*Application of Stress-Wave Theory to Piles, F. B. J. Barends (ed.) © 1992 Balkema, Rotterdam. ISBN 90 5410 082 6*

# Evaluation of measurements for vibratory hammers

G. Likins
*Pile Dynamics, Inc., Cleveland, Ohio, USA*

F. Rausche
*Goble Rausche Likins and Associates, Inc., Cleveland, Ohio, USA*

M. Morrison & R. Raines
*Exxon Production Research Company, Houston, Tex., USA*

ABSTRACT: The installation processes for conventional pile driving is often very time consuming and therefore expensive. Projects could be completed earlier if all facets of constructions took less time. Impressive claims in productivity are often made for vibratory pile hammers. However, confirmation of capacity is critical to engineers designing bearing piles. Traditionally, either extensive static testing or testing under subsequent conventional impact hammers have often negated any time or cost advantage gained by vibratory installations.

Piles were partially installed by a vibratory hammer at two sites. Soils ranged from loose to dense sands and soft to very stiff clay. Both piles were then driven to final elevation by a conventional impact hammer. Measurements under both vibratory and impact hammers provided a direct comparison of capacity evaluation techniques under both hammers; very good correlation was observed. Vibratory hammer performance was also evaluated. The methods of analysis are described and results presented.

## 1 SITE CONDITIONS AND INSTALLATION PROCEDURES

The 60-inch diameter mooring piles were installed at two offshore locations, (GRL & Associates, Inc. 1990) referenced 111 and 114, respectively. Details of the pile geometries and site conditions at the locations are presented in Tables 1 and 2.

Each pile was initially installed with an ICE-1412 vibratory hammer. This hammer, which can be operated at frequencies between 6.7 and 20 Hz, has a rated displacement amplitude of 1.5 inches and a power rating of 550 HP. the hammer vibrated the piles to penetrations (below mudline) of 43 and 63 ft in times of 4 and 9 minutes, respectively.

After vibratory "refusal", a Vulcan 060 steam powered impact hammer was used to drive the piles to their design penetration. For Pile 111, the blow count started at 34 blows per foot (BPF) and reached 96 BPF at the final penetration of 80 ft below mudline. For Pile 114, the blow count started at 75 BPF (at low hammer energy), decreased to as little as 32 BPF as the energy (and penetration) increased, and reached 114 BPF at the final penetration of 85 ft below mudline.

Table 1. Pile Geometries

| Reference | Length ft | Penetration ft | Pile description |
|---|---|---|---|
| 111 | 145 | 85 | 1 inch wall and 40 ft section with 1.5 in wall |
| 114 | 130 | 85 | 1 inch wall with 50 ft section of 1.5 in wall |

Both piles had internal baffles located near the transducer locations.

### 1.1 Pile installation monitoring and analyses

For impact measurements and analysis a standard PDA pile monitoring system was used (Rausche, Likins, Hussein 1988) with sensors attached approximately 17 ft below the pile tops. Since the PDA is self balanced after every blow and acquires relatively short records digitized at a high sampling rate, a different set of signal conditioning equipment was required under the steady state vibratory hammer. It included two manual balancing and amplifying strain units and two accelerometer power supplies. The data was recorded on an FM instrumentation tape recorder and viewed on an oscilloscope.

For all vibratory records, acceleration and strain analog data were replayed from the tape recorder through a 400 Hz Low-Pass filter and digitized at 2000 samples per second for each channel. The following calculations were then performed:

Table 2. Soil Conditions

| Reference (and water depth, ft) | Penetration ft | Soil description |
|---|---|---|
| 111 | 0 - 9 | Clayey fine sand |
| (42') | 9 - 44 | Soft to stiff clay |
| | 44 - 213 | Fine to coarse sand |
| 114 | 0 - 26 | Loose to medium dense fine sand |
| (22') | | |
| | 26 - 44 | Medium dense fine sand |
| | 44 - 79 | Firm to very stiff clay |
| | 79 - 162 | Dense fine sand |

1. Integration of acceleration to velocity including an integration constant (the average penetration rate). Integration of velocity to displacement.
2. Addition of the hammer weight to the measured force.
3. Integration of the product of force and velocity to yield transferred energy as a time function. Calculate power by dividing transferred energy by the time period.
4. Calculation of "soil resistance" by rigid body assumptions.
5. Calculation of the Fourier coefficients of acceleration and force. Calculation of a dynamic stiffness by dividing the force coefficients by the corresponding displacement values.

All resulting time and frequency curves were plotted and maxima, minima and amplitudes spans were determined and are summarized in Tables 3a and 3b. A sample plot result is included in Figure 1.

PDA results were obtained under the impact hammer from the beginning to the end of impact pile driving. In the present case, the primary interest was the correlation between bearing capacity predictions from the end of vibratory driving records and the beginning of impact testing. Therefore, a very early record of the impact driving of each pile was analyzed by CAPWAP.

