

BEHAVIOR OF CYLINDER PILES DURING PILE INSTALLATION

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ABSTRACT: Cylinder piles have a high axial compressive strength for high bearing capacities, they have a high moment of inertia and therefore can serve well as both a column and a foundation pile under high vertical and lateral loads. Cylinder piles often are used in nearshore applications where smaller foundation piles would require cofferdam construction and other costly measures. Drilled shafts have similar load bearing properties and capabilities, however, they are generally more costly than piles installed by impact driving.

While impact driven cylinder piles are a cost effective foundation solution they can pose problems during installation. One problem that has received repeated attention is the potential for pile damage, a second one is an uncertainty about the end bearing capacity. During pile installation the end bearing may not be present due to high soil inertia effects, during a static load application it is more likely, though it is not guaranteed, that the soil plug transfers its full end bearing to the inside of the pile.

Pile damage developing during the installation can occur at the pile top, along the pile both due to longitudinal and vertical cracking and near the pile bottom. Based on analyses and measurements, this paper examines the likelihood of pile damage developing during pile driving and discusses the end bearing problem.

INTRODUCTION

Renewed interest in large diameter cylinder piles for bridge foundations can be attributed to the need for larger horizontal and axial design loads. Alternatives to large diameter cylinder piles would be numerous smaller piles driven inside a cofferdam or drilled shafts, both more expensive solutions.

Currently research efforts are underway (e.g. McVay et al., 2004) to better predict the static behavior of the cylinder piles. The primary question in such studies is the behavior of the soil plug inside the piles. However, much research has already been done on the static and dynamic behavior of large diameter steel piles (e.g. Randolph, 2003). While their wall thickness is smaller than that of cylinder piles, the static

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analysis approach recommended in API, 1993, including its background material, should be of help to the designer of cylinder piles.

A problem of equal importance is the question of cylinder pile damage. Unfortunately, when pile damage occurs, it causes only temporary concern. Possibly, because of liability threats, this problem is not dealt with thoroughly during construction and interest in solving it quickly wanes after the foundation has been completed. Issues of cylinder pile damage are also more of a structural problem and therefore find little sympathy with the geotechnical engineer.

Dynamic testing of cylinder piles has been conducted during driving or restrike testing since 1974. A report submitted to the State of New York, Department of Transportation (Goble and Likins, 1974) summarizes dynamic measurements and correlations with static load tests for 36 inch diameter concrete cylinder piles. It is of interest to review these results in the light of the discussion on plugging and pile damage.

PROPERTIES OF CYLINDER PILES

In the United States 925, 1370 and 1680 mm (36, 54 and 66 inch) diameter cylinders with 127 and 152 mm (5 and 6 inch) wall thickness are the most common types and will be referred to in the inch-units in this paper. In other countries cylinder piles are typically only up to 750 mm (30 inch) diameter.

The large diameter cylinders in the US are made by either conventional casting in horizontal beds or horizontal spinning during hardening. The spun cylinder sections are typically 4.8 m long and post tensioned to form longer piles. The tendons are run through ducts in the cylinder walls. These unlined tendon ducts are formed during concrete hardening, with the forming inserts extracted shortly after the spinning process is completed. The conventionally formed cylinders are prestressed and manufactured in full length. In general, field splicing of cylinder piles is not practical. Both types of cylinder piles have a high concrete strength, often exceeding the typical specification of 49 MPa and the effective post tensioning stress may vary between 7 and 8.4 MPa. Based on these material properties, allowable maximum compressive and tensile driving stresses are normally at least 33 and 8.6 MPa, i.e., 85% of the concrete compressive strength minus the effective prestress and $\frac{1}{2}$ of the concrete's flexural strength plus effective prestress, respectively. However, for cylinder piles lower limits probably should be imposed (see discussion below). Dynamically measured stress wave speeds vary relatively little between 4400 and 5100 m/s corresponding to a dynamic elastic modulus between 47 and 52 GPa.

In other countries smaller diameter cylinder piles are made with concrete strength exceeding 70 MPa. They are often fitted with a steel band at top and bottom of each pile section. This protection against pile top or bottom damage also allows for splicing of piles by welding with cover plates.

