

Pile Driving Analysis for Top Hammering and Bottom Hammering

Jonggeun Choe¹ and Hans C. Juvkam-Wold²

Abstract: Piles are used for platform foundations and other offshore structures. Pile driving performance is predicted and analyzed using the wave equation analysis method. In general, the hammering point can be any part of the pile and the same analyses used for hammering at the top of the pile (top hammering) can be used for pile driving by hammering at the bottom of the pile (bottom hammering). Based on the numerical analyses in this research including residual stresses in the pile, there is little difference between the predictions of pile penetration per hammer blow by single- or multiple-blow analyses when soil resistance is low, such as 10 blows/m. The same is true for top hammering and bottom hammering when soil resistance is low. However, when soil resistance is high compared to that of the pile-hammer system, single-blow analysis predicts early refusal for top hammering and unrealistically high pile penetration for bottom hammering. Therefore, multiple-blow analysis, which considers residual stresses, should be used for better understanding of realistic pile driving performance and predictions. Additionally, this study shows that gravity is another controlling factor for pile driving in low-resistance soil.

DOI: 10.1061/(ASCE)1090-0241(2002)128:2(174)

CE Database keywords: Pile driving; Pile hammers; Residual stress; Wave equations.

Introduction

Piles are used for platform foundations and other offshore structures. In the early stage of development of the pile driving practice, Newton's laws were applied to pile driving analysis on the assumption that the energy delivered by the hammer would be immediately transmitted to the tip of the pile at impact. However, the results from this method yield poor reliability because of elastic and plastic behaviors of the pile and soil. Smith (1962) developed a mathematical solution to the wave equation to solve complex pile driving problems. He developed discrete element formulations of the impact pile driving problem based on one-dimensional wave propagation theory. His method has been modified and refined since then.

For the analysis and prediction of pile driving performance, there are several physical uncertainties associated with key wave equation variables and pile driving operations due to the characteristics of the pile, hammer, and soil. Hammer characteristics include thermodynamic behaviors of the hammer and conditions of the cushion and other accessories. Soil characteristics include soil strength, setup effects, degree of consolidation and cohesiveness, quakes, and damping constants. Despite these uncertainties,

the wave equation analysis method has been used successfully with engineering judgment, and several wave equation analysis programs are publicly available.

Lowery et al. (1969) have done intensive studies on pile driving analysis and have developed several versions of wave equation analysis computer programs based on the Smith mathematical model. Their models can predict driving stresses in the pile and pile load-bearing capacities by generating a plot of driving resistance versus blows. However, their models are valid only for a single hammer blow and are limited to a maximum of 70 pile elements. Their models always include the gravity effect, whereas Edwards' program (1969) has an option to include the gravity effect. These programs are valid for a single hammer blow at the top of the pile (top hammering) and ignore the effects of residual stresses in the pile.

Holloway et al. (1978a; b) provided a good review of a number of wave equation analysis programs including capacities, limitations, and features of each model. Although their model incorporates residual stresses in the solution, it is valid only for top hammering. The existence and importance of residual stresses have been recognized for a long time, but they have not been routinely included in pile driving analysis. Briaud and Tucker (1984) gave a good review of the importance of residual stress. They analyzed available instrumented pile load tests including residual stresses. However, their analyses on the load-settlement data were affected by the natural variation of the soil and low data quality. Rieke and Crowser (1987) conducted instrumented pile tests and analyzed the results of tension and compression tests including residual stresses. They found that residual stresses depend on loading history and degree of soil disturbance, and one or two uplift loading cycles will not completely dissipate residual stresses. However, they did not provide any systematic procedure to be used in numerical analysis of residual stresses.

Commercial wave equation analysis programs such as that of Goble Rausche Likins and Associates, Inc. (GRL) (1996) are also available. The GRL program has excellent features for handling

¹Assistant Professor, School of Civil, Urban, and Geosystems Engineering, Seoul National Univ., Seoul 151-742, S. Korea. E-mail: johnchoe@snu.ac.kr

²Holt Chair Professor, Dept. of Petroleum Engineering, Texas A&M Univ., College Station, TX 77843-3116. E-mail: juvkam-wold@spindletop.tamu.edu

Note. Discussion open until July 1, 2002. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on February 1, 1999; approved on May 15, 2001. This paper is part of the *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 128, No. 2, February 1, 2002. ©ASCE, ISSN 1090-0241/2002/2-174-182/\$8.00+\$0.50 per page.

not only conventional drop hammers but also diesel hammers, which require an accurate simulation of the thermodynamics of the hammer. The GRL program uses an intensive algorithm of stroke convergence for diesel hammers and is capable of handling residual stress analysis for limited cases. The GRL software has a separate program for long piles.

Most of the models above are DOS-based programs mainly designed for single-blow analysis (SBA) at the top of the pile. The model of Holloway et al. (1978a; b) incorporates residual stresses and the GRL program may simulate bottom hammering by adjusting hammering location. For feasibility studies on long pile installation intended to isolate strong water flow zones in the Gulf of Mexico (Choe and Juvkam-Wold 1997), it was necessary to consider both top and bottom hammering. It was also necessary to consider residual stress effects in the pile for the deeper pile penetrations. Therefore, development of a new pile driving analysis program was needed.

This paper presents the development and applications of a wave equation analysis program and theoretical background based on the Smith method and its improvements. This paper also presents detailed numerical analysis results on the effect of a single blow and multiple blows, the effect of gravity on pile driving, and the effect of top and bottom hammering. This study is different from previous work because it provides multiple-blow analyses for bottom hammering with high soil resistance. The analysis results are based on the Smith solution (with its modification and improvements) without a specific site-based experiment.

