Onshore and Offshore Pile Installation in Dense Soils

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Abstract: Open-ended pipe piles are often used for the foundations of both on-land and offshore structures because of their relatively low driving resistance. Piles are usually subjected to the highest level of stresses during installation. Three case histories for overwater bridge pile damage during installation are presented in detail. Also, several case studies for onshore and offshore piles installed in dense soils are compiled and analyzed in an attempt to improve the available guidelines. Based on field data analyses for many case studies of piles installed in dense soils, a limitation for pile diameter to thickness ratio adjusted for driving energy is proposed. A maximum driving stress at the pile head and toe of about 50% of the steel yield stress should be considered for piles installed into very dense soils. Also, general guidelines and recommendations from a design and construction prospective are provided.
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NOTATION

A_s	Cross sectional area
d	Pile outside diameter
F_{hc}	Critical hoop buckling stresses
F_y	Yield Stress
L	Pile penetration length
t	Pile wall thickness
CSX	Maximum compressive stress at sensors
CSB	Maximum compressive stress at pile toe
En	Energy
EMX	Maximum transferred energy at sensors
ETR	Energy transfer ratio

1. Introduction:

Bridges, offshore platforms, marine structures and different structures are sometimes installed in dense sands. With high demands on piles, high axial and lateral pile capacities are required, which results in the need to penetrate deeper in denser soils. In general, geotechnical pile capacity calculations are based on soil properties, pile diameter and length. Preliminary design and analysis conducted by many practitioners does not usually take into consideration the potential problems encountered during pile installation through strata that may contain cobbles and boulders. Initial attempts of pile installation in these soils have been frequently unsuccessful. Consequently, construction delays and costs increase substantially.

Although general guidance on pile driving is provided in the literature but insufficient information is available for pile installation in very dense soils.

This paper presents three case histories for pile damage during driving into very dense soils with some cobbles and/or boulders. Pile Dynamic Analyzer (PDA) tests and Case Pile Wave Analysis Program (CAPWAP) analyses were conducted to confirm pile capacities, evaluate the driving system and to determine the location of pile damage if any.

Observations and data analyses from the three presented case histories and from several other projects at different onshore and offshore sites comprising dense soils are compared to published guidelines and literature. Based on the data analyses, a limitation for pile diameter to thickness ratio adjusted for driving energy is proposed. Also, a limit for maximum driving stress at the pile head and toe is proposed.

2. Background Information

General reasons of pile damage during driving include the use of inappropriate hammer, insufficient cushion, tight pile cap, misalignment between pile and driving system, difficult driving conditions, obstructions in the ground, uneven contact between hammer and pile head and concentrated soil resistance (**Hussein and Goble, 2000**).

Wave equation analysis is very important for the preparation of pile driving, selection of hammer, cushion and determination of bearing capacity and driving set criteria. Dynamic measurements using Pile Driving Analyzer (PDA) is preferred to evaluate the performance of hammer-cushion-pile-soil system during driving. PDA testing has been well accepted worldwide for onshore industry projects. Dynamic testing has been also used in the oil and gas industry for offshore piles and has been reported in some publications (**Harnar and Likins, 1996, Webster *et al.*, 2008 and Rusche *et al.*, 2009**).

From a design prospective, main factors should be considered to reduce the risks of steel pipe pile damage during driving. These factors are mainly: hammer type and energy, pile diameter (d), pile thickness (t) and pile yield stress.

A brief review of available guidelines in some codes and the literature for selection of pile diameter (d) to thickness (t) ratio, d/t, and of limiting hammer energy and pile buckling stresses to certain levels is presented in the following subsections.

2.1 Hammer energy

The minimum pile wall thickness recommended by the American Petroleum Institute (**API, 2007**) for offshore piles subjected to hard driving (250 blows/300 mm) is given as:

$t = 6.35 + d/100$ (1)

where t and d are the wall thickness and pile diameter in mm, respectively. Based on the above equation, d/t ratio for typical offshore steel pipe piles ranges from 47 to 82. However, **API (2007)** recommends decreasing d/t ratio with increasing hammer energy as shown in Figure 1. This figure presents the relation between d/t and rated hammer energy divided by the pile cross sectional area (En/A_s).

The Canadian Foundation Engineering Manual (**CFEM, 2006**) recommends limiting the rated hammer energy to 6000 kJ times the pile cross sectional area. It states that pile head damage is not usually induced from large impact force, but from misalignment of the pile in the helmet and non-concentric impacts.

2.2 Local Buckling Stress

Elastic and inelastic local buckling stresses are discussed in **API (2007)**. The inelastic buckling stress, which is considered more critical, depends on the steel yield stress, F_y, and d/t ratio.

2.3 Hoop Buckling Stress

The elastic and critical hoop buckling stresses for cylinders can be calculated using the **API (2007)** guidelines for structural steel design. The main parameters affecting the critical hoop buckling stresses are the elastic hoop buckling stress, d/t ratio, yield stress for steel and the length of cylinder between stiffening rings or end connections.

2.4 Denting Damage

As described in the offshore technology report provided by the Health and Safety Executive (**HSE, 2001**) a theoretical comparison with **Ellinas and Walker (1983)** was conducted to determine the load causing damage in the form of a dent or dental damage in tubular members due to handling operations. The maximum load applied to a cantilevered end of a pile depends on steel yield stress, pile diameter, wall thickness and the dent depth. As discussed in **HSE (2001)**, the subsequent loading during driving could lead to enlargement of the initial dental damage.

3. Case Histories of Pile Damage During Driving

In the following subsections, three case histories of steel pipe pile damage during driving in dense sand with some cobbles and/or boulders are reported. The author was directly involved in these case histories. The three case histories were located in British Columbia, Canada.

3.1 Case History 1

Design and construction phases of a river bridge in the Lower Mainland, British Columbia, Canada were carried out between 2004 and 2005. The bridge comprised two abutments and three piers. The piers were supported on five piles each and the abutments were supported on 12 piles each. Due to environmental concerns, only limited geotechnical investigation was feasible. The investigation included some test holes and penetration resistance profiling using a Becker Hammer drill.

The soil at the site comprised compact to dense sand and gravel with some cobbles and boulders over very dense sandy gravel to gravely sand. Figure 2 shows the soil conditions at the site, required pile penetration length and Becker Penetration Test (BPT) results corrected to N_{60}.

Without consideration of potential driving stresses, piles with an outside diameter, d, of 1219 mm and a wall thickness, t, of 12.7 mm (d/t=96) were selected and then pre-ordered by the owner. Fifteen spiral welded open ended steel pipe piles were required to be driven to 10 to 12 m penetration. The piles were Grade 3 steel with a yield stress of more than 300 MPa. An internal driving shoe of about 2 m length was utilized to reduce the risk of pile toe damage. Before construction, the boulders were removed by excavation to the depth indicated on the test hole logs. However, during pile driving, it appeared that some cobbles/boulders were present above the anticipated depth of pile toe and well below the depth encountered in the test holes.