INSTALLATION STRESSES AND REASONS FOR PILE DAMAGE

Compressive Stress Damage and Cushioning Considerations

During impact pile driving, stress waves generate both axial compressive and axial tension stresses. Axial compressive stresses may be highest at the top, below the top or at the bottom of the pile depending on the soil resistance distribution and the shape of the stress wave. Long friction piles have the highest compressive stresses near the top during driving. In piles with high end bearing the highest compressive stresses occur near the bottom. High compressive stresses at top and bottom are particularly dangerous to cylinder piles because (a) short of a tension band surrounding the ends of the pile, these areas are relatively unprotected and the effectiveness of the hoop reinforcement in the relatively thin walls of the cylinder piles is limited, (b) hammer eccentricity or misalignment can add substantial compressive stresses and (c) the stress state at the pile ends is rather complex and may include

- lateral forces form an expanding or laterally shifting pile cushion,
- non-uniformly worn cushions,
- poorly fitting helmets,
- lateral forces due to a poor driving system fit or improper hammer alignment,
- bending due to hammer eccentricity,
- anchoring forces of prestressing or posttensioning strands,
- grout pressure in tendon ducts.

Plywood pile cushions, well constructed to avoid horizontal shifting during pile driving and of greater thickness than normally designed for square prestressed concrete piles have been effective in limiting pile top damage. Thus, since the average compressive stress is not the only contribution to damage, it would be wise limiting it to at most 66% (rather than the standard 85%) of the concrete strength reduced by the effective prestress.

Lowering pile stresses also reduces pile cushion stresses which means longer cushion life and therefore a lesser need for cushion replacement. Of course, a thick pile cushion also dissipates more energy and it is therefore strongly recommended to perform preconstruction wave equation analyses. Their objective would be the determination of the correct hammer-pile cushion combination for low stresses at acceptable blow counts. Further recommendations may be found in Hannigan et al., 2006.

Cracking Due to Uniform Tensile Stresses

Like all concrete piles, cylinder piles are sensitive to tension stresses. Also, since cylinder piles often have a large weight resulting in a low ram-pile weight ratio, tension stresses are very likely to occur in easy driving situations. Tension cracks due to wave propagation effects tend to be horizontal. While the prestressing or post-tensioning forces in cylinder piles tend to keep the cracks closed, repeated opening and then closing in the presence of high compression forces can fatigue the concrete and produce progressive damage. This effect is aggravated in a wet environment by the so-called water jet effect, where water is pulled into the crack under tension

stresses and, when compression follows tension, is expelled at high speeds taking concrete particles with it.

Wave equation analysis conveniently calculates tensile driving stresses, however, the accuracy of the prediction heavily depends on the realism of the assumptions of resistance distribution, cushion condition and hammer performance among other uncertainties. It is therefore strongly recommended to monitor by dynamic measurements the installation of indicator piles to assess the magnitude of stresses and, if necessary, modify the driving system components. This not only helps to avoid damage to the piles but also allows for an optimization of the driving process by selecting the minimum necessary cushion thickness or the highest possible hammer setting.

Damage Due To Bending

Bending may be caused by hammer-pile misalignment, unevenly worn cushioning, forcing the pile direction by lead adjustment, crane-lead movements such as due to unstable crane barge, etc. The presence of obstructions is not predictable and being prepared for removing obstructions is important, if the subsurface investigations or the pile installation process indicate potential problems. Bending, particularly in the presence of high dynamic tension stresses, can crack the pile on its tension side and cause damage on the compression side in combination with stresses from driving and post-tensioning. Standard pile driving measurements can only detect bending at the strain sensor location and for that reason four strain sensors should be installed at 90 degrees around the pile. Unfortunately, the highest bending stresses may occur well below the pile top where they are undetected by standard measurements.

Longitudinal Cracks

Longitudinal cracking of cylinder piles has been vexing the pile driving profession for many years. In an attempt to explain why they occur, theories were developed and discarded, and measurements were taken and, after difficult job completions, forgotten. Because of the sensitivity of the cylinder piles to longitudinal cracking, and since hoop tension reinforcement is not prestressed, it is reasonable to limit the hoop tension stresses to $\frac{1}{2}$ of the concrete's flexural strength or typically 1.7 MPa. The following effects have been mentioned as potential causes for longitudinal cracks.