Algorithm of Wave Equation Analysis

Smith (1962) developed a mathematical solution to the wave equation. The solution was based on a discrete element idealization of an actual hammer-pile-soil system. The hammer-pile system is represented as a series of weights and springs. All springs are assumed to be perfectly elastic, whereas the soil is modeled as a spring and a dashpot, which is often called a Kelvin-Voight model. The action of each weight and each spring is calculated in each time interval to determine pile displacement, force, and ground displacement per hammer blow for the specified soil resistance.

Smith's equations can be generalized as follows. The pile-hammer-soil system is represented in Fig. 1(a).

$$D(i,t) = D(i,t-1) + v(i,t-1)\Delta t \quad (1)$$

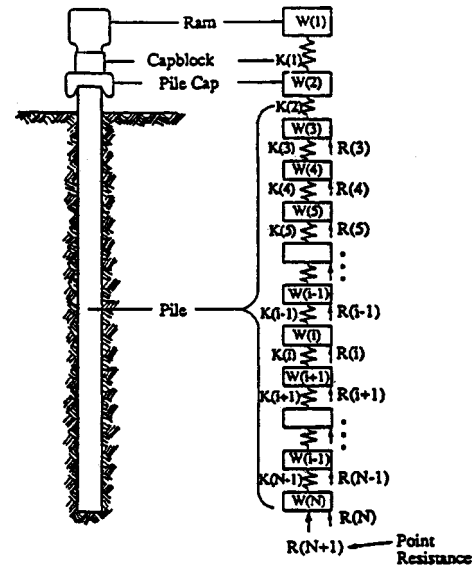
$$C(i,t) = D(i,t) - D(i+1,t) \quad (2)$$

$$F(i,t) = C(i,t)K(i) \quad (3)$$

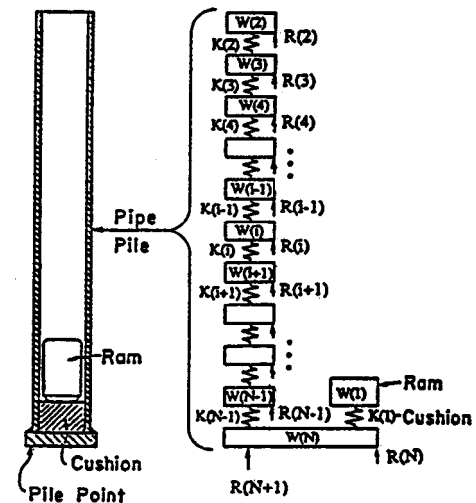
$$R(i,t) = [D(i,t) - D'(i,t)]K'(i)[1 + J(i)v(i,t-1)] \quad (4)$$

$$v(i,t) = v(i,t-1) + [F(i-1,t) - F(i,t) - R(i,t)] \frac{g\Delta t}{W(i)} \quad (5)$$

where $D(i,t)$ = displacement of the pile element i at time t ; v = velocity of the pile element; C = spring compression; F = force due to spring compression; K = spring constant of the pile; R = resistance to the pile; D' = ground plastic displacement; K' = ground spring constant; J = damping constant; g = gravitational acceleration constant; W = weight of the pile element; i = element index number; and t = time. Descriptions of the variables are also provided in the "Notation." Readers who are interested in the Smith solution in detail can read Smith's paper (1962) and others (Edwards 1969; Lowery et al. 1969).



(a) Actual As represented



(b) Actual As represented

Fig. 1. Method of representing pile for purpose of (a) top-hammering and (b) bottom-hammering analysis (modified from Smith 1962)

Quake

A linear elastic-plastic model proposed by Smith is used for the soil resistance to pile driving. It is assumed that the soil (or ground) compresses elastically for a certain distance and then fails plastically with constant resistance. This maximum elastic soil deformation is called the "quake." For any single hammer blow that results in a total displacement in excess of the quake, the pile bounces back a distance equal to the quake that results in net pile penetration. In other words, if a total pile displacement is less than the quake, theoretically there is no net pile penetration.

It is not easy to have an accurate estimation of the quake and there are some variations in quake (Table 1). Without sound theoretical and experimental background, the commonly accepted value of quake in the literature is 0.254 mm (0.1 in.). However,

Table 1. Value of Quake for Various Soil Types in Literature

Reference	Quake side, cm	Quake tip, cm	Soil type
Smith (1962)	0.254	0.254	
Kraft et al. (1980)	0.254	0.254	sand
	0.762	0.254	clay
Lowery et al. (1969)	0.254	0.254	sand and clay
Brown et al. (1972)	0.0254D ^a	0.0254D ^a	clay
Stevens et al. (1982)	0.254	0.254	silt
	0.508	0.508	soft clay
	0.381	0.381	firm clay
	0.254	0.254	stiff to hard clay

^aD is the pile diameter (in.).

the common value does not give satisfactory results for some soil types and conditions, especially when actual field experimental data are analyzed. In this study the common value is used for the numerical analyses because the wave equation method is not sensitive to the quake and qualitative trends found in this paper would not change with use of different quake values.

Damping Constants

When a pile penetrates the ground, the ground will offer more instantaneous resistance to rapid motion than to slow motion. The amount of damping is proportional to velocity, and the proportionality constant is the damping constant. Of course, the damping resistance is temporary and does not affect static pile bearing capacity [see Eq. (4)]. While the soil under the tip of the pile is displaced or caused to flow aside, the soil along the side of the pile is sheared. Therefore, the damping constants are not the same at the tip of the pile and along the pile.

