

Dynamic Testing of Piles Socketed into Weak Rock

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SUMMARY Traditional dynamic pile tests are reviewed and compared with the wave equation method of analysis. The paper then describes results of tests on large diameter piles socketed into weak rock. The tests incorporated instrumentation to provide data for the wave equation analysis.

1. INTRODUCTION

Engineers traditionally associate dynamic testing of piles with the procedure of assessing safe working load using the so-called "dynamic" pile driving formulas. These procedures use simple field measurements of pile penetration and compression, in a variety of empirical formulas based on Newtonian mechanics of rigid bodies. The results obtained are in most cases of unknown reliability and in some cases can be highly misleading.

This paper describes a very different form of testing incorporating electronic measurement of the dynamic response of the pile in a computerised wave equation analysis. This method enables the engineer to evaluate hammer performance confidently and provides estimates of load capacity based on a more realistic physical model of the pile and soil behaviour. The wave equation provides the facility to predict stresses in the pile and also to detect structural inadequacies in the pile shaft.

A brief review of the basis of traditional pile driving formulas is presented with a discussion on their limitations. The more rigorous wave equation analysis is then described together with the instrumentation and interpretation techniques on which the dynamic testing method is based. Some results of recent testing carried out on piles for a bridge project in Melbourne are presented to demonstrate the capabilities of this testing method.

2. TRADITIONAL PILE DRIVING FORMULAS

Traditional dynamic pile driving formulas are based on the simplistic assumption that the ultimate load capacity of the pile is equal to the ultimate driving resistance. There have been literally hundreds of these formulas developed, each neglecting various aspects of the problem by omission of factors that were not considered to have any serious effect on the results within the ranges of pile driving situations for which they were derived. Chellis (1961) presents a comprehensive discussion on the derivation of 38 formulas which have been used in the United States, and two of these which have been used commonly in Australia are described briefly below.

The Engineering News formula represents a group of formulas containing empirical factors which attempt to approximate observed pile behaviour. This formula is based simply on the assumed hammer energy and the measured set of the pile and contains a fixed co-efficient designed to compensate to some degree for elastic losses in helmet, pile and soil.

The formula was designed for driving light timber piles in sands. The factor of safety of six incorporated in the formula is an indication of its supposed reliability, even under the conditions for which it was specifically designed. Despite the obvious limitations of this formula its use has been extrapolated to situations well beyond those for which it was developed and it is still used in some specifications.

The Hiley formula is probably the most commonly specified in current practice. This formula is based on a more rigorous analysis of the efficiency of the hammer blow and the energy losses in the pile, soil, and driving equipment due to temporary compressions during penetration of the pile. The pile is treated as a rigid body and the transfer of energy to the pile is calculated by considering the conservation of momentum of the hammer and pile during impact. Losses of energy are accounted for by assuming that soil, pile, and cushion materials are linearly elastic. A factor of safety varying from 2.5 to 4 is used with the formula to obtain a safe working load but the actual margin of safety afforded by this approach may be significantly different to that inferred by this factor. The formula takes no direct account of pile length, material or cross-sectional area and cannot differentiate between different distributions of resistance. Very misleading results can be obtained from this formula if hammers with insufficient energy, as is often the case due to an over estimate of efficiency, are used to obtain the measurements of set and compression.

3. ANALYSIS OF PILE DRIVING USING THE WAVE EQUATION

Engineers have persevered with the use of the traditional pile driving formulas for over one hundred years despite well-founded criticisms of their accuracy and reliability. The use of these formulas has persisted in spite of criticism because of their simplicity and the lack of an attractive alternative. It was because of these shortcomings that foundation engineers such as Smith (1962) pursued development of the more rigorous elastic solution to the pile driving problem known generally as the wave equation approach. The one dimensional wave equation can be derived by considering the propagation of motion along an elastic rod, yielding a differential equation relating acceleration to the strain gradient according to the following formula :

