# Dynamic Pile Testing at the Mesa A Rail Bridge

## Analyse dynamique d'essais de pieux au pont ferroviaire Mesa A

Cannon J.G.
*Independent Geoscience Pty Ltd, Melbourne, Australia*

ABSTRACT: The paper will describe high strain and low strain dynamic pile testing that was used to assess the foundations for this bridge and how the results led to an innovative decision during construction to change from a bored pile foundation to a driven pile foundation. The revised construction used minimal extra equipment and materials. The foundations originally comprised permanently cased bored piles socketed into rock. Dynamic pile testing identified significant problems with the concrete early in the project. A number of attempts were made to correct the problems but to no avail. In addition, the down-the-hole hammer system being used to install the casings was causing delays. The steel tube casings being used were substantial and a large hydraulic piling hammer was on site for dynamic testing. A decision was made to drive the casings into soft rock far enough to generate the required compression and tension resistance and to delete the concrete sockets. Dynamic pile testing was used to verify that the required resistances were achieved.

RÉSUMÉ : Cet article décrit l'analyse dynamique d'essais de pieux sous forte et sous faible déformation, effectuée pour évaluer les fondations du pont, et montre comment les résultats ont mené à une décision innovante pendant la construction, passant de fondations par pieux forés à des fondations par pieux foncés. Le changement de construction nécessita peu d'équipements et matériaux supplémentaires. Les fondations comprenaient à l'origine des pieux forés dans la roche maintenus de manière permanente par des conduites en béton. L'analyse dynamique des pieux mit en évidence un sérieux problème lié au béton dès le début du projet. Plusieurs tentatives pour le régler furent infructueuses. De plus, le système de marteau de fond de trou utilisé pour installer les pieux posait problème et créait des retards. Les conduites en acier utilisées étaiei
décision fut prise d'enfoncer les boitiers dans une roche meuble
nécessaires et de supprimer les conduites en en béton. L'analyse dyna
nécessaire était atteinte.

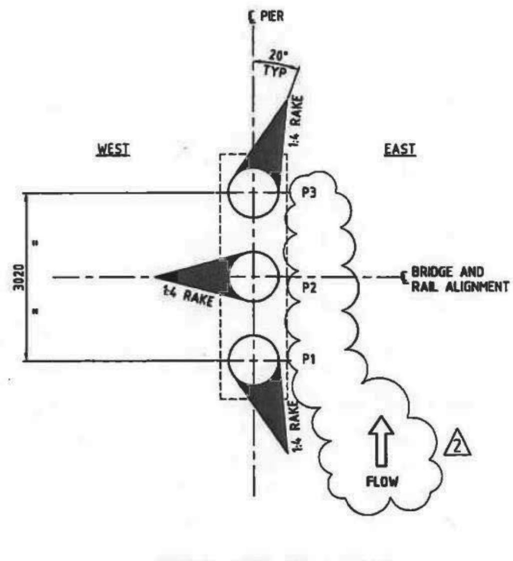
KEYWORDS: Dynamic pile testing, Bridge Foundations, PDA, CAP'

## 1 INTRODUCTION.

The Mesa A bridge is located over the Robe River in the Pilbara Region of Western Australia. Iron ore is mined from various Mesa, or low flat-topped mountains, in the area. The iron ore is carried directly from near the mine face by rail to a processing and export facility about 200km away. The trains are some of the longest in the world and have some of the highest axle loads in the world. At the bridge site the Robe River is dry on the surface for much of the year but up to 15m deep and fast-flowing during the "wet" cyclone season. Significant scour of the foundations was expected. High lateral loading from river debris was also anticipated.

During the "dry" construction season conditions are arduous for construction work and, in particular, concreting with daytime temperatures typically 38-42° C and within the construction zone in the low-lying riverbed they were frequently higher.

The Specification for the work included a test pile separate from the bridge works, which was to be tested statically in tension, together with testing for integrity using cross-hole sonic logging (CSL). The contractor offered high strain dynamic testing with a Pile Driving Analyzer® (PDA) as an alternative to the static test. The client required PDA testing at 40kHz, which required the newest PAX model PDA.




TYPICAL PIER PILE LAYOUT
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The Contractor submitted alternative pier foundations that comprised three raking piles in a tripod arrangement as shown below. (Figure 1). The tripod arrangement meant there were no redundancies and it was absolutely essential for every pile to perform as designed. Abutment foundations comprised four vertical piles. The size of the abutments meant there was the possibility of installing extra piles to augment any under-strength piles.

