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## EXPERIENCES WITH HEAVY DROP HAMMER TESTING OF ROCK-SOCKETED SHAFTS

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Since the early 1980s, when the Road Construction Authority of Victoria authorized the dynamic proof testing of roughly 100 drilled shafts of 1.5 m diameter and typically 60 m length in Melbourne Australia, dynamic load testing has been applied worldwide for quality control of drilled shafts. Today, test capacities of 4000 tons or more are not unusual. On numerous occasions the authors have been involved in similar tests in the United States and test engineers from other countries have reported on experiences in many different countries.

Without doubt, generating high test loads is most economically accomplished by dynamic load testing and this method is particularly well suited for testing piles founded in rock because of the rocks low energy dissipation. However, deep foundation professionals have on occasion expressed reservations about subjecting a drilled shaft to impact loadings fearing, for example, a degradation of the shaft – rock interface properties or potential error sources due to uncertain concrete properties or shaft geometry.

This paper will present numerous case studies, demonstrating the performance of rock socketed drilled shafts under dynamic loads. It will discuss the mechanics of the dynamic tests and the method of data interpretation, addressing the concerns stated above and investigating the limitations of the test method. The paper will formulate a set of recommendations for the design and execution of these tests.

### Introduction

Dynamic load testing is widely used on driven piles where the pile driving hammer provides the loading device and sensors mounted to the pile near the top acquire strain and acceleration signals which are then analyzed to extract the static pile bearing capacity. Expanding this technology to augered piles or drilled shafts is a natural development process. In fact, the first dynamic tests on a large drilled shaft were conducted in Mexico in 1974. One of the earliest large scale correlation test series was conducted in Melbourne, Australia on 1.5 m diameter shafts (Seidel and Rausche, 1984). This Class A series of static and dynamic tests produced satisfactory agreement and more than 100 additional production shafts were then subjected to dynamic pile load testing. The subsurface conditions included both mudstone and basalt as a bearing layer. Since these early

tests, further correlations were made in many parts of the world, and a significant amount of data has been published that suggests good correlations have been achieved in many parts of the world (Likins et al., 2004).

### Why Dynamic Testing of Drilled Shafts

The modern design process for deep foundations requires a relationship between factor of safety and quality assurance. Quality assurance concerns both the structural integrity of the element as well as its geotechnical strength. Structural integrity can be assessed with low strain testing or cross hole sonic logging, however, even if the shaft is of perfect quality, it may have insufficient bearing capacity due to the variability of the site conditions. Thus, while static testing may yield important information about the basic properties of the bearing layer's geomaterials, such tests would

not give sufficient information about the quality of production piles. It is therefore important to choose additional testing methods which have minimal interference with the progress on the construction site, generate loads that are adequate for their intended purpose and can be conducted in a short time period. Compared to other loading methods, the dynamic test requires only a small foot-print, and is relatively quick and inexpensive.

### **Testing and Analysis**

Dynamic testing requires that (a) an impact load is generated with a cushioned drop hammer, (b) measurements of pile top force and velocity are taken during the impact loading with four strain sensors and four accelerometers connected to a Pile Driving Analyzer® (PDA) and (c) an analysis is conducted that reduces the dynamic force - motion measurements to a static load - set curve. The most commonly performed analysis is called CAPWAP® (Pile Dynamics, 2006); based on an elastic pile model and a static, elasto-plastic and a viscous dynamic soil model, it matches computed with measured signals in a trial and error type signal matching procedure. Figure 1 shows the PDA in the foreground and the so-called APPLE drop weight in the background. In this example, the ram has a weight of 40 tons. It is guided by a frame that also supports the ram's weight prior to its release (to avoid crane whipping).

The impact load should be sufficient to generate a permanent pile penetration, if the ultimate pile capacity is to be activated and therefore calculated by the dynamic analysis. However, when a shaft is drilled into a good quality bedrock formation, it would be unwise to attempt to reach the ultimate capacity. In fact not even static testing, whether top or bottom loaded, attempts to reach the ultimate in such cases as it would likely cause a failure of the shaft's concrete. Thus, it is generally satisfactory to activate a proof load. The testing procedure therefore begins with a relatively low impact (drop height) and upon calculation of the activated capacity by the PDA the impact energy is either increased or decreased by modifying the cushioning and/or the drop height. It is recommended that at least two impacts are recorded since the first one usually is chosen with a conservative energy and stress level.