- (a) internal hydrostatic water pressure
- (b) internal dynamic air/water pressure
- (c) internal excess soil or pore water pressure
- (d) Poisson's effect
- (e) complex stress states at pile top or pile bottom
- (f) concrete and/or manufacturing defects
- (g) insufficient hoop reinforcement

Longitudinal cracks have been reported to occur both at the pile top and some distance below the pile top. Thus, because not all causes (a) through (g) apply to either pile top or the shaft below, different combinations of reasons may be

responsible for longitudinal cracking. An evaluation of the possible stress contributions of the various cracking causes follows.

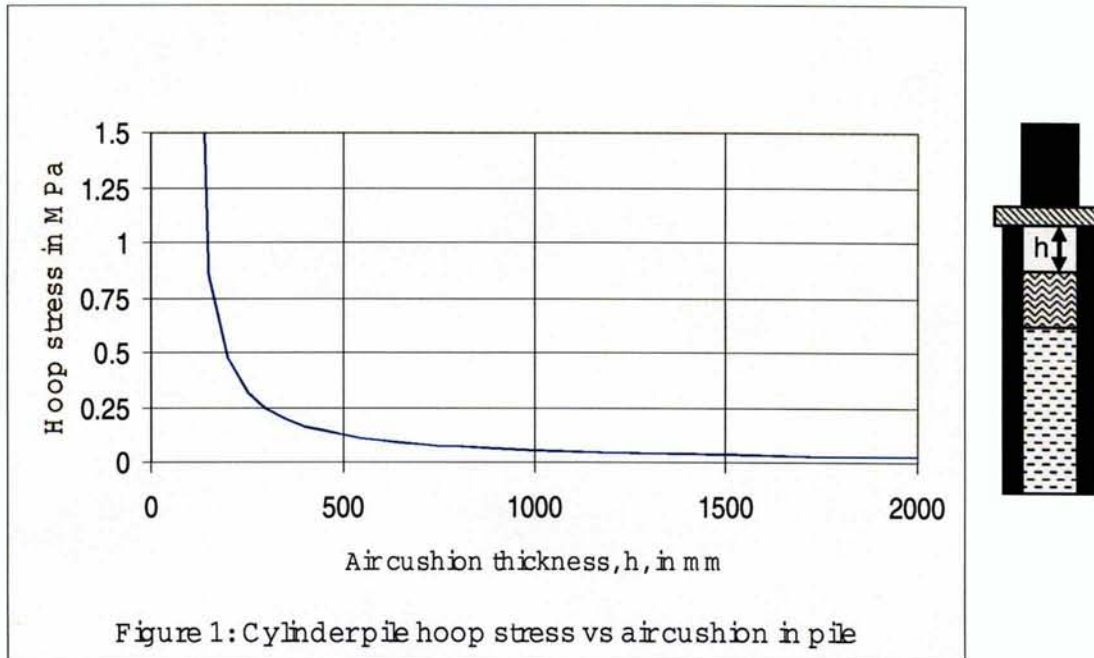
Hydrostatic water pressure inside the pile can only be a problem if the head is higher inside the cylinder than outside. Indeed, during pile driving the water often rises inside the pile relative to the outside. For that reason, cylinder piles should have and often do have a water release hole. The maximum pressure inside a pile due hydrostatic pressure can therefore be easily limited and calculated. For example, it should be possible to limit the excess water column inside the pile to 6 m. In that case, the hoop tension stress in a 66x6 inch cylinder pile would be less than 0.3 MPa. For smaller diameter piles the stresses would be even lower.

The internal *dynamic water pressure* is more interesting. It could be caused by the temporary reduction of the volume of the soil-water-air column inside the cylinder pile. For example, the maximum temporary penetration of a pile under a strong hammer blow in hard driving is around 25 mm while in easy driving, it should be kept at less than 50 mm or tension stresses would be a real problem. Additionally a new cushion of 300 mm thickness at a typical maximum dynamic stress of 20 MPa would temporarily compress about 30 mm for a total compression of air, water and soil of 80 mm. Adiabatic compression of the air inside the pile (assuming it cannot escape instantaneously through the pressure equalization hole or underneath the helmet) produces pressures that depend strongly on the initial amount of air inside the pile. Figure 1 shows for an air cushion of less than 200 mm thickness that the hoop stress in a 66x6 inch pile would be greater than 0.5 MPa. For an air cushion thickness greater than 0.5 m, the hoop stress would be immaterial.