$$\frac{\delta^2 u}{\delta t^2} = c^2 \frac{\delta^2 u}{\delta x^2}$$

In this formula the wave speed, or celerity, c , is a material property and can be calculated from the modulus of elasticity (E) and the mass density (ρ) as follows :

$$c = \sqrt{\frac{E}{\rho}}$$

It is only recently that direct solutions to this differential equation have been used to analyse pile behaviour (e.g. Hansen and Denver, 1979). However, a more convenient numerical solution was proposed by Smith (1962) and has been used as a basis for most of the commonly available "wave equation" programmes. In this solution the pile is modelled as a series of lumped mass and spring elements and the soil resistance is applied as a series of discrete resistances to each lumped mass. The passage of the wave through the pile is analysed by considering the dynamic equilibrium of the elements in successive increments of time and calculating the velocity, displacement, and force for each mass element. Computer programmes using this solution method have been described by Bowles (1974) and Goble and Rausche (1981).

The analysis proceeds by firstly calculating the striking velocity of the pile driving hammer based either on the free fall height, or in the case of the more complicated mechanical hammer programmes, the fall of the ram and the thermodynamic characteristics of the hammer assembly (Goble and Rausche, 1981). At each pile element the displacement can be calculated from the velocities during the previous time increment. From these displacements and velocities the resulting element forces can be determined taking into account the spring deformations and the adjacent soil resistance. The element accelerations can then be calculated and the resulting velocity can be calculated by integration. The computation then proceeds to the next time increment.

In the model proposed by Smith (1962) the soil resistance at each element is modelled as an elastoplastic spring in parallel with a viscous damper. The static resistance at each point on the pile increases linearly with pile movement until a limiting movement known as the quake is reached. The elastic stiffness of the soil up to this stage is calculated by the programme from the specified ultimate static resistance at the point being considered, and the quake, which is usually specified as a single value for the whole length of the pile. The distribution of resistance may be varied along the length of the pile. When the quake is exceeded at any point the pile continues to move without increase in static soil resistance. The dynamic component of resistance is assumed to vary linearly with velocity and static resistance. The total static and dynamic resistance is thus given by the equation :

$$R = R_u (1 + J \cdot v)$$

where R is the total resistance, R_u is the static resistance, J is the dynamic damping factor, and v is the instantaneous velocity of the pile segment being considered. The typical behaviour of combined static and dynamic resistance during penetration and rebound of a pile is shown in figure 1.

4. VALIDITY OF THE WAVE EQUATION ANALYSIS

The validity of the wave equation as a means of estimating pile capacity has been extensively tested theoretically and experimentally. The published results indicate that the technique can be used with confidence provided that the hammer energy characteristics are known and the soil parameters can be estimated.

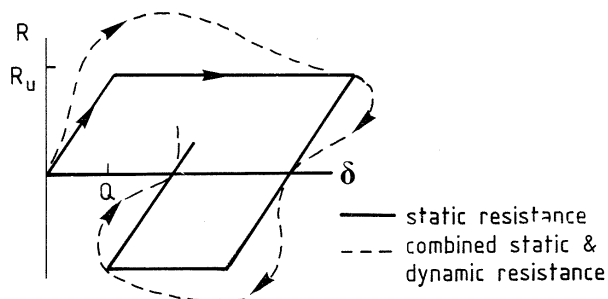


Figure 1—Force displacement relationship for a pile element

It will be appreciated that the behaviour of real soils cannot be described accurately by the simple mathematical model described previously. Such a model cannot pretend to account for changes in soil strength with time such as occurs in the case of "set up" where the bearing capacity of piles driven in clays may increase due to dissipation of pore pressures, or in the case of "relaxation", where dense fine sands may exhibit a decrease in strength after driving. Notwithstanding the difficulties associated with soil modelling, the predictions of static pile capacity achieved using the wave equation have been very impressive. Comparisons between static load test results and wave equation predictions have been reported by a number of authors including Crapps (1977), Goble et al (1980), Gravarre and Hermansson (1980) and Thompson and Devata (1980).