Before construction, wave equation analyses were performed using the software program GRLWEAP 2005 to select the driving hammer and to determine the required criteria (blows/300 mm) to achieve the required axial pile working capacity of 4000 kN. The analyses indicated that the output is very sensitive to the choice of toe quake value which is usually assumed to be between d/60 to d/120. The analyses indicated that a hammer with a rated energy of 223 kJ such as Delmag D60-62 is needed to achieve the required capacity with a factor of safety (FoS) of at least 2. However, the analyses indicated that it may be very difficult to install the pile to the required capacity depending on the toe quake value. The predicted impact driving stress was about 200 MPa, which is below 0.7 times the pile yield stress (F_y).

Although precautions were taken during pile driving, the pile head and toe were damaged locally at a resistance of about 90 blows/300 mm using a Delmag D60-62. Figures 3 and 4 present photos of pile head and toe damage, respectively. To reduce impact stresses and potential damage, it was decided to drive the remaining piles with a Delmag D30-32 diesel hammer, which has a rated energy of about 102 kJ. No additional evidence of pile head damage was observed. However, the toes of six piles were damaged even before reaching the required driving resistance of 200 blows/300 mm. Furthermore, one of the piles experienced toe failure at about 40 blows/300 mm. All damaged piles had to be replaced and driven again.

To avoid further damage to the pile shells, different methods were utilized to install the piles. At several locations, vibratory hammer was used to advance the piles until it reached obstruction, then churn drilling was conducted to the required pile penetration depth, and then the piles were seated (driven) using the D30-32 diesel hammer. The churn drilling was very slow and time consuming. At one pile, a 508 mm diameter pile (insert pile) was driven inside the 1219 mm diameter pile so that both piles would be able to achieve the required axial capacity.

During pile driving, pile dynamic analyzer (PDA) tests were conducted on three piles, located on two different bridge piers, to confirm their axial capacity at restrike and to allow the use of a FoS of 2. The Delmag D62-22 diesel hammer and a 44.5 kN drop hammer were used during the testing. Table 1 presents summary data of the three tested piles and hammer information.

Table 2 presents the dynamic test data including transferred energy, hammer stroke, equivalent penetration resistance values, maximum compressive stresses at sensors (pile head) and at pile toe. Case Pile Wave Analysis Program (CAPWAP) analyses were performed on the three tested piles to interpret the mobilised static resistance, smith damping factor and quake values. Summary of CAPWAP results are shown in Table 2. Figure 5 shows the maximum compressive stress, maximum transferred energy and maximum velocity along the length of Pile No. 1 derived from CAPWAP analyses.

The impact force on the three tested piles ranged between about 7600 to 9500 kN. The maximum compressive stresses measured at sensors close to pile heads (CSX) ranged between 158 to 195 MPa. The maximum measured compressive stresses at pile toes (CSB) ranged between 135 and 151 MPa. The mobilized vertical compression resistance ranged between about 5600 and 7000 kN. The interpreted toe quake values ranged from about 6.6 and 10.9 mm. The mobilized toe resistance accounted for about 70% to 80% of the total static resistance. As expected, the hammers could not fully mobilize the tested piles as dynamic testing is not designed to effectively mobilize the soil plug.

Another useful application of PDA testing is that it can detect the location of pile damage. During PDA monitoring of the three tested piles, damage was observed within the lower 1.0 to 1.5 m of Pile 2 as shown in Figure 6.

One of the lessons learnt from this case history is that lack of geotechnical investigation can be very costly. For this case, the pile diameter to thickness ratio, d/t was high. It is believed that a lower d/t ratio would have helped reduce the risk of pile damage. It was also learnt that pile heads and toes could experience significant damage even with a maximum driving stresses of less than $0.7F_y$, which is normally used as a driving stress limit, and without exceeding the design set criteria.

3.2. Case History 2

An overwater bridge in the Lower Mainland, British Columbia, Canada was designed and constructed between 2004 and 2005. The bridge comprised two abutments supported on eight piles at each side. A very limited field geotechnical investigation was conducted due to environmental concerns. The soil at the site comprised dense to very dense sand and gravel to about 8.5 m depth over till-like soil comprising silt and very dense sand with some gravel and cobbles to the end of borehole depth at refusal at about 10 m. Figure 7 shows the soil conditions at the site, required pile penetration length and Becker Penetration Test (BPT) blow counts corrected to N_{60}.

The piles had an outside diameter of 508 mm and a wall thickness of 15.9 mm (d/t=32).

Sixteen open ended steel pipe piles were required to be driven to nominally 10 m depth such that pile toes penetrate about two pile diameters into the till-like soil. The piles were Grade 3 steel with a yield stress of more than 300 MPa. The required axial pile working capacity was 1400 kN. Wave equation analyses were performed using the software program GRLWEAP. Based on the analyses, a Delmag D30-32 diesel hammer was selected to drive the piles to the required depth. The analyses were performed to obtain a factor of safety (FoS) of 3 assuming toe quake value ranging from 5 to 12 mm. The maximum calculated compressive stress ranged between about 150 to 210 MPa. The termination criteria was selected to be 200 blows/300 mm.

After driving the first pile, a PDA test was conducted on that pile to confirm pile compressive resistance, hammer energy and efficiency at restrike. Table 3 presents summary data of the tested pile and hammer information. Table 4 presents the PDA data and results of CAPWAP analyses performed on the tested pile. Figure 8 shows the maximum compressive stress, maximum transferred energy and maximum velocity along pile length derived from CAPWAP analyses.

The maximum transferred energy at sensors near the pile head was about 52 kJ. The maximum compressive stresses at pile head and toe were 223 and 221 MPa, respectively, which were about $0.7F_y$. The mobilized static resistance was 4100 kN indicating that a FoS of about 2.9 was achieved.

Pile driving continued using the same diesel hammer. Almost 50% of the piles experienced local toe failure at resistance of 200 blows/300 mm or less. The main reason for pile damage is believed to be the presence of obstructions during driving. No evidence of pile head damage was observed. All damaged piles had to be pulled out and driven again. To avoid pile damage in the second attempt of installation, churn drilling was conducted to the required depth, and then the piles were seated (driven) using the Delmag D30-32 diesel hammer.

3.3. Case History 3

A bridge over a creek in British Columbia, Canada was constructed in 2005. The bridge abutments were supported on five, Grade 3, 610 mm diameter by 12.7 mm wall steel pipe piles (d/t=48). The required pile penetration length was about 20 m. The top 3.5 m were cleaned out and filled with concrete. The design vertical dead and dead plus live load for each pile was 720 and 950 kN, respectively.