While it is well known that an air cushion is effective in maintaining reasonably low hoop stresses due to dynamic effects, it would be interesting to check what would happen if no air existed between helmet and water. In that case the above calculation is not applicable (zero air volume). Instead a stress wave would be generated in both pile and water. Under the crude assumption of one-dimensional wave theory, the pressure in the water column would be

$$p_w = v_m c_w \rho_w \quad (1)$$

where v_m is the maximum velocity in the pile and therefore also in the water column, c_w is the wave speed in water (420 m/s) and ρ_w is the mass density of water. Impact velocities on concrete piles generally are less than 3 m/s, corresponding to a compressive concrete stress of 32 MPa. With this maximum velocity value, Eq. 1 leads to a water pressure of 1.3 kPa, which corresponds to a hoop stress of 5.9 MPa for the 66 inch pile. Obviously, such a high hoop stress could lead to vertical cracks. Again, this is only a critical situation when there is no or only a small air gap between helmet and water surface.



Internal *soil or pore water* pressures are much more difficult to assess. It is questionable whether or not these pressures can be higher inside the pile than outside and it would be even harder to assess whether or not longitudinal cracks actually occurred below ground and/or below the top surface of the internal soil plug. Probably, a plug formed during driving could produce much higher internal pressures than on the outside. Thus, when high blow counts develop and plugging is suspected, the number of hammer blows should be limited or else damage happens even though axial stresses, measured or calculated, are within reasonable limits.

Poisson's effect has occasionally been mentioned as a potential reason for longitudinal cracking. However, hoop strain due to Poisson's effect does not cause stresses and, therefore, should not cause cracking. The authors have seen horizontal strain measurement results that, as expected, ranged between 20% and 30% of vertical compressive strain. Exactly, how the hoop reinforcement interacts with the expanding concrete cylinder, particularly, since this cylinder is not uniform (grouted tendon channels), could be part of an interesting study.

The *stress state at the pile top* of a cylinder pile near its top or bottom is complex as has been discussed earlier and may not only cause the typical compressive failure but also longitudinal cracks very close to the top and/or bottom surfaces.

Manufacturing defects may include non-uniform concrete quality, improper curing, high shrinkage (resisted by the hoop reinforcement and therefore causing longitudinal cracks) excessive tendon grouting pressure, excessive non-uniform stresses due to improper post-tensioning sequences, etc. These problems may not result in immediately visible longitudinal cracks, but added to relatively moderate internal

pressures or non-uniform driving stresses, they may be partially responsible for crack development during driving.

Ideally the hoop reinforcement of large cylinder piles would be prestressed, because regular *hoop reinforcement* generally cannot prevent hairline longitudinal cracks. However, it can prevent longitudinal cracks from becoming major damage.

Installation Example

In 1985, at the Choptank River Bridge in Maryland, 54x6 inch cylinder piles had to be installed. A Conmaco 5300 air hammer, having a ram weight of 134 kN drove the piles with a reduced stroke of 0.9 m. During an initial test program, comparing the performance of 66 and 54 inch cylinder piles, pile damage developed. It was attributed to high internal pressures caused by either soil or water pressures. A very careful driving procedure was then designed, requiring that the pile was washed out to a depth not greater than 5 m of pile toe. Then the water had to be bailed out. Driving would then resume for an additional penetration of at most 3 m before repeating the cleaning out procedure. The Choptank production piles were successfully installed to their full required depth and passed the specified static and dynamic test requirements. Dynamic measurements indicated that pile top compressive and tensile stresses were less than 21 MPa and 8.4 MPa, respectively.