Figure 1. 40 ton drop hammer on 78" diameter shaft with PDA in foreground

### **Examples**

Eleven pile tests from 5 different sites are presented in this paper and summarized in Table 1. Fall height and set results represent the highest energy impacts among the two to four impact loads applied. These cases were chosen because of their similar rock socket properties leading to a very stiff response during the tests with very little permanent penetration. Cases 1-1 through 1-4 were shafts of varying socket diameters and socket length drilled into a hard limestone. The bedrock of Sites 4 and 5 were of a similar type and quality with strength values in excess of the concrete strength of the shafts. Case 3-1 was installed by the Auger Cast method into a Florida limestone which had SPT values ranging from that of a soil to 50 blows for 1 inch at the bottom of the shaft. This pile was also statically load tested to 1600 kips capacity; the load-set curve was however a straight line which did not indicate a failure. On Site 2 the rock sockets of Shafts 2-2 and 2-3 penetrated through weathered into sound claystone. For Case 2-1 the socket material of shaft included a coal seam near the toe and the

socket material was designated in Table 1 as a Soft Claystone. A much larger number of shafts were tested on Site 4 than the 2 cases selected

as examples. However, the two shafts described here yielded the highest and lowest capacities in the analysis.

Table 1: Example cases

Site Shaft	Socket dia inch	Socket length ft	Shaft Length Ft	Rock Type*	Ram Weight kips	Drop Height inch	Pile Set inch	ETR <sup>x</sup> ratio
1 - 1	60	5.0	83.0	LS	80	24	0.03	0.34
1 - 2	78	6.5	81.5	LS	80	36	0.04	0.20
1 - 3	78	6.5	71.5	LS	80	36	0.01	0.21
1 - 4	42	3.5	68.0	LS	80	18	0.01	0.20
2 - 1	42	32.0	48.0	CS+	40	48	0.05	0.64
2 - 2	42	37.0	50.5	CS	40	38.4	0.01	0.48
2 - 3	42	37.0	37.0	CS	40	43.2	0.01	0.59
3 - 1	36	27.0	38.0	FL-LS	40	44	0.01	0.59
4 - 1	36	6.2	63.3	LS	46	150	0.01	0.09
4 - 2	36	5.4	68.4	LS	46	150	0.03	0.11
5 - 1	78	16.0	63.5	LS	40	48	0.01	0.13

\*LS - Limestone; CS - Claystone; FL-LS - Florida limestone; CS+ Soft claystone

<sup>x</sup>ETR - Energy Transfer Ratio: Energy measured at sensor location divided by ram weight times drop height

All shafts with the exception of 3-1 were production piles and for that reason careful attention was paid to stresses during the tests and both drop heights and the number of impacts applied were limited. The larger two of the four shafts on Site 1 (Shafts 1-2 and 1-3) were designed for almost 4000 kips working load and for that reason, a 40 ton ram was employed for testing on this site. The remaining shafts were tested with a ram weight of roughly 20 tons. On average, the ram weight was less than 1.3% of the calculated activated capacity values (see below). This is in good agreement with the rule-of-thumb recommendation that the ram weight should be at least 1% of the required test load for shafts founded in rock. For shafts founded in soils, a minimum ram weight of 2% of test capacity is normally recommended. Thin plywood cushions served to moderate the impact stresses in all cases. No attempt was made in any one of these eleven cases to activate the ultimate pile capacity and only moderate fall heights were chosen. An exception was the testing on site 4; there the ram consisted of a compressible material and relatively high impact velocities had to be chosen to activate sufficient test capacity. Consequently, the pile sets were very small.

Table 2 summarizes some of the measurement and analysis results. The measurement results

include the maxima of force, velocity, displacement, all at the point of measurement, and the maximum pile toe displacement. The CAPWAP analysis results include the activated capacity and its toe response. Because of the rather small pile movements, the ultimate end bearing was probably not reached in any one of the eleven cases.

## Discussion of Results

### Shaft Displacements

Table 2 shows that the maximum displacements of most test piles are rather small reaching little more than ¼ inch at the top and less than 1/10 inch at the bottom. An exception is Case 2-1 which did not have the typical rock response that the other shafts displayed. The maximum toe displacements show a similar tendency. The difference between the maximum top and toe displacements is the elastic shortening of the shaft during the test. The difference between the maximum displacements and the final set is the shaft rebound. Note that a set of 0.01 inches is within the accuracy of the measurements and can therefore be considered a zero permanent penetration.