5. INSTRUMENTED DYNAMIC LOAD TESTS

Methods for direct measurement of pile head force and pile head velocity were developed between 1965 and 1975 at the Case Western Reserve University by Rausche and others under the direction of Professor G Goble. The measured force and velocity could then be used instead of the predicted hammer energy as input in the wave equation programme thereby eliminating this uncertainty and significantly improving the reliability of the technique. In the method developed by these workers a strain transducer and an accelerometer are bolted to the top of the pile being tested to provide a record of pile head force and pile head velocity as a function of time. The measured force and velocity are used in a simplified field analysis method (the CASE method) or in a more sophisticated wave equation analysis (the CAPWAP method) to analyse the behaviour of soil and pile. These techniques are described in detail by Rausche et al (1972) and by Goble et al (1980). The CAPWAP method of analysis is described briefly below :

- (a) Traces of pile head force and pile head velocity are obtained during driving of the pile as described above.
- (b) The pile head velocity is input as a boundary value in a wave equation analysis of an equivalent pile without soil resistance.
- (c) Elements of soil resistance based on estimates of soil parameters, are progressively applied to the "free" pile in the wave equation until the computed force curve matches the force measured in the field as closely as possible.

The depth of a particular element of resistance on the pile determines at what time it will affect the pile head force trace (that is, the time for the reflected wave to reach the top), the magnitude of the resistance will affect the magnitude of the wave reflected, and the stiffness characteristic (that is, elastic, plastic, or viscous) will affect the time that it takes for the reflected wave to attenuate.

- (d) In obtaining a match between the measured force and the computed force the analysis will provide a distribution of the soil resistance parameters; R_u (the static resistance), Q (the quake) and J (the dynamic damping factor) along the length of pile. The static resistance parameters R_u and Q can then be used to obtain a static load settlement curve for progressive increments of static pile head load. In this way the ultimate static capacity of the pile is obtained.

6. DYNAMIC TESTS ON ROCK SOCKETED PILES

During the past 18 months the RCA has been carrying out a program of selective proof loading of piles for the West Gate Freeway structures in South Melbourne. The piles, which are 1.1 to 1.5m in diameter and typically 45m long, extend through the deep alluvial deposits to a socketed foundation in the Silurian mudstone basement or the overlying basalt flows. The piles were constructed by either vibrating or percussing a steel casing to refusal in the rock and then excavating a socket into the rock using a bucket auger. A detailed account of the construction methods adopted for these piles has been presented by Holden (1984).

Very little information was available on the dynamic testing of large cast-in-place piles, and to the author's knowledge the application of this method to large diameter rock socketed piles is unique. It was therefore considered necessary to carry out a series of trial tests on several piles which had been previously static load tested, to provide correlation between the two methods and a measure of confidence in the dynamic method.

The need for such reassurance can be readily appreciated when the requirements of the two test methods are compared. For example, a static load test on a 1.5m diameter pile might involve application of a load of 20MN to the head of the pile in a loading programme lasting several days. This loading could be achieved by using 2000 tonnes of kentledge (an awesome prospect) or by installing rock anchors to provide the load reaction. Even in a large test programme the cost of such a procedure would be at least \$25,000 per pile, and would take 2 to 3 weeks to prepare. In comparison the dynamic test seems trivial. The test involves dropping a 20 tonne hammer onto the head of the pile from a height up to 2.5 metres. Up to 5 piles can be tested per day and the cost of each test is less than \$4,000, including the purchase of the necessary equipment, training of staff, fabrication of hammer and leaders, and other incidental costs.