BEARING CAPACITY AND THE PLUG PROBLEM

General Remarks

Large diameter steel pipe piles are often driven offshore or nearshore. Satisfactory design procedure have been adopted which frequently rely on the API design specifications, dynamic analysis and monitoring during and after pile installation. Cylinder piles have different properties requiring some modifications to the design and installation specifications, however, they basically pose the same design dilemma as far as the end bearing and internal pile friction is concerned.

Plugging in the Dynamic Situation

The dilemma, the difference between the static and dynamic behavior of the soil plug, only occurs in coarse grained soils with potentially high end bearing in the static situation. In the dynamic case, the large mass of the soil plug may cause at best partial plugging, i.e. only during part of the hammer blow a fraction of the soil plug moves with the pile. Frequently, soil plug measurements are taken after the pile has been installed. If the soil surface inside the pile is deeper than grade outside of the pile, one considers the pile plugged. Of course, there could be other reasons such as volume change (densification) that are responsible for the lower plug top elevation. Also, it is usually not known, when and to what degree the relative plug movement occurred. For example, the plug could have moved with the pile during the last half of the pile installation, or it could have been partially plugging throughout the pile driving process.

The inertia force of the soil plug increases with the square of the pile diameter and proportionally with the pile acceleration at and above the pile toe when the plug

moves with the pile. On the other hand, the internal friction increases proportionally with the inside diameter, D_i . Plug slippage will occur when the internal friction force, F_f , is less than inertia force, F_i , plus end bearing against the plug, R_b , i.e., when

$$F_f > F_i + R_b \quad (2a)$$

and with $F_i = a_p L_p \pi \frac{1}{4} (D_i)^2 \rho_s$; $F_f = f_s L_p \pi D_i$ and $R_b = q_u \pi \frac{1}{4} (D_i)^2$ the inequality becomes

$$f_s > \frac{1}{4} (D_i/L_p) (a_p L_p \rho_s + q_u) \quad (2b)$$

In these expressions, a_p is the plug acceleration, L_p is the length of the soil plug, ρ_s is the plug mass density, f_s is the unit skin friction inside the pile, and q_u is the unit end bearing.

In the static case, ($a_p = 0$), the inequality requires that the plug length L_p is greater than $\frac{1}{4} (D_i)(q_u/f_s)$. For cases of interest with high unit end bearing, q_u is typically 100 times f_s and the plug length then has to be greater than 25 D_i for full end bearing development. If a significantly higher internal friction develops than on the outside, e.g. due to the reverse silo effect (Randolph, 2003), then the plug length can be shorter.

The dynamic case is more complex, since it cannot be expected that the whole plug length reacts with full internal shaft resistance. For example, if the pile is relatively long, the upper part of the pile may already rebound (taking the soil plug upward) before the pile toe even starts to move downwards. Thus the upper portion of the soil plug does not help balance the end bearing forces. In addition, the shaft resistance usually degrades during driving and it can be expected that the inside friction decreases to an even greater degree than the outside friction, because of the more limited effective stresses against the pile wall. For this reason, driveability analyses by the wave equation for large diameter steel piles are usually conducted without consideration of internal friction in the unplugged case. Again an exception would be in very dense granular soils a higher internal friction due the reverse silo effect and thus plugging. An analysis of the plugged case would assume full end bearing and no internal friction. In summary, only a short plug length above the pile toe may provide friction against soil plug movement in the dynamic case. The maximum participating plug length may be a function of the diameter (for example 5 inside diameters), but also not more than the length of the stress wave which can be determined from measurements or wave equation analysis.

The plug inertia complicates the problem, but not necessarily as much as generally thought, because the peak pile toe acceleration only lasts a very short time period. Figure 2 shows a graph of plug inertia (pile toe acceleration times plug mass based on a 5 D_i plug length and full soil mass, i.e. not reduced by buoyancy) together with the pile toe resistance (static plus dynamic) as a function of time. These curves were calculated by wave equation analysis for a plugged 66 inch pile in sand, representing

an actual hammer-driving system-pile situation. The analysis assumed a 7100 kN capacity and a modest unit end bearing of 5.3 MPa yielding a calculated blow count of 94 bl/0.30 m. A hydraulic hammer with 270 kN ram weight was cushioned with 380 mm of used plywood and analyzed at a moderate stroke of 0.9 m. Figure 2 shows that the plug inertia sometimes adds and sometimes reduces the total upward force acting on the plug. More importantly, the maximum inertia (corresponding to an acceleration of roughly 50 g's) happens at a time when the soil resistance is not yet fully activated. After the initial positive inertia pulse, the sum of toe resistance plus inertia (marked "total") is, on the average, lower than the total toe resistance acting on the soil plug, obviously, because the pile rebounds with, on the average, negative velocities. This example demonstrates why it would be wrong to merely look at a dynamic plug force balance involving the peak acceleration when considering potential plug slippage.