Gravity Effect

Smith's original method did not account for the gravity effect, which has been added by several authors (Edwards 1969; Lowery et al. 1969). The model in this paper includes both gravity and the buoyancy effect. For a closed-ended pile that has different fluids inside and outside the pile, a general equation for the buoyancy factor (BF) is shown in Eq. (6a):

$$BF = \frac{(1 - \rho_o / \rho_s) - (d_i / d_o)^2 (1 - \rho_i / \rho_s)}{1 - (d_i / d_o)^2} \quad (6a)$$

where d = pile diameter; and ρ = fluid density. Descriptions of the variables are also provided in the "Notation." For an open-ended pile that has the same fluid inside and outside the pile, the BF is simplified to Eq. (6b):

$$BF = \left(1 - \frac{\rho_f}{\rho_s} \right) \quad (6b)$$

When considering the gravity and buoyancy effects, Eq. (5) should be modified as follows:

$$v(i, t) = v(i, t - 1) + [F(i - 1, t) - F(i, t) - R(i, t) + W(i)BF] \frac{g \Delta t}{W(i)} \quad (7)$$

The calculation of the wave equation analysis program starts with an initial ram velocity at the beginning of impact at time zero. All other variables have an initial value of zero if the gravity

effect is ignored. Otherwise, they have initial values of displacements and compressions at time zero because of the gravity. After completing a single-hammer-blow analysis, all information is available for a subsequent blow if necessary.

Hammering at the Bottom of the Pile

The same analyses used for top hammering can be modified for pile driving by bottom hammering. In general, the hammering point can be any part of the pile and the same analyses can be applied to analyze the hammer-pile-soil system. However, there are some different aspects of the phenomena. From the compression and tension tests of the pile, it is known that unit shaft resistance is proportional to the effective overburden stress in the soil surrounding the pile. Therefore, the shaft resistance for bottom hammering will be smaller than that for top hammering because of the reduction in pile diameter (Poisson's ratio effect) and unloading of the effective stress (Fellenius 1991). The ratio of the shaft resistance for top to that for bottom hammering varies depending on soil type, degree of consolidation, pile type, and so on. Since a mathematical model for the ratio has not been well established, the effect of different shaft resistance was not included in this study.

Fig. 1(b) shows a pipe pile being driven by a hammer operating inside the pile and striking on the cushion inside the bottom of the pile. Equations (1)–(7) apply to both Figs. 1(a and b). However, the following modifications are added because of different geometries and element numbering. As can be seen in Fig. 1(b), the ram element $W(1)$ is connected to the last element of the pile $W(N)$. Unlike in the top-hammering case, the second element is connected only to the third element. The cushion can be represented as one of the spring elements $K(1)$:

$$C(1, t) = D(1, t) - D(N, t) \quad (8)$$

$$v(2, t) = v(2, t - 1) + [-F(2, t) - R(2, t) + W(2)BF] \frac{g \Delta t}{W(2)} \quad (9)$$

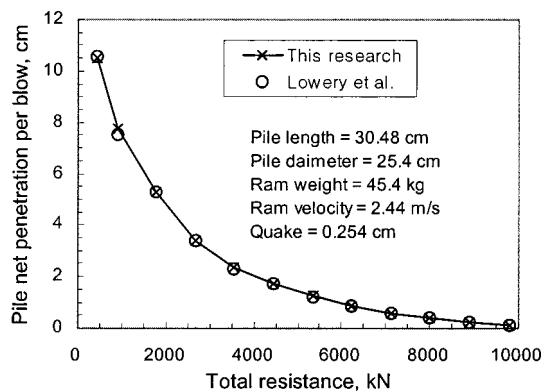
$$v(N, t) = v(N, t - 1) + [F(1, t) + F(N - 1, t) - F(N, t) - R(N, t) + W(N)BF] \frac{g \Delta t}{W(N)} \quad (10)$$

The same notations are used as in Eqs. (1)–(5), and descriptions of the variables are also provided in the "Notation."

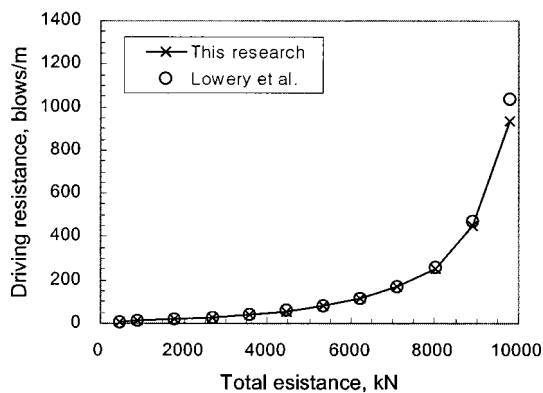
Multiple-Blow Analysis

During a hammer blow, the pile will move downward initially, then rebound, and then converge to a final position. At its final position, the residual stresses in the pile are not zero because movements of adjacent pile elements are not the same. For the GRL program (1996), residual stress analysis is possible but its usage is recommended for research purposes only.

Holloway et al. (1978a) showed the importance of a series of hammer blows rather than a single hammer blow. They also mentioned that a solution by multiple-blow analysis (MBA) was obtained typically after 3–5 simulated blows. However, this is shown in this study to be incorrect for high soil resistance, as discussed later in detail. No multiple-blow analysis is available for bottom hammering that is different from that of top hammering. In this study, residual stresses for multiple-blow analysis are calculated from relative displacements of adjacent pile elements from the preceding blow and become initial stresses in the pile for the next hammer blow analysis.



