of the piles in second and third phase. EOD and most of the 28 d values are correct, but the shapes of the curves between those points are different. At the beginning the increase should be intensive and then equalize as the time goes by.
According to the literature study the bearing resistance increase will continue over 100 days after EOD (Chow et al., 1998; Axelsson, 2000). The main reason for the bearing resistance increase was due to increase in shaft resistance. Shaft resistance development of Zatelliitti steel pipe piles is presented in figure 4. Toe resistance development is not presented because the degree of mobilization for toe resistance varied greatly. Axelsson (2000) showed that the toe resistance was almost constant and did not increase as the shaft resistance did.

One of the precast concrete piles, pile ZEB2, suffered from increasing pile damage during driving. Development of the damage severity was observer from the stress wave graph as the pile driving went on. The pile damage located approximately in the middle of the lowest pile segment which then eventually broke in half. So the pile length shortened and bearing resistance was lower than the bearing resistance of other precast concrete piles, which can be seen in figure 3.

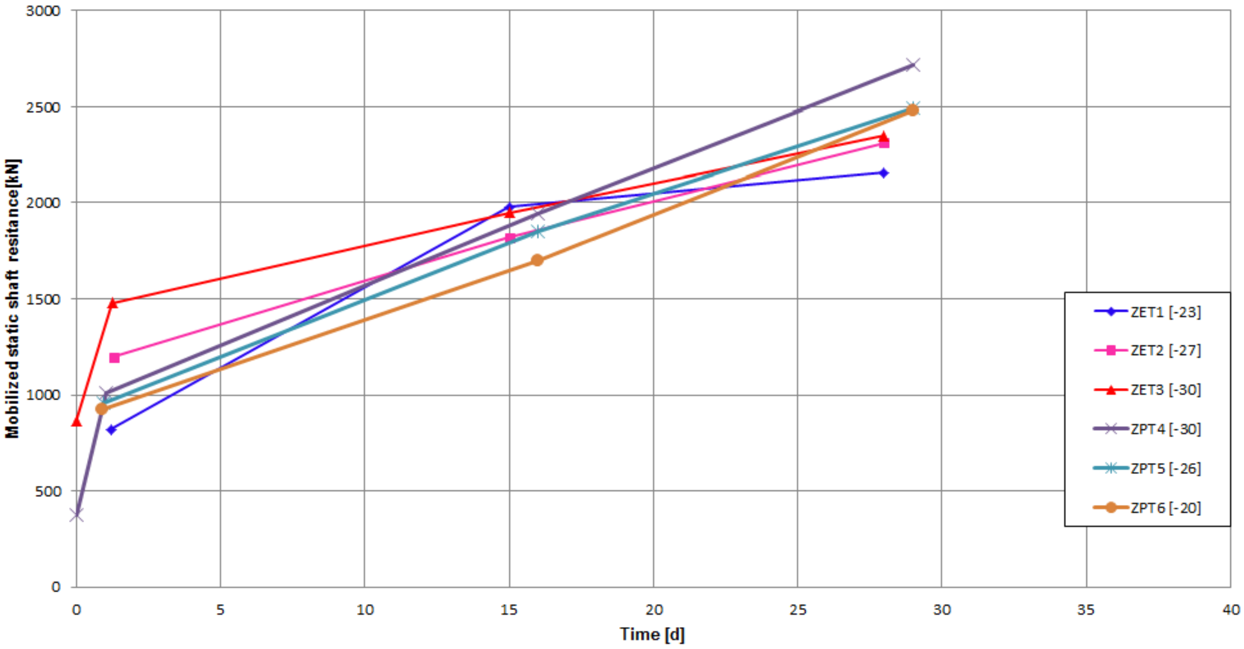

Figure 4 Shaft resistance developments of Zatelliitti steel pipe piles.

It was earlier said that the amount of blows was kept to a minimum so that the set-up would not be disturbed, but in phase 3 piles ZET1 and ZPT5 were disturbed with a great amount of blows. The meaning of the blows was to mobilize the toe resistance, but piles behaved differently to that. It can be clearly seen that the set-up of pile ZET1 was disturbed but the set-up process of pile ZPT5 continued almost normally despite the great amount of blows.

Pile ZPT6 has almost the same shaft resistance as the pile ZPT4 which is 10 meters longer. One explanation to this is that, stiffer soil layers exist on higher level at the location of pile ZPT6.

Pile ZET2 shaft resistance increase as a function of driven pile length is shown in figure 5. Shaft resistance increase curves are based on CAPWAP-analyses. Presented curves are from the last three phases.

It can be seen that the shaft resistance increased below 20 m depth where denser layers exists. Shaft resistance remained almost constant at the upper part of the pile, near the soil surface. Shaft resistance clearly increased as a function of depth in all phases.