Table 2: Analysis results

Site Pile	Fmax top	Vmax top	Vmin Top	Dmax top	Dmax toe	Ract total	Ract toe	Rdyn total
	kips	ft/s	ft/s	inch	inch	kips	kips	kips
1 - 1	4260	2.6	-2.7	0.22	0.08	4400	1100	2090
1 - 2	5320	2.2	-2.9	0.15	0.03	8480	4590	1180
1 - 3	5830	2.0	-2.6	0.14	0.04	8120	4250	1830
1 - 4	3190	2.2	-1.8	0.14	0.04	3320	770	960
2 - 1	4700	7.2	-2.9	0.42	0.34	2140	810	2845
2 - 2	3280	6.1	-3.6	0.26	0.09	4480	300	2560
2 - 3	4340	5.5	-3.8	0.26	0.09	4255	1780	2655
3 - 1	5560	7.0	-5.5	0.26	0.03	5380	2490	4900
4 - 1	4115	6.5	-3.1	0.26	0.06	5400	870	2000
4 - 2	3270	6.3	-1.4	0.25	0.09	3220	660	2340
5 - 1	3566	1.1	-1.7	0.10	0.03	4050	1020	1273

**Velocity Response**

Two examples of measured force and velocity data are shown in Figures 2 and 3 for Cases 1-2 and 2-1, respectively. The velocity curves are presented after multiplication with the pile top impedance as a proportionality factor. The force pulses generated by the impact are very similar in Figures 2 and 3 while the velocity traces are remarkably different: the velocity response in Figure 2 after the first peak steadily decreases and reaches a negative (upwards) velocity peak

whose absolute value is greater than the positive peak. Figure 3, on the other hand, shows first a positive second peak at the time when the stress wave returns from the pile toe and only later negative values of moderate magnitude. Clearly, in the second case the velocity response is more delayed and the rebound is much less pronounced than in the first case and is also typical of the response that is observed when testing piles driven or drilled into soils rather than hard rock.

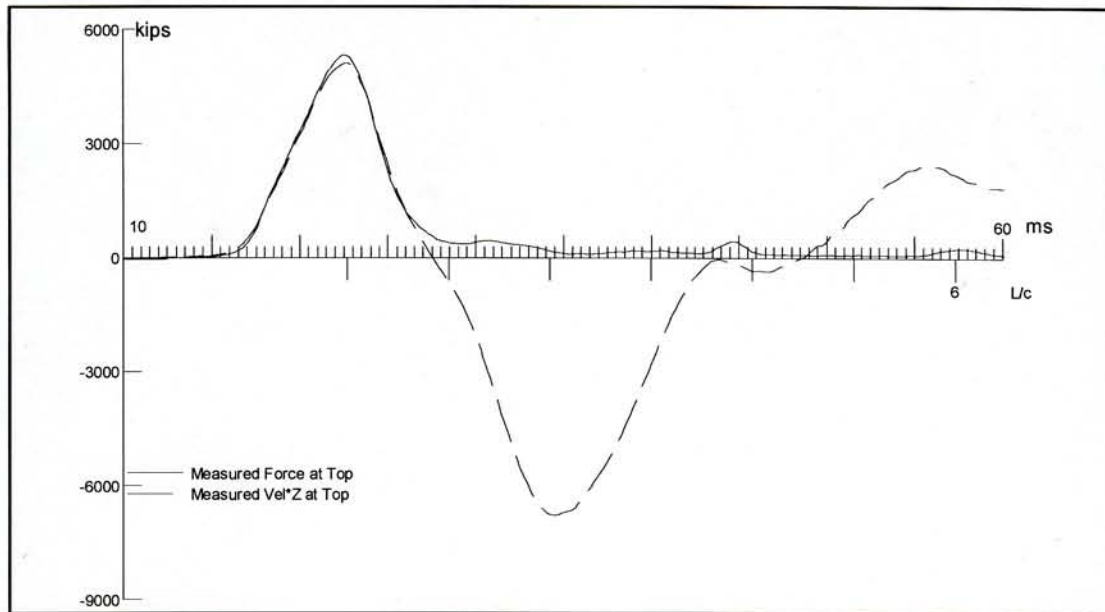


Figure 2. Force and velocity (times impedance). Records of a hard rock socket response