Test holes indicated that soil conditions comprise loose to compact sand and gravel with some cobbles to about 6 m depth over dense to very dense sand and gravel with some cobbles to 14 m depth underlain by hard silt and very dense silty sand. The logs indicated that the maximum particle size was less than 100 mm. Figure 9 shows the soil conditions at the site, required pile penetration length and BPT blow counts corrected to N_{60}.

Wave equation analysis was performed using the software program GRLWEAP. The analysis considered using a 44.5 kN drop hammer, 2.75 m drop height inducing an equivalent energy of 60 kNm and using 102 mm thick oak or hardwood cushion. To achieve a Factor of Safety (FoS) of 3 on working load, a driving resistance of at least 130 blows/300 mm was predicted. As PDA testing was planned for, a FoS of 2 on the working load was specified. To achieve a FoS of 2 a driving resistance of at least 25 to 35 blows/300 mm was predicted.

PDA tests were conducted on the first two driven piles, named here as Pile 1 and 2, at restrike. Table 5 presents summary data of the two tested piles and hammer information. Table 6 presents the PDA data and results of CAPWAP analyses performed on the tested piles. The maximum transferred energy at sensors near the heads of Pile 1 and 2 was about 41 and 38 kJ, respectively. The mobilized static resistance for the two piles was 2300 and 2200 kN. The mobilized toe resistance was about 60% of the total resistance. The maximum compressive stress at the sensors (CSX) near the heads of Pile 1 and 2 was 181 and 169 MPa, respectively. The maximum estimated compressive stress at the toes (CSB) of Pile 1 and 2 was 93 and 107 MPa, respectively. These values were well below $0.7F_y$.

The same hammer and driving criteria were used during the driving of the remaining piles. Out of the ten driven piles, two piles were buckled at the head and toe due to obstructions with cobbles.

To reduce the risk of obstructions affecting the pile driving schedule, it was decided to drive the remaining piles using a down-hole-hammer. At the design penetration, piles were seated using a 44.5 kN drop hammer. Drilling the piles using a down-hole hammer proved to be a successful solution and faster than using churn drilling. However, it should be noted that using down-hole hammer tends to loosen the soil surrounding the pile as the hammer has a slightly larger diameter than the pile which may result in reduction of lateral pile capacity.

4. Observations from Presented Case Histories and Other Collected Data

In this paper, data was collected from case studies of steel pipe piles driven in dense soils. The data from the presented three case histories and other data collected from the literature and by personal communications are presented in Tables 7 and 8. Table 7 summarizes the cases that encountered pile damage during driving in dense soils. Table 8 summarizes the cases in which piles were driven successfully, without damage, in dense soils. The data represents piles for bridges, fixed offshore platforms, marine structures and buildings. A comparison between observations and available guidelines from literature was conducted. The observations indicate that some available guidelines should be refined to account for pile installation in very dense soils with cobbles/boulders as discussed in the following subsections.

4.1. Pile Diameter to Thickness Ratio and Hammer Energy

Tsinker (1997) reported that, for offshore piles, bending stress is not critical when d/t is less than or equal to 60 and if d/t is greater than 60, the piles should be checked for buckling stability. However, Tsinker's recommendation did not consider soil type or hammer energy. **Gerwick (2000)** reported that, in the case of Goodwin offshore platform on the Northwest Shelf of Australia, piles were driven into dense sands and they had to first penetrate a surface layer of cemented sands (calcarenite). The pile diameter and wall thickness were 2.6 m and 45 mm, respectively (i.e., d/t=58). It was believed that the relatively thin walled piles driven into the calcarenite is the major reason for the historic pile damage that occurred at this platform.

Figure 10 presents a relationship between normalized d/t ratio and hammer rated energy to cross sectional area (En/A_s) ratio for steel pipe piles. The figure shows some available data presented in Tables 7 and 8. The impact hammers were either diesel hammers or drop hammers.

It can be shown in Figure 10 that pile damage during driving in strata that contains cobbles/boulders may occur even with relatively low d/t and En/As ratios. Also, it is noted that the Canadian Foundation Engineering Manual (CFEM) recommended driving energy limit of 6000 kJ times pile cross sectional area (A_s) proved to be sufficient to avoid pile damage during driving. However, this limit should be reduced if piles are driven into very dense soils comprising cobbles and/or boulders. It should be noted that the CFEM recommended driving energy limit does not take into consideration d/t ratio while **API (2007)** guidelines do consider the d/t ratio.

Tables 7 and 8 and Figure 10 indicate that steel pipe piles driven into soils that contain cobbles/boulders are susceptible to head and/or toe damage even if d/t ratio is as low as 32. Based on the presented case histories and the compiled data, it appears that a d/t ratio of less than 32 and a rated energy of less than 3000 kJ times the pile cross sectional area should be considered if piles are to be driven into very dense soils comprising some cobbles or boulders.

4.2. Maximum Driving Stresses

Dismuke (1979) recommended a limiting driving stress of 1.4 to 1.7 times the specified yield strength (F_y). **Davisson (1979)** recommended a maximum dynamic stress level of $1.1F_y$. It is believed that the allowable driving stress recommended by **Dismuke (1979)** and **Davisson (1979)** is too high.

Thompson and Thompson (1979) reported that steel piles with yield strengths of 240 to 350 MPa (Grade 2 to 3) were driven to impact stresses of more than $0.8F_y$ without damage, as long as the seating of the helmet was satisfactory. They compiled data from nine sites as included in Table 8. Most of their reported cases were for steel piles driven in sandy silt till or dense to very dense sand. They reported that in a few cases, gravel or shale bedrock was encountered at pile toes. **Lee *et al.* (1995)** recommended limiting the driving stress of steel pipe pile to $1.0F_y$ and $0.5F_y$ at pile head and toe, respectively, based on field test results.

The American Association of State Highway and Transportation (**AASHTO, 2002**) indicated that the maximum allowable driving stresses for top driven steel piles should be limited to $0.9F_y$.

US Army Corps of Engineers (2004) recommended the maximum allowable driving stresses for steel piles be limited to $0.85F_y$.

Schneider *et al.* (2003) reported that published mean hammer efficiencies may be unconservatively low. Due to different hammer performance and soil conditions, significant variation in transferred energy may induce high compressive stresses near the end of driving. They concluded that eccentric pile stresses ranges between 15% to 25% may be induced during driving, especially during restrikes. This implies that allowable driving stresses close to the pile yield stress may cause pile damage during driving in dense soils.

Figure 11 presents a chart between d/t ratio and the impact driving stresses at pile heads for some case studies. Data from Tables 7 and 8 is used in Figure 11. The driving stresses shown in this figure represent the maximum stresses at the pile heads measured during PDA tests or calculated using the wave equation analysis. It can be shown in this figure that several piles installed in dense soils were damaged at pile head and/or toe even when the driving stresses were below $0.6F_y$.

It is noted that most recommendations reported in the literature for maximum driving stresses are somewhat general and do not provide limits for piles driven into very dense soils comprising cobbles and/or boulders.