To get a feel for peak acceleration levels in a plug, Figure 3 shows a chart which was based on numerous wave equation analyses for a 30 m long 66 inch concrete cylinder pile driven by either a diesel hammer with 100 kN ram weight or a hydraulic hammer with 300 kN ram weight. For consistency the peak toe acceleration levels were chosen at a high driving resistance (100 blows/0.3 m). The calculated pile toe accelerations, based on the assumption of unplugged driving, vary between 35 and 90 g's. For the diesel hammer they are roughly 40 to 60% higher than for the hydraulic hammer. The analyses also showed that the bottom accelerations were 80 to 120% of the top values. For a low driving resistance, e.g. of 30 bl/0.30 m, these ratios could reach almost 140%. Thus, knowing the peak pile top accelerations can lead to reasonably close estimates of the bottom peak acceleration values.

How can it be assured that a dynamic load test does indicate the correct end bearing? In cohesive soils which often lose friction due to the dynamic effects, the internal friction degrades to low values, however, in cohesive soils the end bearing is generally not very high and therefore not of much interest, however, it would require setup time and a restrike test to mobilize the end bearing. In loose sands internal friction should be low both during the static and the dynamic case, because of the lack of effective stress build up. End bearing will only reliably act on the concrete annulus in both static and dynamic situations. In very dense sands static end bearing forces may cause the reverse silo effect and therefore full end bearing which will lead to very high end bearing and low toe acceleration. In all cases it is therefore desirable to test the piles after a setup period with a high energy but low acceleration (soft cushioning, low impact velocity high ram weight hammer) and such a procedure should yield the desired capacity mobilization and therefore a meaningful dynamic test.

Example

The first cylinder piles dynamically tested by the Case Method and analyzed by CAPWAP were installed for a preconstruction test program in Lake Chautauqua in 1974 and reported by Goble et al., 1976. Four 36x5 inch piles were driven by a diesel hammer with 60 kN ram weight, rated at 143 kJ, through 30 m of silt and then either

fine sand ($SPT-N \geq 50$) or stiff silts and clays. Water depth was between 2.5 and 5 m. The New York DOT called for static tests and for dynamic tests during restrrike, the latter as part of an ongoing research program.