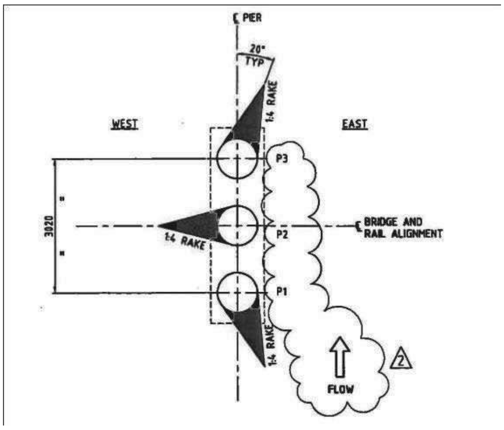


**Figure 1 - Pier foundation Layout**

The piles were 900mm OD with 20mm wall thickness steel tubes with a concrete socket in the rock. At this site the socket was 780mm diameter. The concrete was to be continued into the pile a sufficient distance to transfer the load by steel/concrete bond - about 8m. The concrete was to be heavily reinforced owing to high tension loading, with ten clusters of three 36mm diameter bars.

The contractor initially installed the casings with a large down-the-hole hammer. This used a 940mm hammer positioned at the bottom of the excavation and powered by compressed air. The hammer drags the casing behind it by engaging a collar at the bottom of the casing. Upon reaching rock the hammer can disengage the casing and bore a socket. At this site the hammer was removed after reaching the rock and a 780mm rock-roller drill was used to drill the socket. The method was well known to the contractor and is reputed to have excellent production rates in hard ground. However, at this site the gravels and cobbles were loose and saturated. There was frequently insufficient resistance to start the hammer and it stalled frequently. Progress was slow and delays were extensive.

Owing to the remote site concrete was produced by a mobile batch plant at the site but was otherwise unremarkable. The piles were concreted using a tremie pipe with the shaft full of water or drilling fluid.

## 4 SOCKETED TEST PILES

The first pile constructed at the site was a test pile that was not to be incorporated into the bridge. It was a vertical full sized pile. To provide for dynamic testing the socket concrete was extended to the top of the pile. The test gauges were initially attached to the concrete through "windows" in the steel tube and the pile was considered as a combined section. The top of the steel tube was cut back 25mm below the top of concrete to ensure the hammer acted on the concrete. A 19mm plywood cushion was used between the hammer and the pile.

Other than the permanent casing there was nothing unusual about the dynamic testing. Concreting had been completed about 14days prior to the test.

Very strange measurements were obtained initially, with almost no force measured in the concrete. (Figure 2). In addition it was observed after several blows that the concrete in the "window" had moved down relative to the steel.

The gauges were moved to the steel tube and again very strange measurements were obtained The only way the measurements made sense was to use the properties of the steel tube only and assume the concrete was not working as part of the pile. (Figure 3). The hammer was removed and it was noted the top was level with the steel. The hammer was indeed acting on the steel tube.

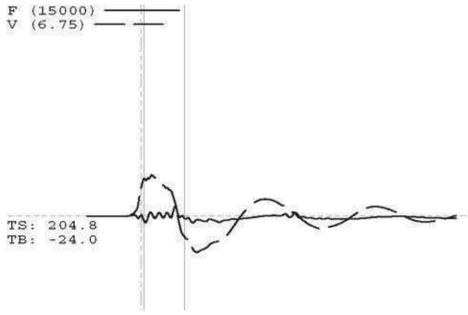


**Figure 2 - PDA measurements on concrete**

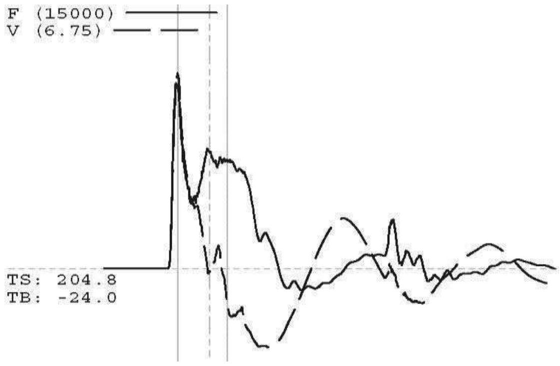


**Figure 3 PDA measurements on steel**

The concrete between the pile top and the gauges must have crushed by up to 44mm, being the 25mm upstand of concrete plus the thickness of the pile cushion. The hammer was replaced, with increased cushion, and more blows were applied with no improvement of the measurements and with further crushing of the concrete.