Based on field observations and data analyses for several case studies, it is recommended that the driving stresses be generally limited to about $0.5F_y$ if piles are driven in very dense sands, especially if cobbles and/or boulders are present.

4.3. Critical Hoop Buckling Stresses

The critical hoop buckling stresses, F_{hc}, for piles were calculated using the API (2007) equations. Figure 12 indicates that F_{hc} for piles with d/t ratio of greater than 80 can be as low as about 25 MPa. This suggests that piles with high d/t ratio could potentially fail in hoop buckling, especially when driven in dense or till-like soils. Also, it is clear that F_{hc} decreases significantly with the increase in d/t ratio. However, in case of driving in cobbles/boulders, local buckling may still occur.

5. General Recommendations for Pile Driven Into Dense Soils

Different construction methods and precautions should be utilized to reduce the risk and extent of damage caused by steel pile driving into dense soils. Some of these methods were utilised successfully to overcome the problem of hard driving in the three presented case histories in Section 3. Other precautions and construction methods provided in the literature are also listed for guidance. These precautions and construction methods are listed as follows:

1- Usually, open ended piles are easier to drive into difficult soil conditions. If closed ended piles are required, conical driving tip should be utilized.
2- For large diameter piles such as those used for offshore platforms, the pile shoe should be reinforced to at least one diameter in length and have a wall thickness 1.5 times the minimum thickness of pile section in that pile. However, if piles are driven through limestone containing cobbles, the pile shoe should be increased to two diameters in length to prevent buckling **Tsinker (1997).** This recommendation was used in Case History 1 but was not sufficient to mitigate pile damage.
3- Reinforcing pile toe with APF type hardened cast shoes which retains the roundness of the pile toe cross section. This reinforcement type works well during driving into soft rock.
4- Reinforcing pile toe with a Doubler plate. This reinforcement type is generally stronger than the driving shoe but it does not maintain the roundness of the pile toe cross section.
5- Using Spin-Fin piles, which are large radial fins welded on the bottom 2 to 3 m of the pile. This pile type was driven successfully through rip rap and boulders such as in the Bell Street Pier Wave Barrier in Seattle (**Peratrovinch, 1998**).
6- Drill and drive using a churn drill or hammer grab for cleaning out the material inside the piles (**Gerwick, 2004**). This method was used in Case Histories 1 and 2. However, this method may be very slow and time consuming.
7- Drill and drive using down-hole-hammer which is faster than the churn drill. This method was used successfully in Case History 3 especially after the damage of some piles. However, it should be noted that this method may loosen the soil surrounding the pile which may result in reduction of lateral pile capacity.
8- Have a heavy duty jet and pumps able to develop pressure (**Gerwick, 2004**). However, jetting may reduce the axial and lateral pile capacities.
9- Using toe driving instead of head driving.

If a large number of blow counts is required to achieve a certain pile axial capacity, consideration may be given to the following solutions: (a) Constructing a composite step-tapered pile by advancing a smaller diameter pipe out of the base of the larger pile, so that the total capacity is distributed between the two piles. This solution was used successfully in Case History 1 to achieve the capacity in one of the piles. (b) Driving the pile open ended and placing a plate inside the pile at a certain depth. This plate will act as an internal diaphragm forming a plug, which results in axial pile capacity increase. This method was utilised in some offshore piles (**Tomlinson, 1993**).

The recommendations outlined in this section should be considered before the beginning of construction, so that construction delay and overrun cost are minimized.

Table 1. Summary information of Pile data for Case History 1

Pile No.	Hammer type	Testing condition	Pile type	d (mm)	t (mm)	d/t	A_s (m^2)	Total pile length (m)	Length below gauges	Embedded pile length (m)	Notes
1	Delmag D62-22 diesel hammer	Restrike	Spiral welded pipe	1219	12.7	96	0.0481	20.1	12.5	8.8	
2	Delmag D62-22 diesel hammer	Restrike	Spiral welded pipe	1219	12.7	96	0.0481	18.9	11.7	10.2	Damage observed within the lower 1.5 m of pile
3	44.5 kN drop hammer	Restrike	Spiral welded pipe	1219	12.7	96	0.0481	15.2	9.7	9.4	

Table 2. Summary information of PDA data and CAPWAP results for Case History 1

Pile No.	Equivalent penetration resistance (bl/25 mm)	Pile Driving Analyzer (PDA) Data						CAPWAP Results							
		EMX (kJ)	Hammer stroke (m)	ETR (%)	Impact force (kN)	CSX (MPa)	CSB (MPa)	Mobilised static resistance			Smith damping factor		Quake		Max. toe displ. (mm)
								Total (kN)	Shaft (kN)	Toe (kN)	Shaft (s/m)	Toe (s/m)	Shaft (mm)	Toe (mm)	
1	9.8	69.8	2.6	44%	7604	158	135	6200	1900	4300	0.35	0.3	2.5	7.5	7.8
2	19.5	71.0	2.3	51%	8356	172	139	7000	1500	5500	0.3	0.3	2.5	6.6	7.0
3	>20	83.2	4.9	37%	9477	195	151	5600	1100	4500	0.4	0.3	2.5	10.9	11.0

EMX: Maximum transferred energy at sensors
ETR: Energy transfer ratio
CSX: Maximum compressive stress at sensors
CSB: Maximum compressive stress at pile toe

Table 3. Summary information of Pile data for Case History 2

Pile No.	Hammer type	Testing condition	Pile type	d (mm)	t (mm)	d/t	A_s (m^2)	Total pile length (m)	Length below gauges	Embedded pile length (m)
1	Delmag D30-32 diesel hammer	Restrike	Pipe	508	15.9	32	0.0245	13.5	10.4	9.2

Table 4. Summary information of PDA data and CAPWAP results for Case History 2

Pile No.	Equivalent penetration resistance (bl/25 mm)	Pile Driving Analyzer (PDA) Data						CAPWAP Results							
		EMX (kJ)	Hammer stroke (m)	ETR (%)	Impact force (kN)	CSX (MPa)	CSB (MPa)	Mobilised static resistance			Smith damping factor		Quake		Max. toe displ. (mm)
								Total (kN)	Shaft (kN)	Toe (kN)	Shaft (s/m)	Toe (s/m)	Shaft (mm)	Toe (mm)	
1	>20	52.2	2.9	30%	5049	223	221	4100	300	3800	0.5	0.3	2.5	8.5	8.9

EMX: Maximum transferred energy at sensors
ETR: Energy transfer ratio
CSX: Maximum compressive stress at sensors
CSB: Maximum compressive stress at pile toe