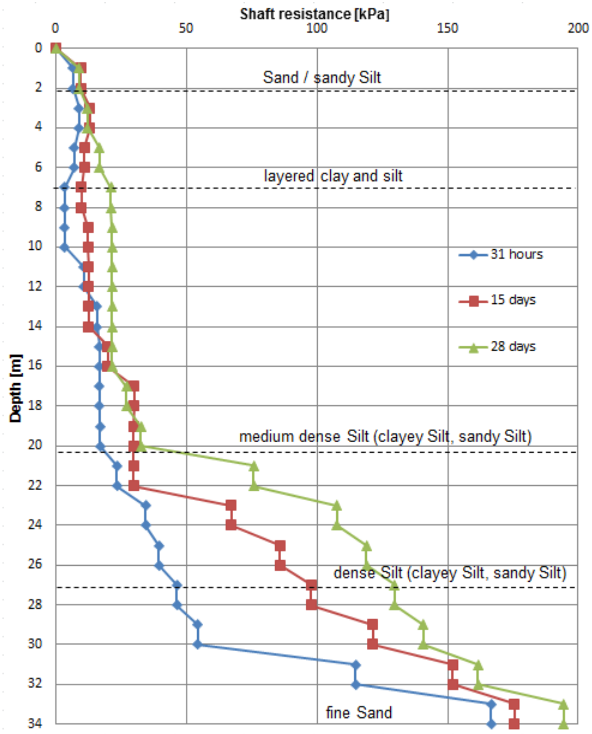

Figure 5 Pile ZET2 shaft resistance increase as a function of driven length.

Tuuliharju test piles' bearing resistance development is presented in figure 6.

Pile TU-T2 was driven 1 m deeper during third phase and that is why the bearing resistance remained constant between third and fourth phase. At Tuuliharju site the increase of bearing resistance was more moderate than at Zatelliitti site. At Tuuliharju site soil layers are mostly fine grained, clay or clayey layers where as in Zatelliiti site layers are mainly coarser grained. It is possible that the set-up process only takes longer in fine grained soils than in grained soils. Long-term pile set-up results from this study and previous studies are presented in figure 7.

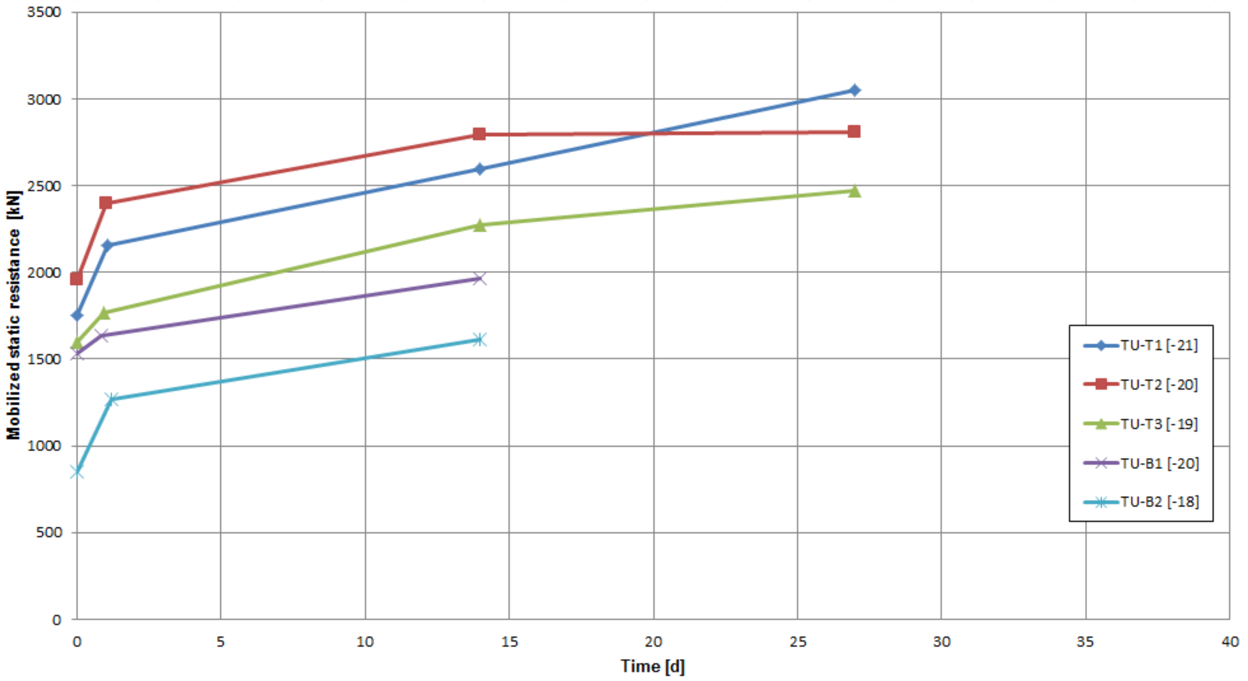

Figure 6 Bearing resistance developments of Tuuliharju test piles.