A second test pile was constructed and this time access tubes for CSL testing were cast in the pile. Concreting records showed potential problems near the toe and part way though the pour. Observations suggested the concrete had started to set early, possibly due to the extreme temperatures. CSL test results confirmed problems at the toe and between 15-20m below the pile top. (Figure 4).

Dynamic pile testing was also conducted and confirmed concrete problems. Gauges were attached to the concrete through windows in the steel casing. Results suggested there was a significant defect 15-20m down the shaft. The steel was also not contributing to the pile. Transfer of load by steel to concrete bond seemed unreliable. (Figure 5).

## 5 PRODUCTION SOCKETED PILES

As it was thought the cause of the concrete problems was known, and related to temperature and the behaviour of concrete additives, it was decided to proceed with production piles, but only at abutments, where repairs such as additional piles were possible.

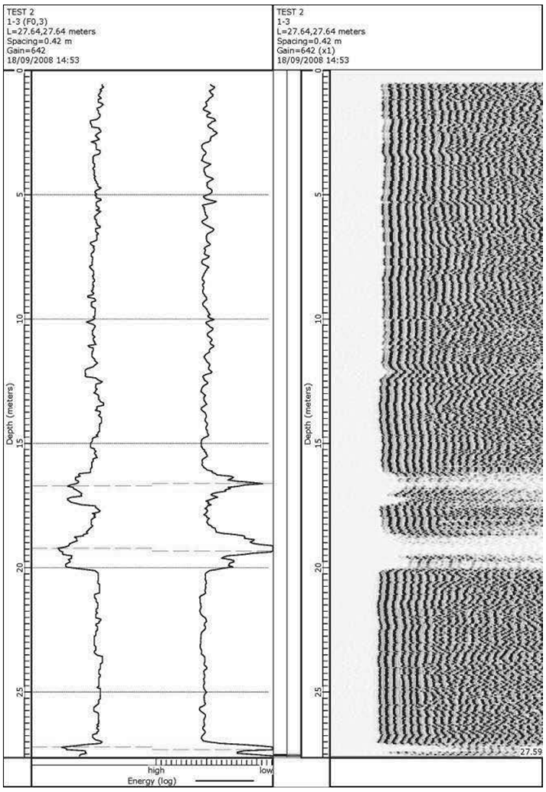

**Figure 4 - CSL test results Test Pile 2**

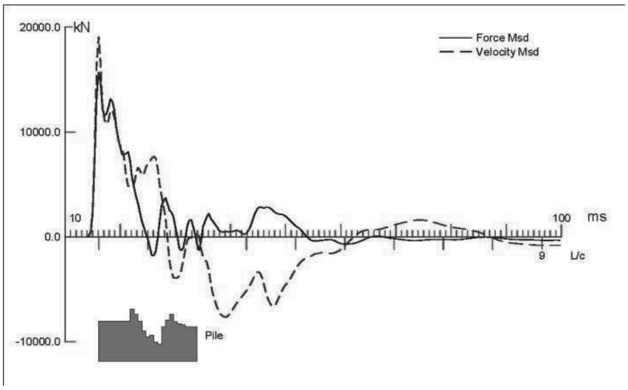


**Figure 5 - PDA measurements Test Pile 2**

At the same time the author was requested to conduct driveability predictions using wave equation analysis of piles (WEAP) to assess if the casings could be used as driven piles. One of the main criteria of interest was whether the piles could be driven far enough into the soft rock to achieve the required tension resistance, which was a controlling aspect of the tripod foundation design. This appeared to be the case, and with a smaller hammer than was being used for the dynamic testing.

Construction started on the East Abutment and, again, problems with the concrete quickly became obvious. CSL testing indicated a significant defect at 25m below the top. (Figure 6). Drill cores were conducted to investigate and repair the problem zone. Coring confirmed a zone of aggregate only with no cement - exactly as predicted by the CSL. (Figure 7).

Further attempts were made to improve the concrete but in every case problems were detected. If anything the concrete problems got worse, both in size and significance. (Figure 8).