Table 5. Summary information of Pile data for Case History 3

Pile No.	Hammer type	Testing condition	Pile type	d (mm)	t (mm)	d/t	A_s (m^2)	Total pile length (m)	Length below gauges	Embedded pile length (m)
1	44.5 kN drop hammer	Restrike	Pipe	610	12.7	48	0.0238	36.5	28.5	27.3
2	44.5 kN drop hammer	Restrike	Pipe	610	12.7	48	0.0238	24.3	22.0	21.0

Table 6. Summary information of PDA data and CAPWAP results for Case History 3

Pile No.	Equivalent penetration resistance (bl/25 mm)	Pile Driving Analyzer (PDA) Data						CAPWAP Results							
		EMX (kJ)	Hammer stroke (m)	ETR (%)	Impact force (kN)	CSX (MPa)	CSB (MPa)	Mobilised static resistance			Smith damping factor		Quake		Max. toe displ. (mm)
								Total (kN)	Shaft (kN)	Toe (kN)	Shaft (s/m)	Toe (s/m)	Shaft (mm)	Toe (mm)	
1	4	40.7	2.0	45%	4193	181	93	2300	930	1370	0.3	0.1	2.5	6.0	10.2
2	4	37.7	2.0	42%	3912	169	107	2200	900	1300	0.3	0.1	2.5	6.0	10.4

Table 7. Summary information of damaged steel pipe piles during construction

Pile No.	d (mm)	t (mm)	d/t	L (m)	Soil conditions at pile toe	Hammer Type	Estimated Impact driving stresses (MPa)		Failure Location	Source/Location
							Pile Head	Pile Toe		
1	1219	12.7	96	10	Very dense sand with occasional Cobbles/boulders	Diesel D60-62 & Diesel D30-32	175-220	135-150	Head & Toe	Case History 1 (this paper)
2	508	15.8	32	10	Very dense sand with occasional Cobbles/boulders	Diesel D30-32	220	220	Toe	Case History 2 (this paper)
3	610	12.7	48	20	Very dense sand with occasional Cobbles	44.5 kN drop hammer	170-180	110-170	Head & Toe	Case History 3 (this paper)
4	219	6.4	34	20	Compact to dense sand	22.25 kN drop hammer	175-240	50-125	Head	Building, BC, Canada
5	219	6.7	33	20	Compact to dense sand	22.25 kN drop hammer	175-240	50-125	Head	Building, BC, Canada
6	2600	45.0	58	N/A	Calcareous sand	N/A	N/A	N/A	Toe	Goodwyn Platform, Australia
7	1219	15.0	81	N/A	Till-like	N/A	N/A	N/A	Toe	A Dock, BC, Canada
8	1016	12.7	80	N/A	Cobbles/boulders	N/A	N/A	N/A	Toe	A Dock, BC, Canada
9	1829	19.1	96	N/A	Cobbles/boulders	N/A	N/A	N/A	Toe	A Bridge, BC, Canada
10	2134	19.1	112	N/A	Cobbles/boulders	N/A	N/A	N/A	Toe	A Bridge, BC, Canada
11	324	9.5	34	41	Dense sand	Vulcan 80C	138	N/A	Splice	Hussien and Rausche (1991)

Table 8. Summary information of steel pipe piles driven in very dense soils without obvious damage

Pile No.	d (mm)	t (mm)	d/t	L (m)	Soil conditions at pile toe	Impact driving stresses at pile head (MPa)	Source/Location
1	406	12.7	32.0	30	Gravel	N/A	A Bridge over a river, BC, Canada
2	610	12.7	48.0	30	Gravel	N/A	A Bridge over a river, BC, Canada
3	1300	50	26.0	70	Concretions	N/A	North Rankin Offshore Platform, Australia
4	1219	50.8	24.0	N/A	Occasional Cobbles/Boulders	N/A	A Dock, BC, Canada
5	610	12.7	48.0	N/A	Occasional Cobbles/Boulders	N/A	A Dock, BC, Canada
6	914	19.1	48.0	N/A	Occasional Cobbles/Boulders	N/A	A Building, BC, Canada
7	610	12.7	48.0	10	Occasional Cobbles/Boulders	N/A	A Highway Bridge, BC, Canada
8	610	19.1	32.0	10	Cobbles/Boulders	N/A	A Highway Bridge, BC, Canada
9	219	12.7	17.2	25	Dense sand	N/A	A Pedestrian Overpass, BC, Canada
10	324	8.4	38.6	19	Sandy silt till-N=50-100	251	Thompson, C. and Thompson, D.(1979)
11	324	9.5	34.1	19	Sandy silt till-N=50-100	192.0	Thompson, C. and Thompson, D. (1979)
12	324	6.9	47.0	15	Sandy silt till-N=50-100	154.0	Thompson, C. and Thompson, D. (1979)
13	324	9.4	34.5	24	Silt till/gravel-N=80 to >100	203.0	Thompson, C. and Thompson, D. (1979)
14	324	7.1	45.6	24	Silt till/gravel-N=80 to >100	175.0	Thompson, C. and Thompson, D. (1979)
15	324	6.4	50.6	25	Silt till/gravel-N=80 to >100	261.0	Thompson, C. and Thompson, D. (1979)
16	178	7.9	22.5	15	shale bedrock, weathered	188.0	Thompson, C. and Thompson, D. (1979)
17	610	12.7	48.0	35	shale bedrock, weathered	216.0	Thompson, C. and Thompson, D. (1979)
18	1067	51	21	47 - 60	Gypsum rock	225	Sites C, D, Offshore platforms (Webster et al., 2008)
19	1067	44	24	88	Very dense calcareous sand	N/A	Site B, Offshore platform (Webster et al., 2008)
20	1067	38	28	100	Dense Sand/Very stiff clay	224	Offshore pile (Rausche et.al., 2009)
21	1067	44	24	100	Dense Sand/Very stiff clay	224	Offshore pile (Rausche et.al., 2009)