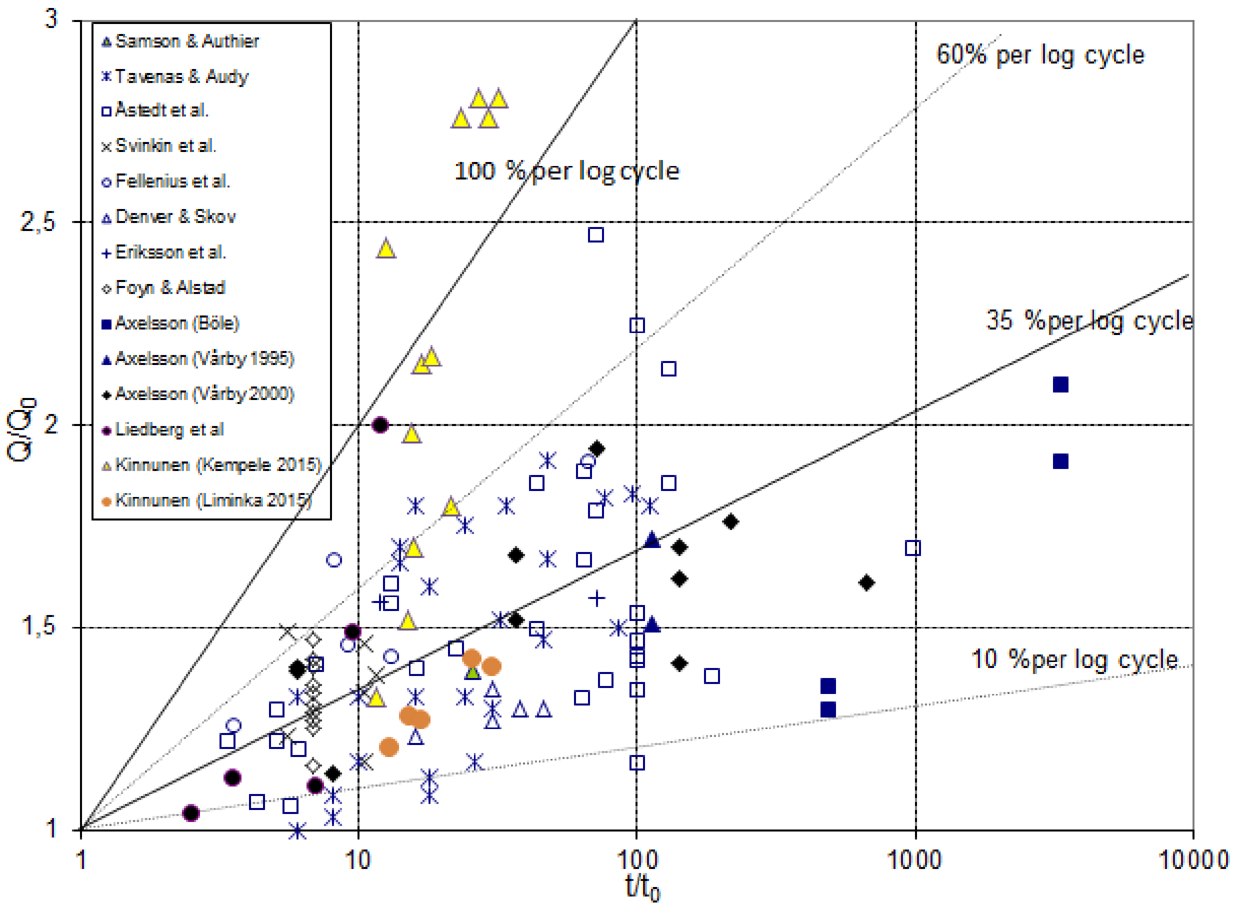

Figure 7 Long-term pile set-ups. Results from Oulu added to original chart presented by Axelsson (2000).

Dynamic load test results where the settlement per blow was less than 10 mm at the second phase are not presented in figure 7. If those results had been presented it would have given incorrect results when the Q-ratio rises. Those pure friction piles from Zatelliitti test site showed over 100 % increase per log cycle. Others Zatelliitti test results are placed on the same area on the chart as the results from the previous studies. Tuuliharju test piles showed fairly moderate increases.

5 CONCLUSION

Based on the experimental and literature studies the magnitude of bearing resistance increase of friction piles' depends on: soil conditions, pile installation method, pile properties and load history of the pile.
Study results shows that the pile bearing resistance increased 17–180 % after the dissipation of pore water pressure. At the Tuuliharju test site the increase was more moderate than at the Zatelliitti test site. Although there were more scatter in the increases at Zatelliitti site because some of the piles were end bearing piles with a high bearing resistance at the end of driving and the rest were friction piles with a low end of driving bearing resistance. In Tuuliharju all the test piles worked almost like end bearing piles.
The test results indicate that in cohesionless soils the major increase in bearing resistance happens in two weeks, but also after two weeks the increase is noticeable.
In Finland the use of friction piles is limited due to Finnish geology. In suitable soil condition friction piles are a rational alternative for end bearing piles and remarkable costs in piling projects can be saved if the increase in bearing resistance of friction piles is taken into account. Although, the set-up time and further load tests should be taken into account in friction pile planning and scheduling.

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