Prior to the abandonment of socketed piles one further test was conducted on the last socketed abutment pile. No access tubes had been cast in this pile so low strain axial sonic pile integrity testing using a Pile Integrity Tester® (PIT) was conducted (Figure 9). No integrity problems were detected

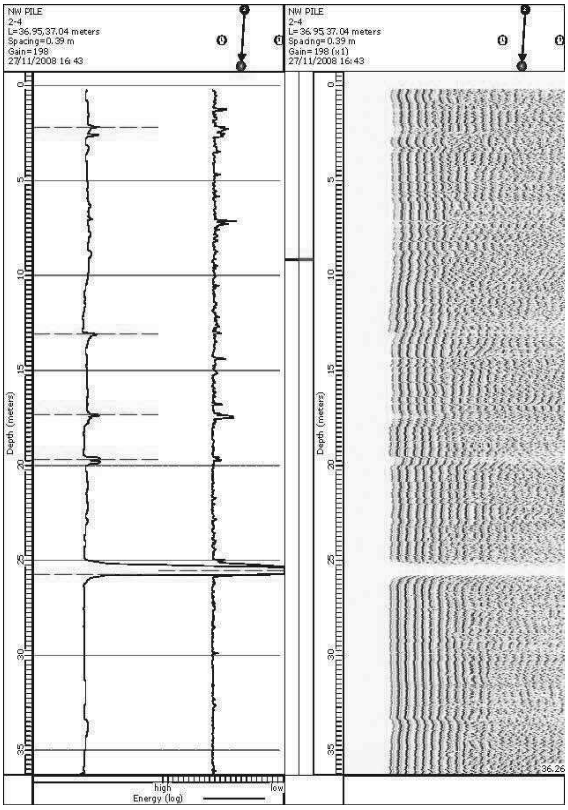

**Figure 6 CSL test result East Abutment NW pile**

**Figure 7 - Core of East Abutment NW pile**

using this method. A very slight increase in velocity was detected at the top of the socket but this was attributed to the change in diameter from 900mm to 780mm.

## 6 DRIVEN TEST PILE

Soon after the WEAP analyses were conducted a third test pile was installed. Again, this was positioned adjacent to the bridge location and was not to be incorporated into the contract works. The 900x20mm steel tube casing was modified by removing the bottom collar and installing a 900x50mm driving shoe of 0.8m length. This third test pile was driven with a Junttan HHK9A hammer, which has a 9t ram. The pile was driven through the gravels and cobbles and 12m into the soft rock at a final set of 2.9mm/bl. A restrike test was conducted the next day using the Junttan HHK16A hammer, which has a 16t ram. The pile had "set-up" to such an extent the set had reduced to 1.3mm/bl despite the hammer being significantly larger than the hammer used to drive the pile. The resistance demonstrated during the

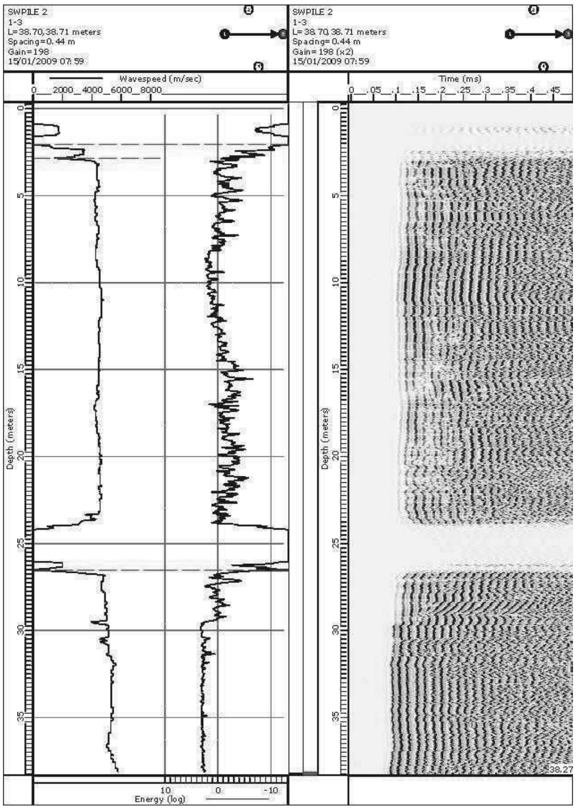


Figure 8 CSL result on production pile

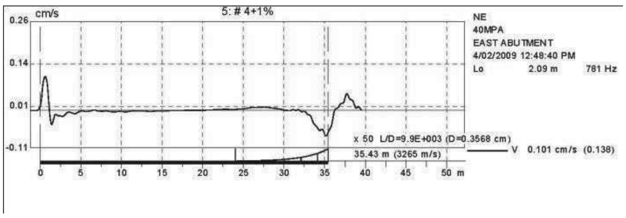


Figure 9 PIT results for East Abutment NW pile

restrike was adequate for all pile positions - upstream, downstream and longitudinal. (Figure 10).