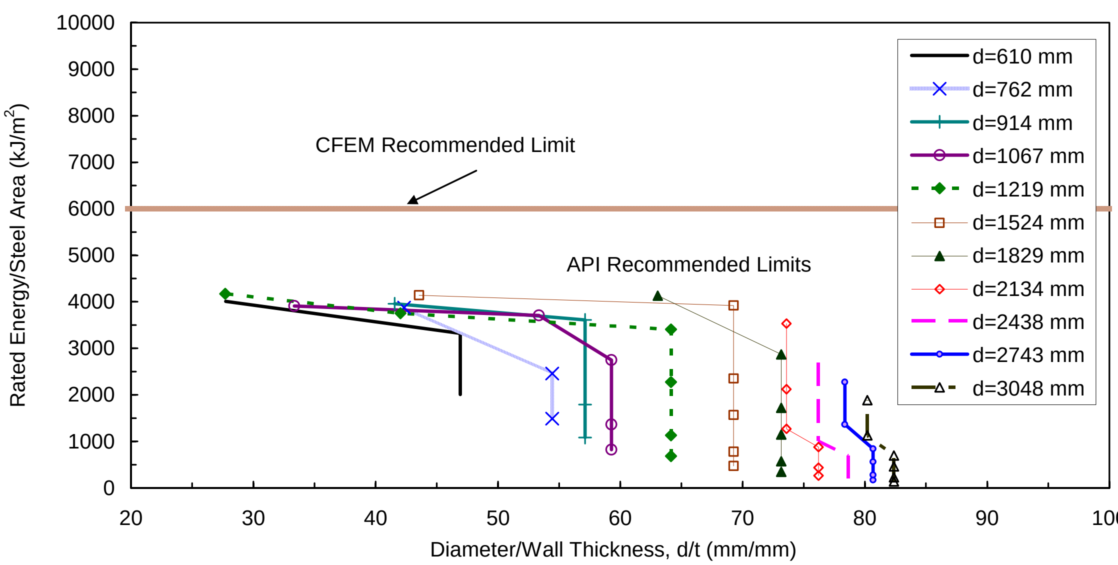

Fig.1. The ratio between pile diameter to wall thickness, d/t, versus rated hammer energy/steel area (based on recommendations of API, 2007 and Canadian Foundation Engineering Manual (CFEM), 2006)

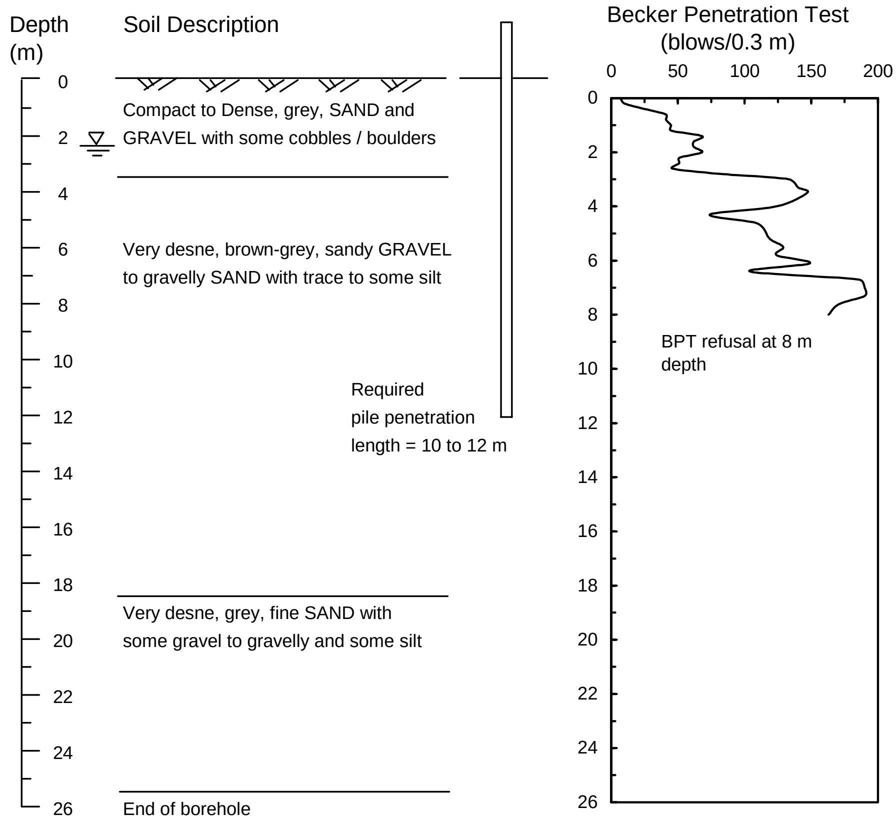

Fig.2. Soil conditions and Becker Penetration Test results for Case History 1

Fig. 3. Pile head failure (d=1219 mm, t=12.7 mm), driven by D60-62 diesel hammer (Case History 1)

Fig. 4. Pile toe failure (d=1219 mm, t=12.7 mm), driven by D30-32 diesel hammer (Case History 1)

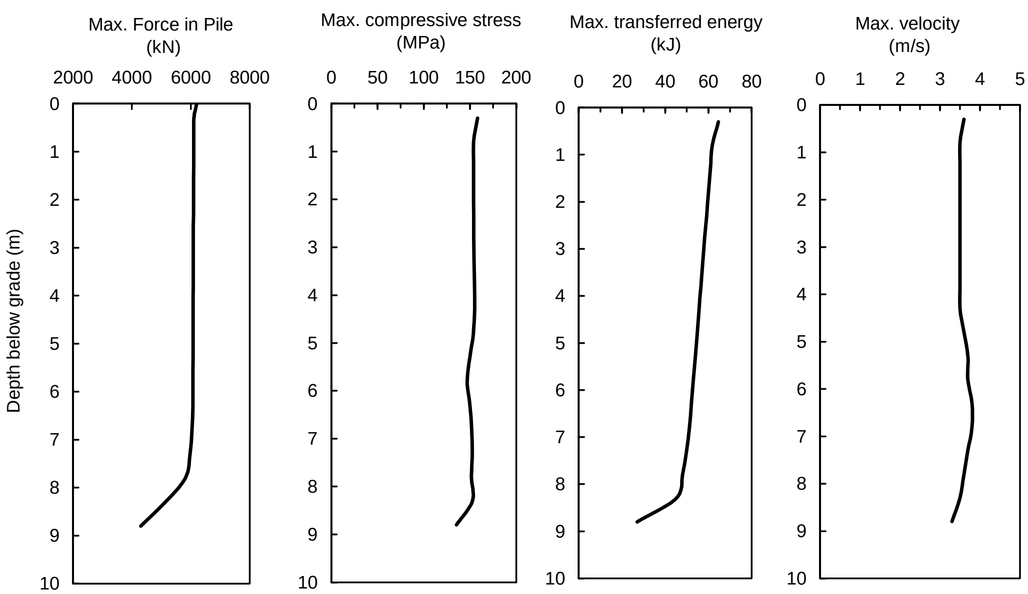

Fig. 5 Maximum force, compressive stress, transferred energy and velocity along the length of Pile No. 1 derived from CAPWAP results (Case History 1).