(a)



(b)

Fig. 2. Model comparison for (a) pile net penetration and (b) driving resistance with Lowery et al.'s model (1969)

Validation of the Program

The wave equation analysis program has been compared with available data and models. Since the program is based on Smith's method, it was first compared with his model results. The program gives the same net pile penetration of 0.52 mm by a single hammer blow using the data in Smith's paper (1962). Figs. 2(a and b) show a comparison of pile net penetration and driving resistance, respectively, versus total static soil resistance with the Lowery et al. model (1969). Gravity effect is included in the analysis to compare for a long range of total resistance. The same penetration results are presented in different terms. As can be seen in the figures, a good match exists between the two models for a wide range of soil resistance. If total resistance is less than 7,000 kN, the difference is less than 1.4%. For the largest resistance in Fig. 2(a), the difference in net penetration is 0.01 cm, which is hardly noticeable but it gives 9.8% difference. Furthermore, the difference in blow counts per meter looks large because the pipe penetration is relatively small [Fig. 2(b)]. In other words, net penetration itself is small so that a little difference yields a relatively high percentage difference. These differences result from the use of different soil models. Lowery et al. used a dynamic soil deformation model. Note the rapid increase of the blow counts as the total resistance approaches refusal. Fig. 3 also shows good agreement with the Edwards model (1969) for different damping constants.

For low soil resistance, a pile sinks by its own weight until equilibrium is reached between the pile weight and soil resistance. If soil resistance is slightly greater than pile weight, the

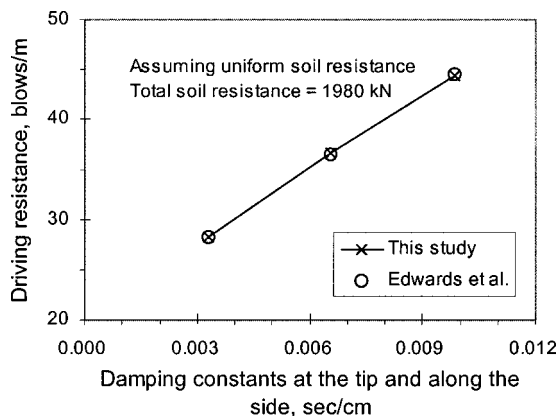


Fig. 3. Model comparison with Edward's model (1969)

results from the models discussed previously depend strongly on the maximum number of iterations specified as a program stop criterion in each program. Of course, results are very close if the same maximum number of iterations is used. If total soil resistance is close to the value for theoretical refusal (i.e., zero net penetration) of a given hammer-pile system by a single hammer blow, all the models above provide different results depending on the modifications made to handle very small values of net penetration. However, this would not cause any serious problem because it indicates almost zero net penetration. We suggest that blow count per unit depth penetration as an indicator of driving resistance is not reliable for very small penetration because it is very sensitive to small differences.

Comments on Wave Equation Analysis Programming and Usage

For the feasibility study on long pile installation, we developed a Windows-based program to study bottom hammering as well as top hammering, and multiple-blow analysis including residual stresses. In this section, some considerations and tips are provided for programming and usage of available wave equation analysis programs. The following are considered in our program.

Unit soil resistance does not always increase linearly or uniformly with depth. Therefore, simple triangular or uniform distribution of the soil resistance to pile driving (Edwards 1969; Lowery et al. 1969) is often not suitable for practical field applications. A program should have the capacity to handle a stepwise linear soil strength profile with depth that will cover almost all trends in soil strength. For the tip resistance, a program should allow an option to input or to calculate the value from the input soil strength data.

The critical time interval may be defined as the largest time interval that will produce a numerically stable solution. Smith (1955) provided equations to compute the critical time and recommended the use of about half of this critical time so as to prevent numerical instability resulting from other factors that were not included in his equations. A program should allow the use of either the Smith equations or the user's input for time interval. Consequently, it is also possible that users determine a proper time interval after checking the critical time interval from the Smith equations.

Wave equation analysis programs usually stop if all velocities are less than zero and there is no more pile penetration. Another common way to stop wave equation analysis programs is to

specify a maximum number of iterations. This is a good practice to avoid endless loop calculation resulting from numerical instability. However, it is not a useful stop criterion for long pile analysis. For a long pile, it takes more time for the impact wave to travel from the impact point of the pile to the other end of the pile. In other words, a long pile needs more time (or a greater number of iterations) to complete a single-hammer-blow analysis. Therefore, wave equation analysis programs could stop before completing all the necessary calculations, especially in the case of long piles, if the maximum number of iterations specified is small.

A stop criterion for maximum program run time should be based on impact wave travel time for the given total pile length. This approach is also independent of time step size and applicable for any pile length. Therefore, it will reduce the chance of errors in multiple runs for the same analysis by a working group. In order to complete single-hammer-blow analysis, the maximum program run time should be longer than twice the impact wave travel time for the pile length given. Four times the impact wave travel time was found to be adequate for almost all cases studied.

Here is an example. Impact wave speed in steel is about 5,060 m/s (16,600 ft/s). It takes about 0.0593 s to travel the 300-m distance from one end of the pile to the other end. Therefore, the maximum program run time for completing single-hammer-blow analysis should be greater than 0.119 s, which is equivalent to 480 and 1,190 iterations if the time step interval is 0.00025 and 0.0001 s, respectively. A stop criterion using a maximum number of iterations such as 500 may not be enough if the time step size used is 0.0001 s. Note that it takes 59 iterations for the Smith case of a 30.48-m pile (Smith 1962).

Further Analyses and Discussion

The first step may be to evaluate soil resistance to pile driving, which will limit how far into the soil a pile can be driven. The resistance to pile driving, at any one depth, is assumed to be directly proportional to the shear strength of the soil at that depth. For this reason it is critical to know the magnitude of the soil shear strength as a function of depth.