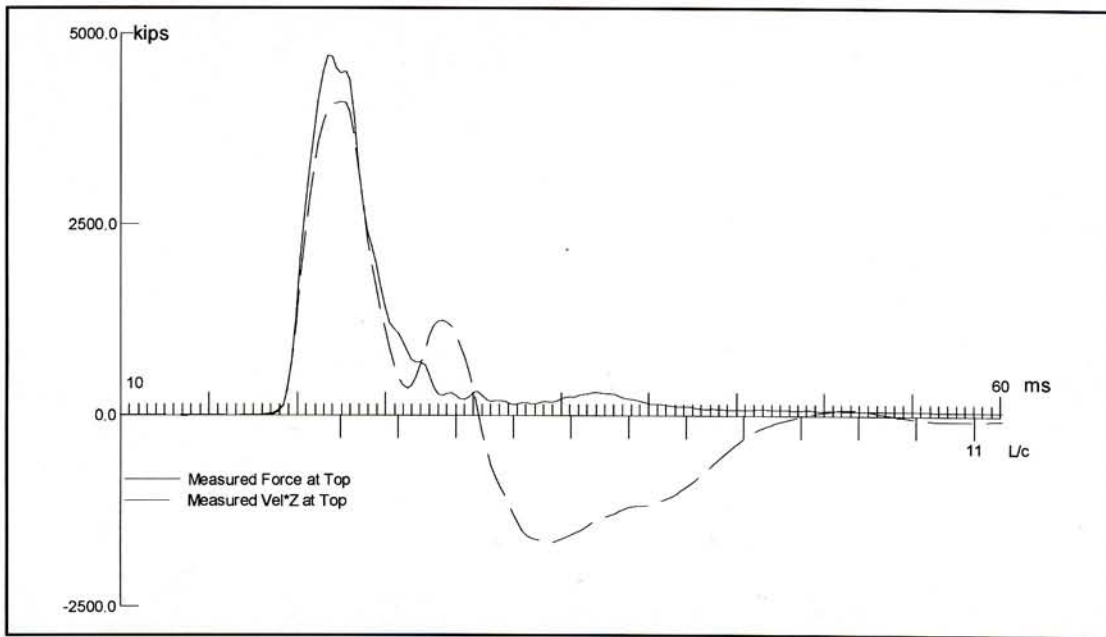


Figure 3. Force velocity records of a soft rock socket response

### Energy Transfer

Figure 4 shows the energy transfer ratio, i.e. the transferred energy measured at the sensor location divided by the potential energy of the hammer, vs. the ram weight to pile weight ratio. This information is interesting because it shows (a) that the ETR values can be rather small (between 9% and 64% with an average of 33%) and yet a high capacity can be activated and (b) that there seems to be an upper bound to the energy ratio: it is at most equal to the ram weight to pile weight ratio. Note that the ram drop was free in all cases and that similar plywood cushion thicknesses were employed. The main difference in testing was the number of applied blows which typically varied between 2 and 4, compressing the cushion to different degrees. The energy transfer is also affected by factors like ram-pile alignment, pile top condition, number of ram modules or ram stiffness, ram friction and even the resistance response of the shaft. Thus, while all of these factors tend to reduce the ETR, the ram weight to pile weight ratio seems to impose an additional limit that could be explained by the need for matching the impedances of ram and pile.

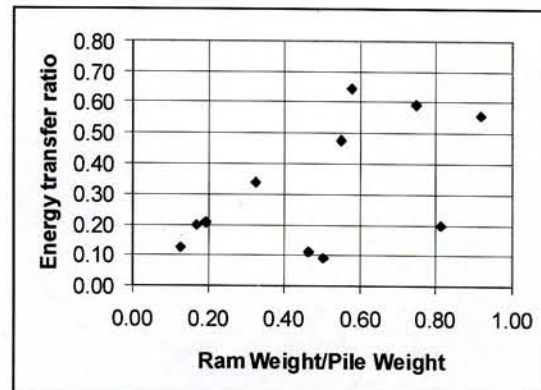


Figure 4. Measured energy transfer ratio vs. ram weight to pile weight ratio

### Activated Capacity

Theoretically, for a fixed shaft toe, the total resistance could be as high as two times the maximum pile top force and for that reason it is important to calculate the resistance force during testing so as to avoid an overstressing at the pile toe. Furthermore, it is desirable to activate as high a capacity with low stresses and therefore as low a pile top force as possible. In the examples presented, the highest ratio of activated static resistance to maximum pile top force reached almost 1.6 (Case 1-2). This ratio is primarily dependent on the stiffness of the rock. The lower the stiffness of the rock

resistance, the more displacement is needed to activate the resistance and this, in turn, reduces the activated capacity. In fact, high rock stiffness requires much lower energy for resistance activation than low stiffness rock or soil.