Table 3a. Summary of vibratory results, Pile 111

| Pentr. ft | Rate of Pentr. ft/s | Freq. Hz | Force Span kips | Accel. Span g's | Power HP | Resistance Max. Min. kips |
|---|---|---|---|---|---|---|
| 20 | .167 | 17.6 | 423 | 3.9 | 66.4 | 330<br>-178 |
| 26 | .250 | 17.6 | 458 | 3.4 | 79.8 | 344<br>-250 |
| 41 | .125 | 16.6 | 499 | 6.0 | 184 | 426<br>-473 |
| 43 | .009 | 16.6 | 510 | 5.8 | 171 | 421<br>-479 |

Table 3b. Summary of vibratory results, Pile 114

| Pentr. ft | Rate of Pentr. ft/s | Freq. Hz | Force Span kips | Accel. Span g's | Power HP | Resistance Max. Min. kips |
|---|---|---|---|---|---|---|
| 15 | .167 | 16.6 | 411 | 8.4 | 145 | 420<br>-415 |
| 35 | .208 | 16.6 | 423 | 7.8 | 138 | 404<br>-311 |
| 45 | .075 | 15.6 | 451 | 10.8 | 187 | 810<br>-473 |
| 60 | .011 | 17.6 | 528 | 6.5 | 107 | 633<br>-470 |

## 2 DISCUSSION OF RESULTS

### 2.1 Results from Fourier Analysis

The Fourier coefficients of both acceleration and force contained the very dominant peak of the vibratory hammer's base frequency. Higher harmonics are also apparent. The base natural frequency of the two piles were 58 and 65 Hz for Pile 111 (L = 145 ft) and 114 (L = 130 ft), respectively. The 58 Hz pile frequency was usually indicated in the acceleration records of Pile 111. For Pile 114, the 4th harmonic (60 to 68 Hz) of the hammer frequency is very near the pile's base frequency and the pile frequency was only apparent in the record at 55 ft depth.

An attempt was made to use the Fourier analysis for a calculation of soil response. Probably, the low frequency stiffness values can be related to the soil response. However, at this time no experience exists and further study is needed.

### 2.2 Hammer Performance

For the vibratory hammer, no readily available measured performance data exists in terms of power, maximum force, etc., with which to compare the current results. However, these quantities may be compared with manufacturer's specifications. For example, the rated 10 kip-inch eccentric moment produces centrifugal forces of 295 kips at a frequency of 17 Hz (the most commonly observed hammer speed). The force amplitude (peak-to-peak) would then be 590 kips. The observed force span amplitudes ranged from 360 and 528 kips. The highest stresses are due to extreme stress concentrations under the clamp, and the low average axial pile stresses are therefore immaterial as far as the potential of pile damage is concerned.

The maximum power transmitted to the piles was 187 HP. There is, of course, appreciable energy (power) remaining in the hammer itself (rotating weights, etc.) and it is not known how much power was actually consumed. Furthermore, the power output depends on the hammer's operating frequency which was always less than the rated 20 Hz. All of these reasons help explain why, at most, only 34% of the rated power (550 HP) was transferred to the pile.

### 2.3 Soil Resistance and Bearing Capacity

The soil resistance was calculated under the vibratory hammer based on a rigid body pile model. This is justified since the loading frequency (18 Hz at most) is significantly lower than the pile's natural frequency (about 60 Hz). The elastic behavior of the pile under such relatively slowly varying loads is therefore of lesser importance than under impact hammers.

The difficulty in assessing the bearing capacity of piles under vibratory loadings lies in the degradation of resistance due to the continuous pile motion. The calculated values are therefore only an estimate of the resistance at the time of driving. The positive values are present under the downward and the negative under upward pile motions. No allowance has been made for any dynamic (viscous) resistance effects. Since velocities were generally less than 1.0 ft/s damping forces were probably also small. It is believed that the resistances encountered do contain some dynamic components; however, the static resistance is degraded (liquefaction, no set-up considered). Reducing