We used soil strength data proposed by Choe and Juvkam-Wold (1997). They evaluated soil strengths for deep waters from a number of projects in the Gulf of Mexico where piles have previously been driven, and from soil strengths determined by the Ocean Drilling Program. They used the remolded miniature vane shear strength (RMVSS) because it shows the best correlation with soil resistance to pile driving, which is commonly presented as number of blows per unit depth of pile penetration. Table 2 shows all input data with soil strength data.

The total resistance to pile driving at a given depth is the summation of the resistance at the external surfaces of the pile (outside friction resistance), the resistance at the toe of the pile (tip resistance), and the resistance at the internal surfaces of the pile (inside friction resistance). There will be an additional damping resistance when the pile is moving. For a long open-ended pile, inside friction resistance, which is dependent on the plugging effect, is assumed to be negligible compared to the outside resistance (Dutt et al. 1995). This is also true when cuttings inside the conductor pile are continuously removed while the conductor is being driven. The same is true for closed-ended piles for both top hammering and bottom hammering.

Outside friction resistance and tip resistance, when the pile is at a depth Z below the sea floor, are calculated from the following equations:

Table 2. Default Input Data with Soil Strength Data

31,000	total pile length, cm
60.96	outer diameter of the pile, cm
55.88	inner diameter of the pile, cm
610	hammer energy, kJ
36	weight of the ram, tons
0.85	hammer efficiency, fraction
1.03	density of sea water, g/cm ³
130	weight of pile cap, kN
76.2	diameter of cap block, cm
24,760	spring constant for cap block, kN/cm
0.5	coefficient of restitution for cap block
0.254	maximum elastic ground deformation (quake), cm
0.0049	damping constant at the point of pile, s/cm
0.0016	damping constant along the side of pile s/cm
Remolded miniature vane shear strength versus depth	
Depth (m)	kN/s m ²
0	0.00
213	71.82
305	88.38
	kip/sq. ft.
0	0.00
213	1.50
305	1.85

$$R_{\text{side}} = \int_0^Z f(z) \pi d_o dz \quad (11)$$

$$R_{\text{tip}} = 27A_{\text{tip}}f(Z) \quad (12)$$

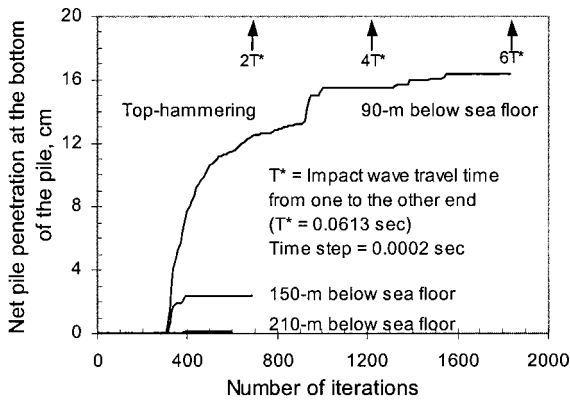
where R = soil resistance to pile driving; d_o = outer diameter of the pile (m); and A_{tip} = area of pile tip (m²). Alternatively, as mentioned in the previous section, any tip resistance can be used as input data if desired.

Top Hammering and Bottom Hammering

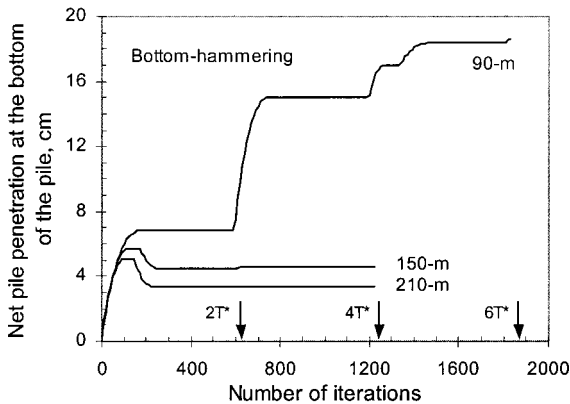
Top hammering has been the most common pile driving practice onshore and offshore. It has many advantages for easy operation with proven technologies. Bottom hammering may give better hammering efficiency and produce less operational noise because of the hammering location.

Fig. 4(a) shows net pile penetration versus number of iterations at three different depths below the sea floor. This is a single-blow analysis for top hammering. For top hammering, the pile is pushed downward by the hammer impact force, and the available force diminishes due to soil resistance as the impact force propagates to the pile bottom. Therefore, the pile penetration at the pile tip is considerably less than that of the pile top. Note that the pile penetration rapidly decreases as pile driving depth increases. For pile penetration at each depth in Fig. 4(a), pile penetration is zero before the impact energy is delivered to the pile bottom. After that net pile penetration increases and then converges to its final value.

As seen in Fig. 4(a), the calculated pile penetration at 90 m below the sea floor keeps increasing after one hammer blow as the number of iterations increases. Therefore, when soil resistance is very low (usually less than 7 blows/m), the predicted results are sensitive to the total number of iterations. This is why the maximum iteration number is important. If soil resistance is high (usually more than 700 blows/m), the predicted results among the models examined (Edwards 1969; Lowery et al. 1969; GRL 1996) start to show deviations as explained before.