Obviously, the higher stiffness of the rock, the higher the rebound of the shaft, and that is expressed in the negative velocity response. It is therefore instructive to look at the ratio of the activated static resistance relative to the pile top force as a function of the positive to negative velocity peaks. The negative velocity peak is actually the rebound velocity of the shaft. Figure 5 is a plot of the ratio of activated resistance vs. pile top force as a function of the velocity peak ratio. The figure shows that there is a clear tendency: the two lowest velocity ratios were obtained for the minimum capacity case at site 4 and the soft claystone case at site 2. Thus, without calculation, the velocity response can be taken as a clear indication of the quality of the rock socket response. Of course, permanent set and, after analysis, the activated static resistance values are further indications of the rock socket's response.

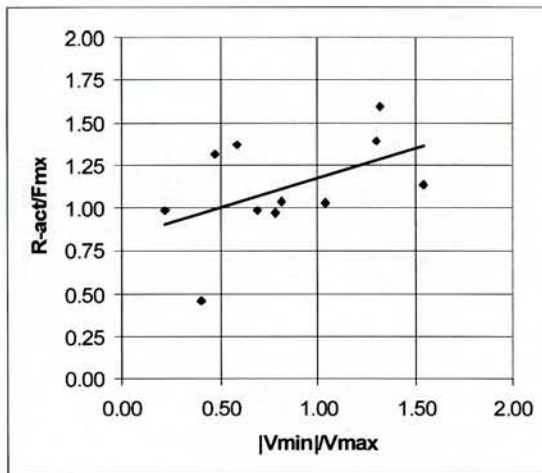


Figure 5. Relative activated rock resistance vs. relative rebound velocity

### Soil/Rock Damping

Another point of interest is the damping behavior of the soil and rock. Soil damping would be expected along the shafts of the piles in the soils, but to a lesser extent along a hard rock socket or the bottom of a shaft drilled into rock. Of course, a contamination of the rock interface

or shaft bottom by slurry deposits or debris could cause a viscous response. A major reason for the relatively high damping factors are the low shaft velocities generated by the dynamic test which means that damping is small regardless of the magnitude of the damping factor. Furthermore, several studies on soil samples (e.g. Coyle and Gibson, 1970) have shown that the damping response is not linear and that high damping factors must be expected when velocities are low. And finally, there is also the likelihood of some radiation damping (i.e. energy dissipation by the rock socket moving with the shaft) whose effect is covered by the damping factor in a standard CAPWAP analysis.

In any event, it would be expected that the damping behavior calculated by the signal matching procedure would be very different for the different shafts (a) because of the length of the shafts extending through softer geomaterials, (b) because of different construction methods (wet or dry) and (c) because of the different rock qualities represented by the example cases. Indeed, Figure 6 shows not much of a tendency, however, closer inspection reveals that the high capacity piles in hard limestone had damping to pile top force ratios less than 0.5 while claystones, Florida limestone and the low capacity Shaft 4-2 had the higher damping ratios. This shows that even at the same site, because of site or shaft variability, the same damping response cannot necessarily be expected. Indeed it is the strength of the stress wave signal matching analysis that it recognizes dynamic resistance and eliminates it from the total resistance to yield the static component.

### Simplified Methods

In the context of the damping behavior discussion, it would be of a great advantage to the test engineer if the amount of damping could be accurately estimated from soil/rock properties. Then a simplified closed form solution could be employed. For the examples shown here, which represent differing rock and shaft conditions, it appears as though the Case Method with a damping factor  $J = 0.4$  would give reasonable and generally conservative results with an average prediction that is 4% lower than CAPWAP. Unfortunately, the simplified method strictly applies only to uniform shafts and since the piles were sometimes highly non-uniform, owing to the use of telescoping casings, some

differences between the simplified and the more elaborate numerical method must be expected. Figure 7 shows the correlation between the two analysis methods.