The skin resistance distribution shown in the CAPWAP ® appears to reduce toward the toe because at such small sets not much of the energy of the input blow reached the lower part of the pile and the reduced energy did not strain the ground enough to generate the full resistance that was available. The available resistance probably does not reduce. If the pile could have been struck harder, then even more resistance would have been demonstrated.

## 7. DRIVEN PRODUCTION PILES

It was considered the driven pile created a more reliable foundation and repairs could be easily accomplished by re-driving if necessary. A decision was made to change the foundations for all piers from bored piles into driven piles. In addition repairs for the abutment piles were also to be driven piles.

The effect of this decision was to dramatically improve production rates on the pile foundations. Because the concrete was eliminated from the pile there were also considerable savings over the original design.

Owing to the numerous problems experienced during initial piling and the tripod design that provided no redundancy, the client required that all driven piles undergo PDA testing. Production testing was conducted remotely or by "stand-alone." (Likins, Hermansson, Kightley, Cannon and Klingberg 2009)

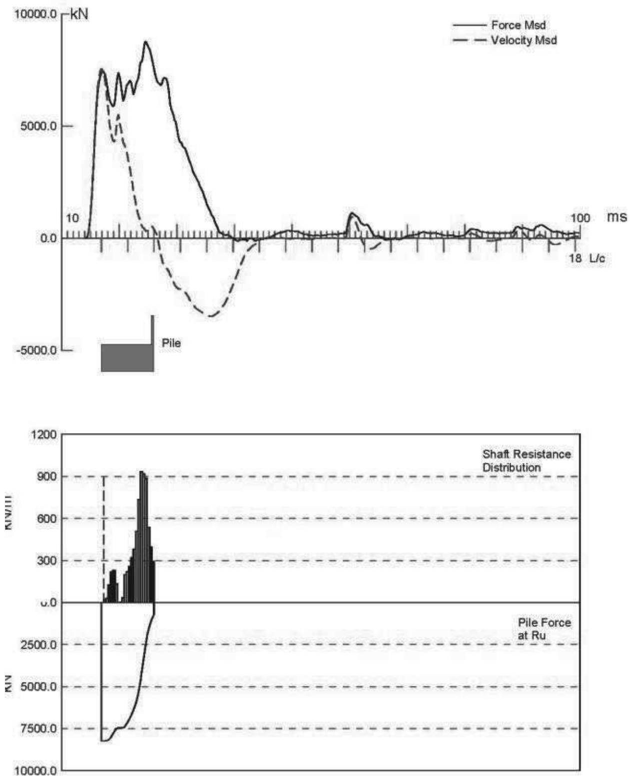


Figure 10 CAPWAP results for Test Pile 3

The shorter piles at the western end of the bridge could be tested with the smaller 9ton hammer, however from almost exactly the centre of the bridge it was necessary to use the larger 16t hammer to demonstrate the required resistance. In some cases it was necessary to wait for "set-up" and conduct restrikes at 1-7 days after driving.

Tension resistance, particularly on upstream piles, was of critical importance. In some cases the resistance demonstrated by CAPWAP appeared to be heavily concentrated near the toe and owing to concern about the ability of CAPWAP to accurately differentiate between skin friction near the toe and toe resistance the author was requested to provide an estimate of the "minimum likely" skin friction. The method adopted was purely arbitrary and comprised starting with "best match" and moving skin friction to toe resistance until the match quality increased by one percent, ie CAPWAP match quality increased from 3.1 to 4.1 percent error.

## 8. CONCLUSIONS

Low strain and high strain dynamic pile testing was incorporated into both the design revision and construction verification aspects of this successful project.

During construction both high strain and low strain dynamic testing demonstrated serious problems with the "constructability" of the original bored pile design.

An innovative decision was made to radically change the pile design, from a bored pile to a driven steel tube pile.

Dynamic pile testing was able to confirm pile resistance and provide a high level of confidence in the foundation. The bridge has been working as designed through 3 cyclone seasons carrying some of the heaviest train axle loadings in the world.

## 9. REFERENCES

Likins, Hermansson, Kightley, Cannon and Klingberg 2009 Advances in Dynamic Foundation Testing Technology IFCEE Orlando, Florida USA