A pilot programme of testing was carried out in mid-1982 to evaluate the dynamic testing method. Twelve piles were tested, nine of which had previously been tested statically. The testing and analysis was carried out for the RCA by the consultants Goble and Associates of Ohio, USA, and the site work was carried out by contractor Austral Piling of Melbourne. The leaders for the hammer and the trip mechanism were designed and fabricated by the RCA. Some results of the testing are described briefly below to give a general indication of the capability of the test method.

6.1 Test Results for a Sample Pile

This particular pile was 1.5m in diameter and 43.7m long. The steel casing passed through 34m of silt and gravel and was driven into the weathered siltstone to obtain a seal. The rock socket was then excavated 6.4m below the bottom of the casing. The completed pile had been statically tested to a load of 19.9MN using rock anchors to provide the loading reaction. The pile was given a total of eight blows of the hammer, the first 4 with a drop height of 2.5m and the last 4 with a drop height of 2m. The energy transferred to the pile varied between 40% and 60% of the available kinetic energy of the hammer. During the test the pile settled a total of nearly 5mm with a maximum set per blow of approximately 1mm. The maximum head force recorded was 50.7MN corresponding to a head stress of 28.7 MPa. A typical field record of force and velocity obtained in the dynamic test is shown in Figure 2.

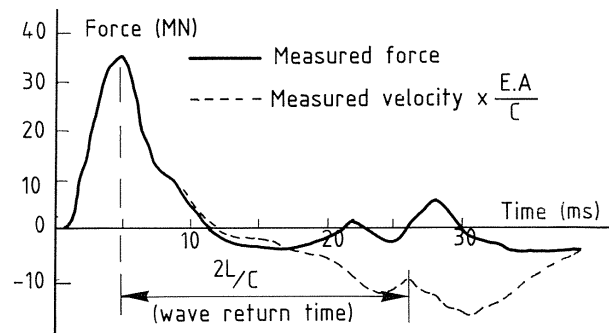


Figure 2—Field record of pile head force and velocity

The measured force and velocity shown in Figure 2 are proportional for the first 11 ms of the record indicating that reflected waves were not affecting the response at the head of the pile. Between 11 and 14 ms the velocity trace rises slightly relative to the force trace indicating a reduction in the impedance of the pile due in this case to the termination of the reinforcement cage. After 15 ms the velocity trace falls below the force trace indicating that the compression wave is being reflected by resistances in the pile socket area.

The results of the wave equation analysis of these field records are presented in figures 3 and 4. In figure 3 the force measured at the head of the pile is compared with the force calculated in the wave equation using the measured velocity as a boundary condition. As can be seen the match between the measured and calculated force is very good indicating that the soil modelling was particularly successful. In general, matches achieved between measured and calculated forces showed greater discrepancies than illustrated here.

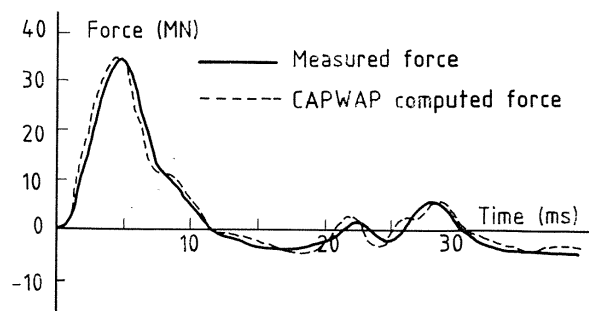


Figure 3—'CAPWAP' force match

From the resistance distribution and the assumed quakes the analysis programme calculates a static load versus settlement relationship by progressively applying increments of head load. The results are shown in Figure 4.