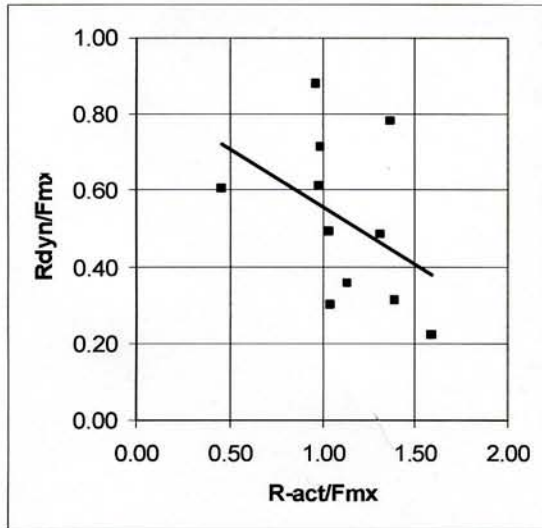


Figure 6. Dynamic vs. static resistance

The closed form approach can also be further expanded to include the so-called PEBWAP result that calculates, again under the assumption of a uniform shaft, the toe displacement as a function of time. Plotting the Case Method resistance under consideration of a damping factor ( $J = 0.4$  in the present case) vs. the calculated toe displacement and adding the shaft elastic deformation, yields a first estimate of the shaft load-set curve. Figure 8 shows for Cases 1-2 and 2-1 (the highest and lowest capacities cases) that this first estimate yields a reasonable agreement with the CAPWAP calculated result, which takes into consideration the non-uniformities and the shaft resistance effects that are not considered in the closed form solution.

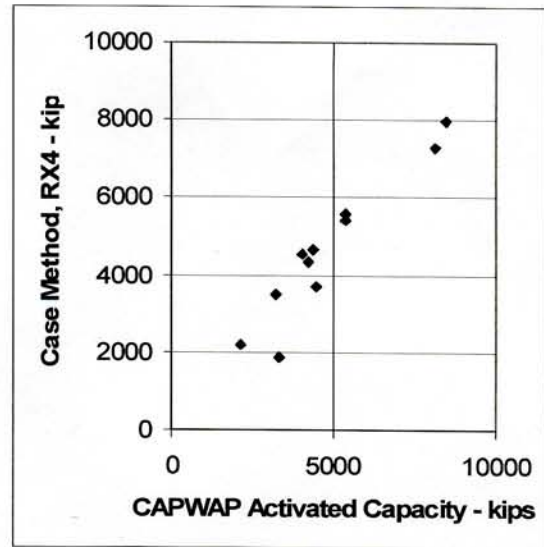


Figure 7. Correlation between simplified and signal matching capacity results

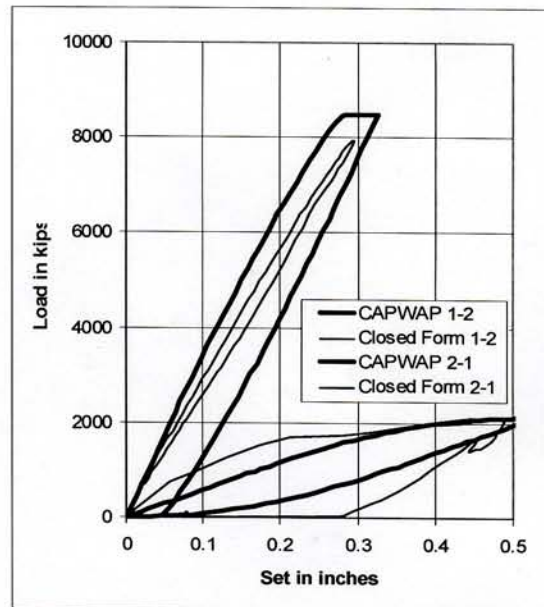


Figure 8. Calculated load set curves from signal matching and closed form solutions



## Summary and Recommendations

The results presented in this paper can be summarized as follows:

- Dynamic tests on rock-socketed shafts provide a quick and relatively inexpensive quality assurance method yielding information on rock stiffness and bearing capacity. The method is therefore particularly well suited for testing of production piles providing the information needed for the LRFD pile design approach.
- The examples demonstrated that the dynamic tests induced low shaft displacements and permanent sets which can be easily accommodated by the concrete rock interface without capacity degradation.
- Immediate testing results include forces and therefore stresses and an estimate of the load-set curve. Further refinement by numerical analysis is needed because of shaft non-uniformities and differences in rock response.
- The rule-of-thumb of a ram weight of at least 1% of test capacity has been shown to yield reasonable results. It is interesting that the transfer energy ratio generally is less than the ram weight-to-pile weight ratio; however, even very low energy transfer ratios are capable of activating high resistance values.
- A better measure for the limit of capacity activation, rather than transferred energy, is the maximum shaft top force. In the present examples the highest capacities activated reached 1.6 times the maximum pile top force.
- As expected, soil/rock damping was generally lower in the hard rock than in the softer rock material. However, it was still surprisingly high, probably because of radiation damping effects and/or low pile particle velocities. The similarity in CASE Method damping supports the use of the same damping factor for a first estimate of activated capacity in all types of rock.

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