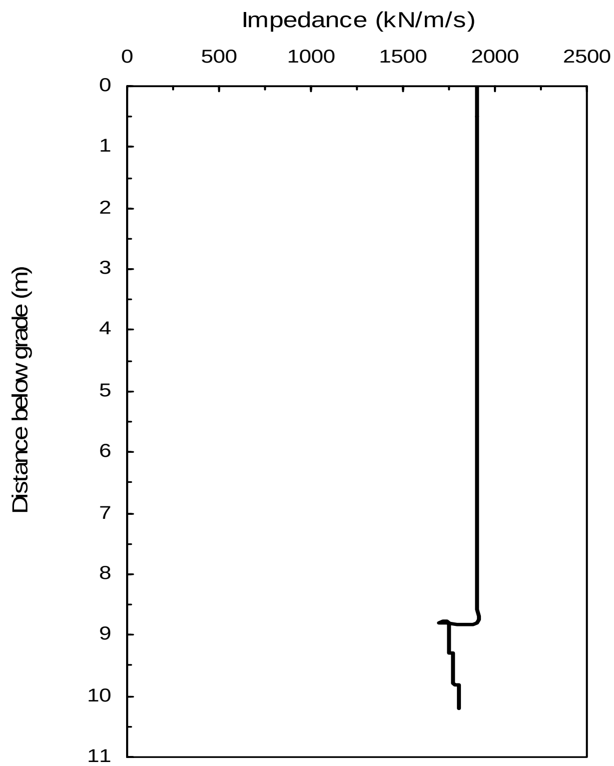

Fig. 6 Impedance profile for Pile 2 indicating damage within the lower 1.5 m (Case History 1)

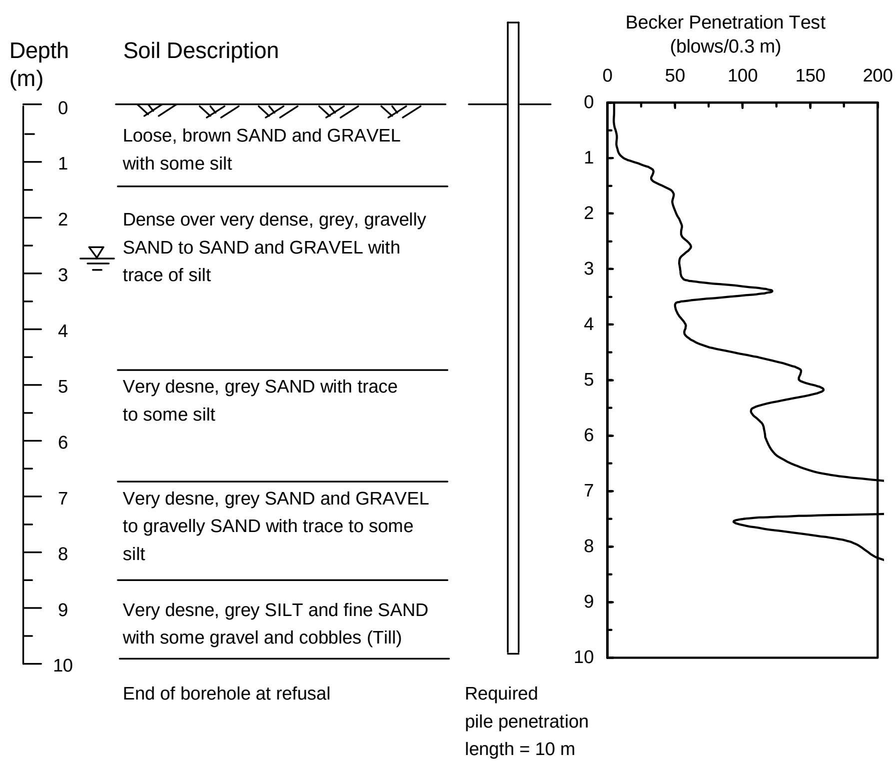

Fig.7 Soil conditions and Becker Penetration Test results for Case History 2

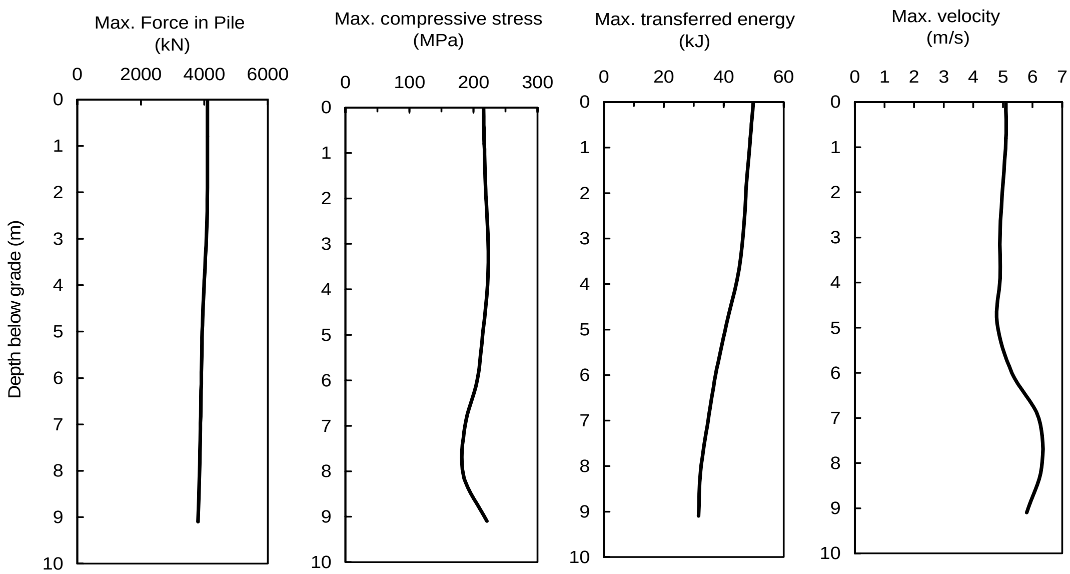

Fig. 8 Maximum Force, compressive stress, transferred energy and velocity along pile length derived from CAPWAP results (Case History 2)