(a)



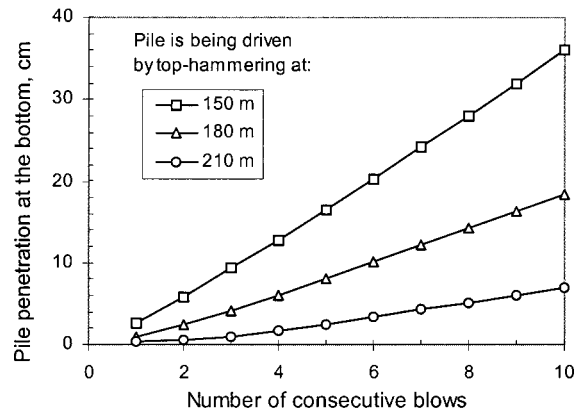
(b)

Fig. 4. Net pile penetration for (a) top hammering and (b) bottom hammering

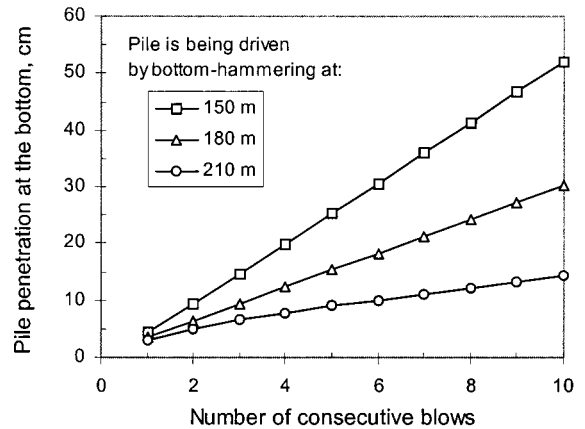
Fig. 4(b) shows the net pile penetration for bottom hammering predicted by single-blow analysis. If the pile is being driven 90 m below the sea floor, a depth that represents low soil resistance, it shows a similar trend to that of top hammering. In other words, hammering location does not affect pile penetration if soil resistance is low. Compared to Fig. 4(a), also note that there is pile penetration at the bottom from the very beginning of the iterations. If the pile is being driven 150 m below the sea floor, the penetration increases initially and then decreases, and then converges to its final penetration value. Also note that the predicted pile penetration at 210 m is unreasonably high (3.4 cm) compared to that of top hammering (0.048 cm) in Fig. 4(a), because the pile is pulled down from the bottom of the pile resulting in large pile penetration near the pile tip.

Single- and Multiple-Blow Analysis

Multiple-blow analysis is applied to see the effect of residual stresses. Figs. 5(a and b) show pile penetration at the bottom of the pile for 10 consecutive blows for top hammering and bottom hammering, respectively. Multiple-blow analysis for both top and bottom hammering shows almost a linear trend if the pile is being driven shallower than 150 m below the sea floor using the data in Table 2. In other words, MBA results are close to those of SBA because residual stress effects are not significant. As the pile is driven deeper, MBA results deviate from those of SBA. For the multiple-blow analysis, the pile penetration for top hammering gradually increases as the number of consecutive blows increases,



(a)



(b)

Fig. 5. Pile penetration for 10 consecutive blows of (a) top hammering and (b) bottom hammering

whereas pile penetration for bottom hammering levels off. This is particularly noticeable in the 210-m case for bottom hammering. However, MBA for both top and bottom hammering becomes almost linear after four or five consecutive blows. Therefore, an average net penetration of the last five blows out of 10 consecutive blows is used in this study. Use of two or three consecutive blows as suggested by Holloway et al. (1978a) will result in quite large predicted penetration for bottom hammering as soil resistance increases.

Fig. 6(a) shows a comparison of pile driving resistance between SBA and MBA for top hammering. The pile sinks about 58 m below the sea floor due to its own weight of 127 tons. Single-blow analysis reaches practical refusal beyond 200 m, whereas MBA reaches more than 250 m below the sea floor. Multiple-blow analysis gives better penetration per blow than SBA as penetration depth increases. Fig. 6(b) shows a comparison of pile driving resistance between SBA and MBA for bottom hammering. Compared to Fig. 6(a), the pile driving resistance for SBA for bottom hammering does not decrease much at great depths where top hammering indicates refusal. This unrealistic result is due to the fact that the tip part of the pile can move even a fraction of a centimeter, regardless of soil strength and the movement of the top portion of the pile. This is one of the disadvantages of single-blow analysis for bottom hammering at high soil resistance. As expected, MBA gives a result close to that of SBA for low soil resistance where the effect of residual stresses is negligible. As the soil resistance to pile driving increases, MBA for bottom ham-

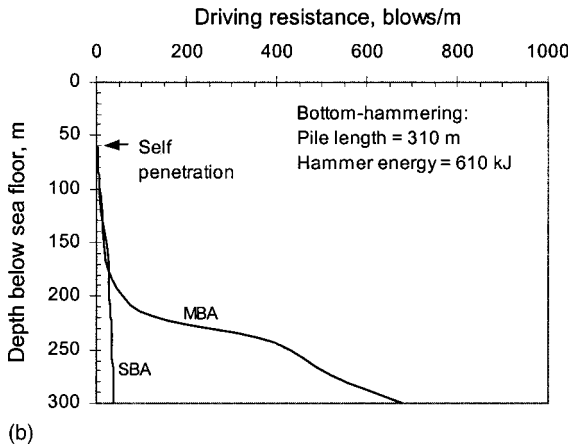
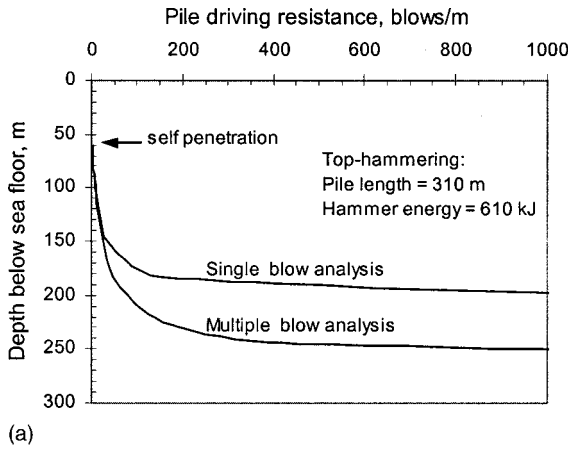


Fig. 6. Comparison of single and multiple blows for (a) top hammering and (b) bottom hammering

mering predicts less penetration than SBA because the tip part of the pile bounces back while the top part of the pile moves downward. In these cases, the pile will have a significant amount of residual stresses, especially at a great depth of penetration. Therefore, MBA should be used for bottom-hammering analysis when the soil resistance is high.