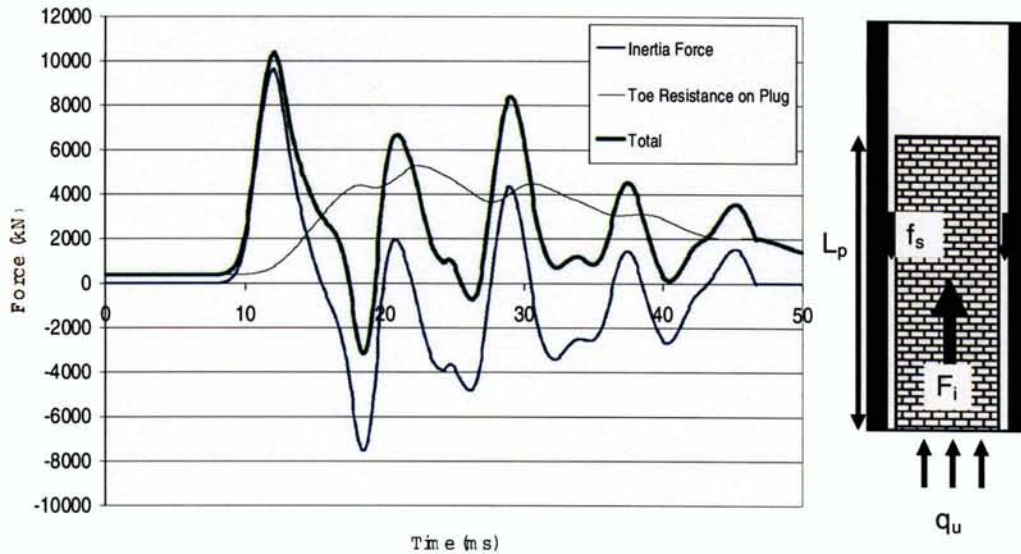


Figure 2: Soil inertia and toe resistance on plug and their sum

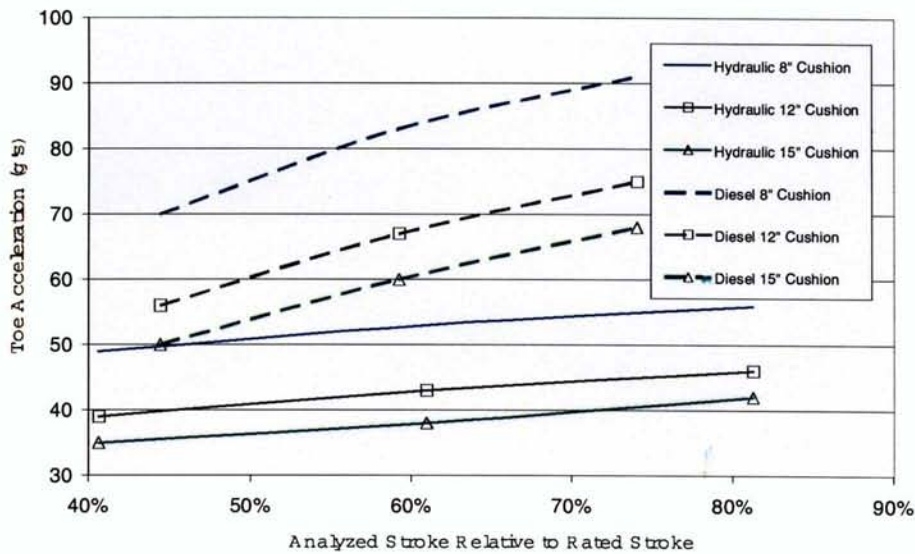


Figure 3: Relative Toe Acceleration for Unplugged Cylinder Piles

The first test pile drove with low blow counts until it penetrated a stiff silt with sand where the driving resistance increased to roughly 60 bl/0.30 m. The blow count again

increased sharply when the pile toe entered the fine sand layer. After having penetrated 2 diameters into the sand, the blow count was 160 bl/0.30 m. After a waiting period of about 2 months, the static test did not fail at 4450 kN; the restrike test performed after another 2 months did not generate any noticeable penetration into the ground. A 1976 CAPWAP calculated 4100 kN capacity; the analysis was repeated with CAPWAP 2006 (PDI, 2006) yielding 4590 kN. The calculated end bearing was 2150 kN, corresponding to 6.8 MPa on the cylinder annulus and 3.3 MPa on the gross toe area. Probably, since there was no permanent penetration under the restrike hammer blows, not all capacity was mobilized during the restrike test.

A GRLWEAP analysis that matched the average measured restrike values of transferred energy of 47 kJ and the top stress of 16 MPa and with static and dynamic soil resistance input as per standard GRLWEAP recommendations (end bearing from GRLWEAP's SA analysis, see PDI 2005), indicated for the end-of-drive blow count of 160 bl/0.30 m an SRD (static resistance to driving) of 6200 kN. Refusal was indicated at a capacity of roughly 7200 kN (see Table 1).

A second pile, driven at Pier 8 of the same project, was founded in a stiff silt. In a first driving and static testing sequence at 40 m depth, the blow count was roughly 20 bl/0.30 m. A restrike after 2 days applied 12 blows for 100 mm penetration. This testing caused pile top damage at low stresses of approximately 17 MPa. A static load test was performed roughly one month later, failing at 2300 kN. The wave equation based on the restrike blow count indicated 3000 kN and CAPWAP calculated 2100 kN (Table 1). In this case static formulas would have yielded very non-conservative capacity predictions even for the unplugged case and that may be the reason why the contractor decided to perform the static test even though the dynamic information suggested a low capacity.