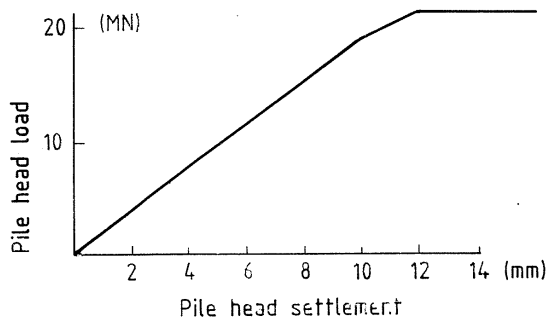


Figure 4-Derived load-settlement curve for pile

6.2 Correlation with Static Load Test Results

The distribution of resistance predicted in the dynamic analysis is plotted in Figure 5 together with the distribution of resistance inferred from strain readings in the earlier static load test. The side resistance predictions provided by the two tests are remarkably similar. Similar comparisons available for 2 other piles showed comparable correlation. The base resistance obtained in the dynamic test is higher than predicted by the static test as would be expected because in the dynamic test the temporary compression of the rock produces a higher resistance response.

- Static test resistance distribution
- Dynamic test resistance distribution

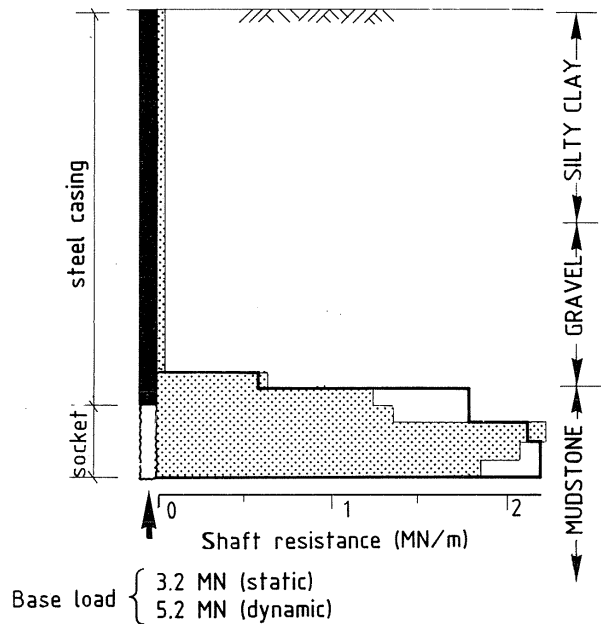


Figure 5-Resistance distributions by static and dynamic test methods

Theoretical resistance distributions obtained from conventional analyses showed no obvious correlation with the static or dynamic test results and this observation was supported by the results obtained in other static and dynamic tests.

The ultimate static load predicted by the dynamic test was compared with an ultimate load inferred from the static load tests. The static test ultimate load was obtained as shown in Fig.6 where the rate of increase of settlement with respect to load is plotted against the settlement. This construction (Chin's method) is based on the assumption that the load settlement curve approaches the ultimate load hyperbolically. The inverse of the gradient of the line thus provides an estimate of the ultimate load. In the case of this pile the ultimate load inferred from the static load test results was 31MN compared with a predicted ultimate load from the dynamic test of 21MN. On this basis the dynamic test underestimated the ultimate load by approximately 30%. It should be noted that the ultimate static load estimated using this method would itself be an underestimate of the true ultimate capacity due to the significant increases in base capacity with increasing settlement demonstrated by these piles.

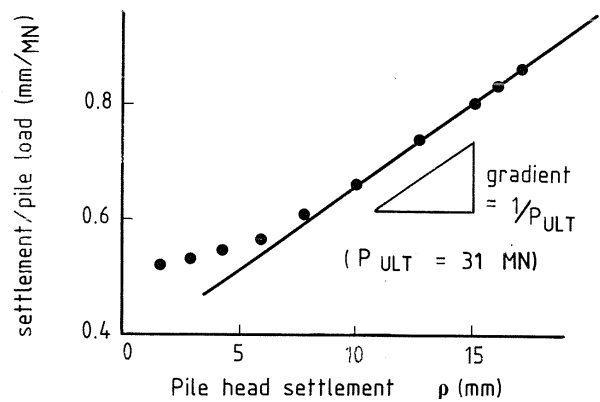


Figure 6-Estimation of ultimate load from static test

The static ultimate loads which could be derived in this way for other piles in the test series are compared with the ultimate loads predicted by the dynamic tests in Figure 7. In all cases the dynamic testing underestimated the ultimate capacity inferred from the static tests.