Gravity Effect

In reality, there is a significant gravity effect, which was not considered in the Smith work (1962). By comparing Eqs. (5) and (7), we know that there is an additional velocity of $g\Delta tBF$ if the gravity effect is considered. As can be seen in Fig. 7, the gravity effect is dominant if soil resistance is small. The pile keeps moving downward because of the additional velocity due to gravity after a hammer blow. As soil resistance increases with depth, the net gravity effect decreases. However, by ignoring the gravity effect on pile driving, early refusal is predicted. Refusal depths in Fig. 7 with and without considering the gravity effect are 260 and 240 m below the sea floor, respectively.

Effect of Soil Resistance Distribution

When a pile is being driven, soil resistance exists along the side of the pile and at the tip of the pile. For analysis of the pile-soil system, we may assume that an equivalent soil resistance exists along the side of the pile (distributed resistance), only at the tip of the pile (concentrated resistance), or at both side and tip of the

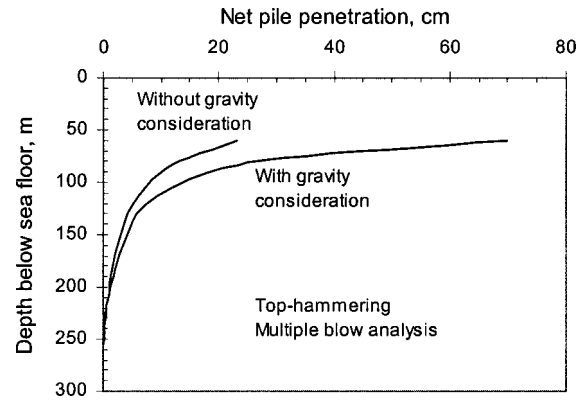


Fig. 7. Effect of gravity on pile penetration for top hammering using multiple-blow analysis

pile. Based on the input data in Table 2, analysis assuming distributed resistance predicts much higher penetration over the whole depth range mainly because the damping constant at the pile tip is three times higher than the damping constant along the pile. If the same damping constant is used, the two results are very close. Therefore, we must have accurate soil strength and soil parameter data for better analysis rather than selection of the resistance distribution type. Further study shows that the contribution of tip resistance to total resistance is less than 1% if driving depth is deeper than 120 m below the sea floor. Pile penetration difference without considering tip resistance is less than 3% under the same conditions. However, tip resistance can be critical in actual pile driving when a pile penetrates a massive sand formation.

Economic Considerations

The cost of pile driving depends on a number of factors: soil strength, pile configuration, hammer energy per blow, hammer blow frequency, cost of vessel operation for offshore operations, and pile installation depth. Among these, hammer blow frequency is found to be one of the most critical factors for deeper pile penetration because it is directly related to total time of pile installation at any design depth.

Fig. 8 shows the cumulative net driving time in hours versus depth below the sea floor assuming no job interruption at all. Pile driving interruption for any reason will cause soil setup, which

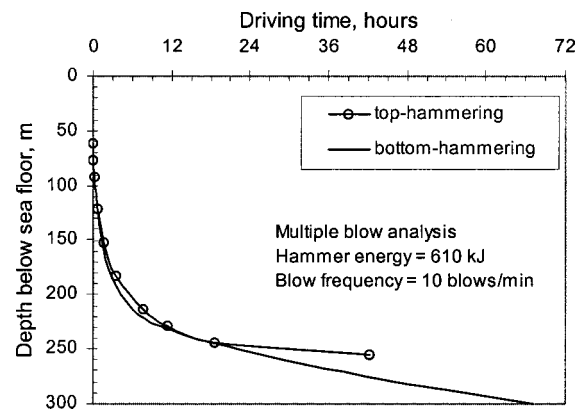


Fig. 8. Cumulative driving time versus depth for top and bottom hammering using multiple-blow analysis

usually results in hard driving and increase of total pile driving time. The assumed hammer frequency is 10 blows per minute. A different hammer frequency would change the scale of the graph, but not the shape of the graph. It should take less than 4 h to reach 230-m penetration depth below the sea floor for both top and bottom hammering. Bottom hammering would reach 300 m below the sea floor and would take about 23 h net driving time. However, top hammering would reach refusal beyond 260 m with the current hammer-pile configuration. Note the sharp driving time increase as the penetration depth approaches the refusal depth.

If it is essential to drive a pile to a greater depth such as 300 m to isolate a problem zone, bottom hammering can be one of the alternatives. Further sensitivity analysis shows that one or a combination of the following can be used for better pile penetration. Increased wall thickness makes the driving more efficient since it is easier to transmit the energy through a thick-walled pipe. This will make it possible to increase the hammer energy per blow without exposing the pipe to excessive stresses. Use of a smaller pipe will reduce soil resistance, and result in better penetration. However, there will be some restrictions due to hammer sizes that fit inside the pile for bottom hammering, stresses developed in the pile, final pile bearing capacity, or subsequent hole sizes to be drilled.