The pile in Pier 8 and another test pile either suffered top damage or broke near the mudline during redrive operations. Such damage must be attributed to other factors than the normal compressive driving stresses and may include bending induced by the driving system, eccentric impacts or, for the break at the mudline, high tension stresses, not monitored at the time, due to very low blow counts in the beginning of driving. Thus, when the pile failed below the required 4450 kN capacity, the contractor opted to jack the pile to capacity rather than re-driving it.

Even though the Chautauqua piles had a relatively low diameter of only 36 inches, the analyses showed that they did not plug during driving, neither in the very dense fine sand nor in the stiff silt. However, whether or not the pile in the sand did completely plug during the static test and would have done so at higher loads is not known because the load test was not run to failure.

RECOMMENDED TESTING AND ANALYSIS PROCEDURE

Installing large diameter cylinder piles can be challenging. Following the recommendations of the FHWA (Hannigan, 2006) for any type of driven pile, a preconstruction wave equation analysis is a must. Cylinder piles should first be

analyzed with an estimate of the SRD and assuming no plugging. Additionally, for very dense soils or weathered rock, a plugged analysis should also be performed with SRD shaft resistance and a plugged toe, modeling a plug length of 5 inside diameters by adding the corresponding soil mass to the pile mass. This analysis will yield conservative (i.e. upper limit) stresses and blow counts. Since the plugged analysis effectively closes the pile end, the toe quake should in this case be the outside diameter divided by 120. For unplugged analyses the standard 2.5 mm (0.1 inch) toe quake is recommended. Finally, a restrike analysis is recommended with the estimated long term shaft resistance.

A preconstruction test program is recommended, not only for pile length and bearing capacity assessment, but also for developing a suitable driving system (cushioning, hammer energy) and a driving procedure that is safe (stresses below the recommended limits) and economical (acceptable blow counts, less than 100 blows/0.30 m). Hammer-pile alignment should be checked during this test program (monitor strains at 90 degree locations during driving). Restrike testing should be done with a hammer of high ram weight and low impact velocity, sized for a minimum of 2.5 mm set per blow.

Table 1: Chautauqua Project – Calculated Capacities and Test Results

Test/Method ¹	Pier 5		Pier 8	
	End Bearing	Total Capacity	End Bearing	Total Capacity
	kN	kN	kN	kN
GW - EOD Blow Count	4100	6200	560	1800
GW - BOR Blow Count	4750	7200	900	2900
CAPWAP - BOR	2150	4590	200	2100
Static Load Test	N/A	>4450	N/A	2300

¹ GW=GRLWEAP; EOD = End Of Driving; BOR = Begin restrike

SUMMARY AND CONCLUSIONS

The paper discussed two topics (a) the potential for damage on cylinder piles and (b) the question of plugging during pile driving or dynamic load testing. It has been concluded that:

- (1) the stress state in a cylinder pile at top and bottom is complex. It is therefore recommended to allow for the dynamic top and bottom stresses during driving at most 66% of concrete strength minus effective prestress.
- (2) Stresses due to bending or hammer misalignment can and should be avoided.
- (3) Internal pressures are generally low during impact driving unless there is no water pressure relief, or a lack of sufficiently large air cushion or a plug forms in very dense soils.

- (4) When a plug forms, washing out the plug and bailing out the water may be necessary if greater pile penetrations are required after plugging occurred.
- (5) Soil plugs often do not develop during driving because of both the plug inertia and lack of internal friction.
- (6) Prior to pile driving a wave equation analysis must be performed as a check on proper equipment selection for satisfactory stresses and blow counts both with optimistic and pessimistic assumptions of soil resistance.
- (7) During a preconstruction test and occasionally during production pile driving, dynamic monitoring should be performed to check stresses, hammer performance and cushion effectiveness.
- (8) Proper dynamic load testing requires sufficient waiting times for full internal and external soil resistance. A high ram mass hammer with low impact velocity has a better chance to mobilize high end bearing
- (9) Plug length measurements are valuable to help improve the state of the art, even though they do not necessarily indicate when the pile plug formed.

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