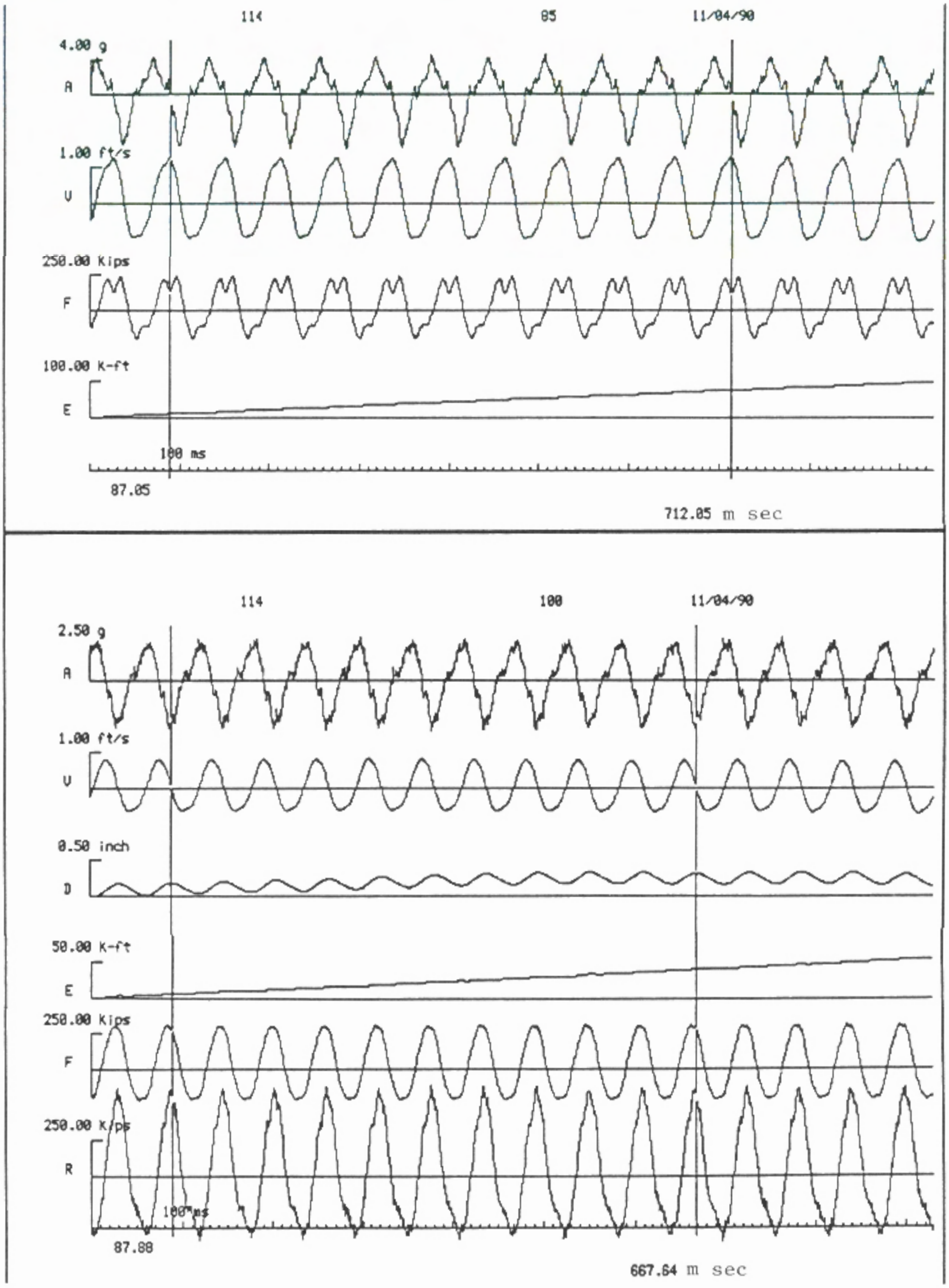

Fig. 1 Data for Pile 114 at a) 15 ft above final elevation and b) final elevation.

the total resistance encountered by the dynamic component (maybe as much as 20% in the sand) would therefore not improve the accuracy of the prediction as perhaps the two effects cancel.

The maximum positive resistance values encountered at the end of vibratory driving were 421 kips for Pile 111 and 633 kips for Pile 114, respectively. The corresponding CAPWAP (Rausche, Moses, Goble 1972) results from the early driving were 474 and 658 kips. The good correlation between the soil resistance results from vibratory and impact data is rather encouraging. Actually, this good correlation may be a coincidence because (a) no dynamic (damping) resistance was deducted from the total vibratory resistance. (The maximum occurred after the time of the highest positive pile velocity but before the velocity zero) and (b) the static resistance occurring during vibratory driving is usually thought to be less than the static resistance occurring during impact driving (liquefaction).

It should be noted that to achieve a match in CAPWAP, significant impedance increases were required in the pile model. For both piles, the impedance was increased in the baffle area. The impedance of the heavy wall section was increased for both piles (but particularly high variations were made for Pile 114). These could be mass effects of the soil or perhaps, more likely, local stress concentrations due to the baffle plates near the transducer locations.

A static analysis for Pile 114 using API RP 2A (1989) Method gave 677 kips ultimate shaft resistance (ratioed to actual 60 inch pile from calculated 36 inch pile result) which is in good agreement with both dynamic measurement results. For Pile 111, the similarly predicted shaft resistance was 377 kips, also in good agreement; however, the end bearing was over estimated in the static analysis.

## 3 CONCLUSIONS

A method of measurement and analysis of dynamic data acquired during vibratory driving is presented. Capacity calculation from the vibratory driving compare favorably with both static analysis predictions and dynamic impact test analyses using CAPWAP. Compution of forces and power were lower than the manufacturer's rated values.

## REFERENCES

Goble Rausche Likins and Associates, Inc. 1990. Project report pile installation by vibratory and impact hammers.

Rausche, F., Likins, G. and Hussein M. 1988. Design and testing of pile foundations. Ottawa, Canada.

Rausche, F., Moses, F. and Goble, G. 1972. Soil resistance predictions from pile dynamics. Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 98, No. SM9, Proc. Paper 9220: 917-937.