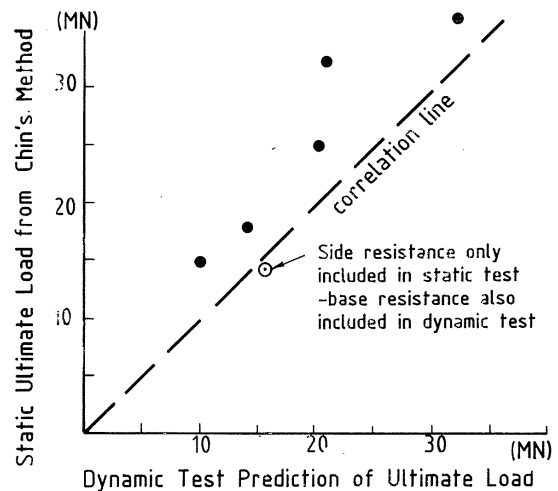


Figure 7-Comparison of ultimate load predictions by static and dynamic tests

6.3 Effect of Testing on Pile Capacity

Dynamic pile tests are normally carried out on driven piles where the test conditions are essentially the same as the conditions prevailing during installation. In the case of the tests described here, however, the piles were not designed to be driven, but were cast in situ after installation of the steel casing. It was therefore not clear what effect driving would have on the pile shaft and there were understandable doubts about the possible effects of dynamic loading on the mechanism of load transfer within the pile socket. These aspects are discussed in the following paragraphs.

6.3.1 Effect of Testing on the Pile Shaft

The maximum head compression recorded was 32.5 MPa, caused by the 20 tonne hammer falling 2.5m onto a head cushion comprising 3 alternating layers of 20mm plywood and 12mm steel plate. Whilst this stress was higher than originally proposed it caused no visible damage to the pile head. It should be noted that the head stress is almost independent of the driving resistance, which may seem surprising when viewed from the traditional rigid pile model, but is in fact quite sensible when one considers that the head stress reached a maximum before the compression wave had progressed half-way down the pile.

The maximum shaft tension stress predicted in the tests was 6.8 MPa, and was obtained on a pile with a significant (2mm) set per blow indicating a low base resistance. This tension would have occurred in the unreinforced pile socket, but did not appear to produce any distress. If a crack had developed it would have been apparent as a significant rise in the velocity trace relative to the force trace in the early stage of the record. This characteristic is used in assessing pile shaft damage and is described in detail by Goble et al (1980). A series of tests on one pile produced a result which indicated damage and subsequent coring revealed that a crack had probably developed at an unreinforced construction joint which appeared to be contaminated by a layer of silty fines.

6.3.2 Effect of Testing on Pile Capacity

In these piles the load is transferred to the rock as compression on the pile base and friction along the sides of the pile socket. The mechanism of load transfer has been described by Williams (1980),

During testing the piles are driven into the rock, leading as would be expected, to an increase in the base capacity. The increases predicted in subsequent blows on piles in the test series were generally consistent with the permanent sets achieved. More importantly, on several piles believed to contain significant inclusions of soft base debris, the base was driven down to firm contact with the rock producing significant increases in capacity. One pile was driven a total distance of 55mm leading to a reduction in permanent set from 7mm per blow to less than 0.5mm per blow.

In practically all of the tests the side shaft resistance appeared to be reduced by consecutive blows. This reduction was believed to be due to a progressive rise in pore water pressure in the rock mass and clay filled joints. Two of the piles were re-tested after a period of 6 months and the results indicated that the side shaft resistance had been restored, presumably due to dissipation of pore pressures.

7. ACKNOWLEDGMENTS

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