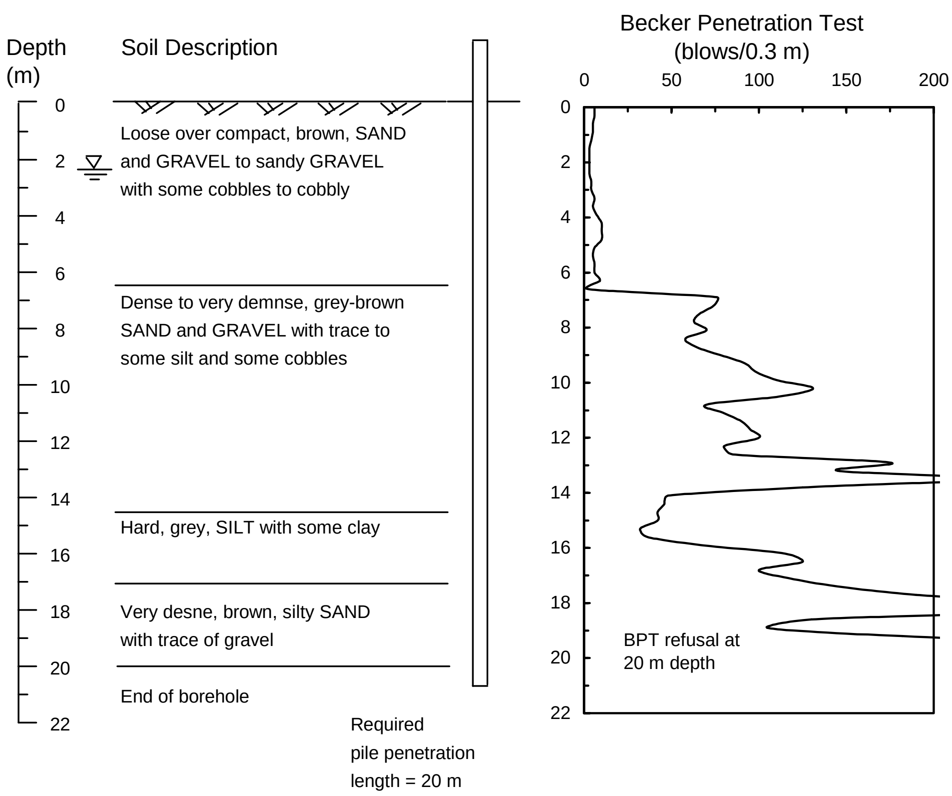

Fig.9 Soil conditions and Becker Penetration Test results for Case History 3

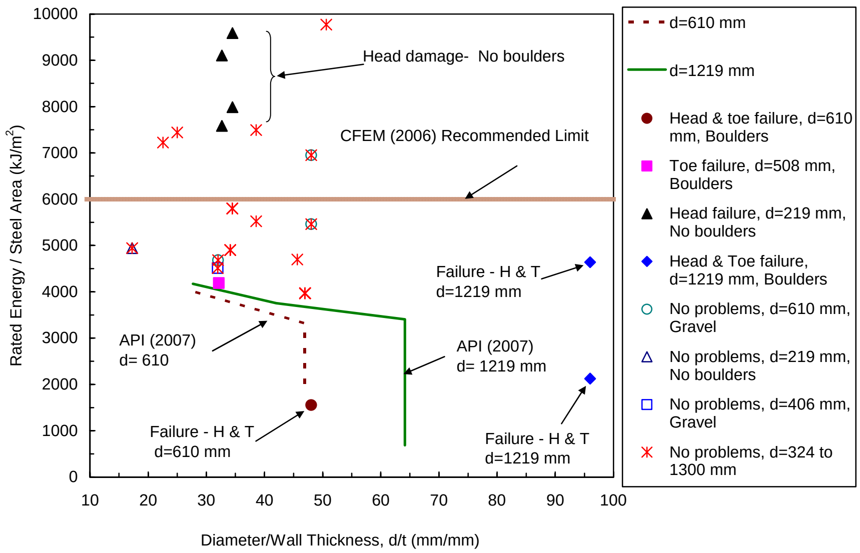

Fig. 10. Data presentation of case studies for d/t ratio versus hammer energy/A_s showing API and CFEM recommended limits

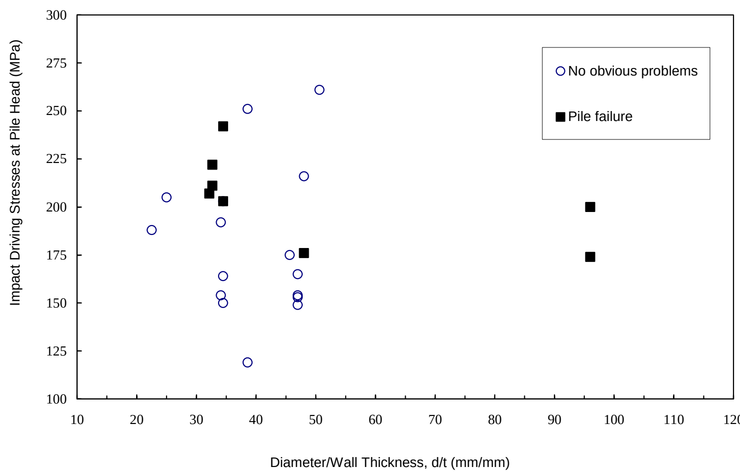

Fig. 11. Data presentation of case studies for d/t ratio versus impact driving stresses at pile head (F_y = 300 MPa)

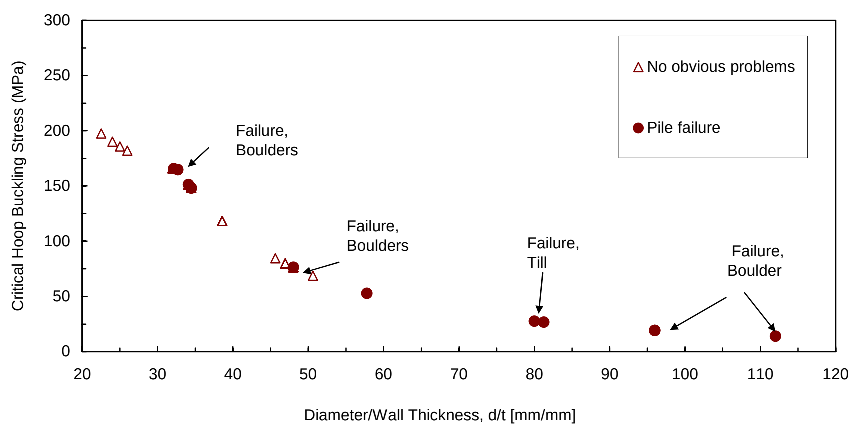

Fig. 12. Data presentation of case studies for d/t ratio versus critical hoop buckling stresses (F_y = 300 MPa)

6. Conclusions

This paper discusses the installation of steel pipe piles in very dense soils from a design and construction perspective. The main design factors impacting pile integrity during driving are the hammer energy, driving stresses and pile diameter to thickness ratio.

Three case histories for overwater bridge pile damage during driving are presented. Additional data from several onshore and offshore steel pipe piles installed in very dense soils are collected and analyzed to examine the design factors impacting the pile integrity during driving.

For piles driven in very dense soils comprising some cobbles or boulders, the following conclusions are drawn:

1- The pile diameter to thickness ratio, d/t, should not exceed 32 providing that the used rated hammer energy is less than 3000 kJ times the pile cross sectional area, especially for large pile diameters such as those used in offshore platforms.
2- The maximum driving stresses at the pile head and toe should not exceed $0.5F_y$.
3- A detailed and comprehensive geotechnical investigation is always desirable in spite of budget constraints during the design phase of some projects. If some cobbles or boulders are encountered, a large number of boreholes should be considered to determine the extent and size of cobbles/boulders and to improve the assessment of pile installation methods.
4- From construction prospective, general recommendations for installing steel piles into dense soils derived from the presented case histories and from literature are discussed in Section 5 and should be taken into consideration together with the above conclusions.

It should be noted that the objective of these conclusions is to reduce, and not to eliminate, the risk of pile damage during installation in very dense soils with cobbles and/or boulders. More field data for pile damage during driving is required to further adjust the limitations in hammer energies and driving stresses.

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