Conclusions

A wave equation analysis program is developed and verified using the Smith model (1962), the Edwards model (1969), and the Lowery et al. model (1969). The program handles long piles for both top and bottom hammering. It is also capable of simulating multiple-blow analysis as well as single-blow analysis. The following conclusions have been drawn from numerical analyses using the program in this study:

1. Single-blow analysis predicts refusal early for top hammering and predicts unrealistically high pile penetration for bottom hammering, especially for high soil resistance.
2. Multiple-blow analysis, which takes into account the effect of residual stresses in the pile, should be used for more realistic pile driving analysis and prediction.
3. The gravity effect on pile driving is significant for low soil resistance and thus needs to be considered for accurate predictions of pile penetration and refusal depth for both top and bottom hammering.
4. Bottom hammering is applicable for deep pile penetration. It offers better penetration per blow than top hammering because hammer impact energy is delivered to the point where soil resistance is highest, and reduction of the pile diameter due to Poisson's ratio effects will further reduce effective soil resistance.

Acknowledgments

The writers would like to thank DeepStar Consortium and Holt Chair for sponsoring part of this study. The help provided by Dr. L. Lowery in making some comparison runs is highly appreciated. This study was conducted through the Research Institute of Engineering Science at Seoul National University, Korea.

Notation

The following symbols are used in this paper:

- BF = buoyancy factor;
 $C()$ = spring compression of the pile element, cm;
 $D()$ = displacement of pile element, cm;
 $D'()$ = displacement of ground, cm;
 d_i = pile inner diameter, cm;
 d_o = pile outer diameter, cm;
 $F()$ = force exerted by spring, kN;
 $f(z)$ = remolded miniature vane shear strength at a depth z , kN/m²;
GRL = Goble Rausche Likins and Associates, Inc.;
 g = gravitational acceleration, 980 cm/s²;
 i = pile-hammer-soil system numbering;
 $J()$ = damping constant for i th pile element, s/cm;
 $K()$ = spring constant, kN/cm;
 $K'()$ = ground spring constant, kN/cm;
MBA = multiple blow analysis;
 $R()$ = resistance, kN;
RMVSS = remolded miniature vane shear strength;
SBA = single blow analysis;
 t = current time level;
 $t-1$ = past time level for the wave equation calculation;
 $v()$ = velocity, cm/s;
 $W()$ = weight of pile, kN;
 Δt = time interval, s;
 ρ_f = density of fluid, g/cm³;
 ρ_i = density of fluid inside the pile, g/cm³;
 ρ_o = density of fluid outside the pile, g/cm³; and
 ρ_s = density of steel, g/cm³.

References

- Briaud, J. L., and Tucker, L. (1984). "Piles in sand: A method including residual stresses." *J. Geotech. Eng.*, 110(11), 1666–1680.
- Brown, R. A., and Coyle, H. M. (1972). "Soil parameters required to simulate the dynamic lateral response of model piles in stiff sand clay." *Proc., Offshore Technology Conf.*, paper OTC 1695.
- Choe, J., and Juvkam-Wold, H. C. (1997). "Unconventional method of conductor installation to solve shallow water flow problems." *Proc., SPE Annual Technical Conf. Exhibition*, paper SPE 38625.
- Dutt, R. N., Doyle, E. H., Collins, J. T., and Ganguly, P. (1995). "A simple method to predict soil resistance to driving for long piles in deepwater normally consolidated clays." *Proc., Offshore Technology Conf.*, paper OTC 7668.
- Edwards, T. C. (1969). "Pile analysis wave equation computer program manual." *Research Rep. No. 33-11*, Texas A&M University, College Station, Tex.
- Fellenius, B. H. (1991). "Chapter 13: Pile foundation." *Foundation engineering handbook*, H. Y. Fang, ed., Van Nostrand-Reinhold, New York.
- Goble Rausche Likins and Associates, Inc. (1996). *GRL wave equation analysis manual*.
- Holloway, D. M., Audibert, J. M. E., and Dover, A. R. (1978a). "Recent advances in predicting pile driveability." *Proc., Offshore Technology Conf.*, paper OTC 3273.
- Holloway, D. M., Clough, G. W., and Vesic, A. S. (1978b). "The effects of residual stresses on pile performance under axial loads." *Proc., Offshore Technology Conf.*, paper OTC 3306.
- Juvkam-Wold, H. C., Choe, J., and Lowery, L. L., Jr. (1996). "Unconventional methods for shallow water flow conductor installation—Model validation and improvement." *Final Rep. to DeepStar Consor-*

- tium, CRT No. DP111 3501-D, Texas A&M University, College Station, Tex.
- Lowery, L. L. Jr., Hirsch, T. J., Edwards, T. C., Coyle, H. M., and Samson, C. H. Jr. (1969). "Pile driving analysis—State of art." *Research Rep. No. 33-13*, Texas A&M University, College Station, Tex.
- Rieke, R. D., and Crowser, J. C. (1987). "Interpretation of a pile load test considering residual stresses." *J. Geotech. Eng.*, 113(4), 320–334.
- Smith, E. A. L. (1955). "Impact and longitudinal wave transmission." *Trans. ASME*, 963–973.
- Smith, E. A. L. (1962). "Pile-driving analysis by the wave equation." *Trans. Am. Soc. Civ. Eng.*, 27, 1145–1193.
- Stevens, R. S., Wiltsie, E. A., and Turton, T. H. (1982). "Evaluating pile drivability for hard clay, very dense sand, and rock." *Proc., Offshore Technology Conf.*, paper OTC 4205.