METHODOLOGY TO DEMONSTRATE PILE CAPACITY IN RELAXING GROUND

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Abstract: Driven pre-stressed concrete piles have been used as a foundation system to support abutments and piers of a bridge constructed near Ballina in New South Wales, Australia. In order to achieve the required geotechnical capacity, the piles were required to be driven through soft clay and sand to moderately weathered rock. Pile Driving Analyser (PDA) testing together with CAPWAP analysis was performed to assess the integrity and geotechnical capacity of the driven piles. Pile damage was observed during driving. To prevent damage a rock shoe was retrofitted to the piles prior to installation. Reductions in pile capacity (or relaxation) were observed between end of drive (EOD) and later restrike testing (RST). A substantial amount of additional pile testing was performed at different times after driving to assess the changes in pile capacity over time. Most piles were re-driven to achieve higher capacity. Pile capacity could not be achieved in one pier and additional piles were installed to reduce the required pile test load

Keywords: Water Conservation, Rainwater, Harvesting

1. Introduction

Twin bridges have been constructed as part of the Ballina Bypass Project. The bridges extend over a length of approximately 133m (between centrelines of abutments). Each bridge has four spans of approximately 33m length with a carriageway width of about 10.5m. The bridge alignment is generally orientated on a north-south bearing over estuarine and alluvial soil deposits.

The adopted foundation system for the bridges comprised 550mm octagonal pre-stressed concrete piles. One row of 7 piles was adopted for the abutments and two rows with 5 piles in each row in a staggered configuration were adopted for the piers. In order to achieve the required geotechnical capacity, piles were driven into moderately weathered bedrock.

Some of the piles were damaged during driving. The cause of damage and method used to avoid damage in piles is discussed in the following sections.

In addition, reductions in pile capacity (or relaxation) were observed between PDA end of drive (EOD) and restrike testing (RST) for many of the piles. Data showing relaxation with time are presented and potential mechanisms for relaxation discussed. Methods adopted to overcome pile relaxation and testing performed to meet acceptance criteria are discussed.

2. Geological conditions and Model

Subsurface ground conditions have been assessed using borehole drilling and cone penetration testing. The investigation results indicate that the twin bridges are underlain by Holocene age estuarine clay deposits overlying Pleistocene age sand, stiff clay, residual soils and weathered Argillite (interbedded metasiltstone / metasandstone).

The underlying Argillite is a fine grained interbedded metasiltstone and metasandstone. The Argillite can be divided into three sub-units; decomposed clays, highly to moderately weathered Argillite and slightly weathered to fresh Argillite. A long section showing the ground conditions is presented in Figure 1.

The bridge is oriented in a north-south direction. To the west of the bridge lies a hillside that dips steeply toward the bridge location and continues to dip from west to east beneath the bridge location.

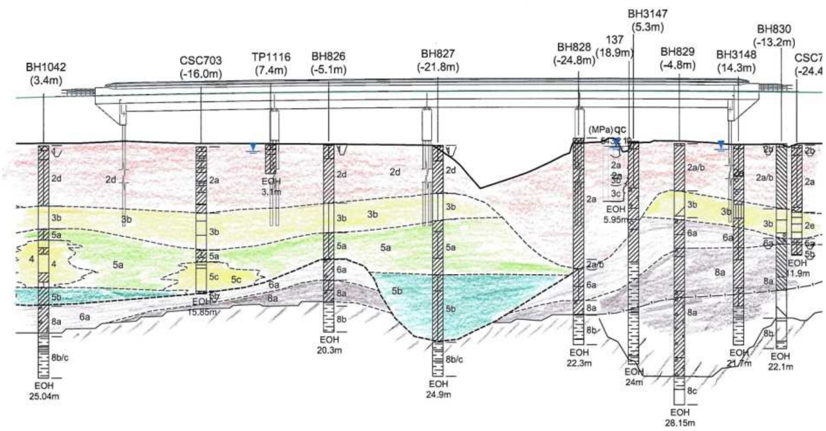

Figure 1: *Geological long section*

The geotechnical model developed for pile design for Pier 2 (northbound) is presented in Table 1 to provide an indication of the soil and rock properties. The geotechnical model shows that the majority of the pile resistance comes from end bearing.

Table 1: *Geotechnical model for pile design*

Soil unit	Reduced Level at Top of Unit (m)	Description	Undrained Shear Strength, c_u (kPa)	Ultimate Skin Friction, f_s (MPa)	Ultimate Base Resistance f_b (MPa)
Unit 2d	0.4	Very soft high plasticity silty clay.	10	0.01	-
Unit 3b	-5.1	Loose silty sand.	-	0.02	-
Unit 5a	-8.1	Firm clay	25	0.03	-
Unit 5b	-14.1	Stiff to hard clay	150	0.06	-
Unit 8b	-18.1	Highly to medium weathered Argillite	-	0.5	15

3. Pile Type

The foundation system for each support comprised 550mm octagonal driven pre-stressed concrete piles with a precast conical driving tip. Figure 2 shows the geometry of the pile.

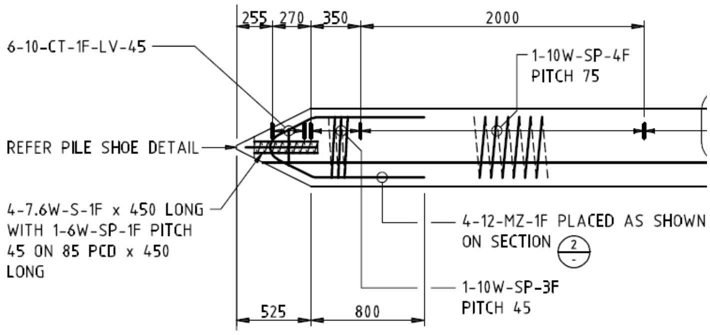

Figure 2: *Precast Octagonal Pile Details*

4. Hammer details

Due to the close proximity of the bridge piers to the creek edge a conventional piling rig was unable to be utilised due to potential instability within the near surface soft ground. For the purpose of positioning heavy plant away from the creek edge the piles were driven using a free hammer attached to a modified 50t crawler crane.

The hammer used for this operation was a 6t hydraulic hammer. It applied up to 55kJ of energy with a 1m drop. Sets measured manually varied from 3mm/blow to 0.5mm/blow at end of drive (EOD).

5. Testing methodology

Project specific QA specifications cited that 10% of piles (at least 1 test per pier or abutment) should be dynamically tested at EOD and upon re-strike (RST) with CAPWAP signal matching analysis. RST testing is performed a minimum of 12 hours post installation. These piles are termed as representative piles and are driven prior to the remainder of the pile group. The remaining production piles were then installed to the driving parameters set by the representative pile.

In addition to the representative pile testing an additional 5% of the remaining production piles were required to be dynamically tested on RST with CAPWAP signal matching analysis.

Acceptance criteria for pile test loads (or pile capacity) are also stipulated in the project specifications that “the acceptance criteria for a restrike test on a pile are that the driven parameters achieved must be equal or better than those measured at the end of driving and the distribution of resistance along the pile must be effectively unchanged”. According to the specification, piles with lower capacity at RST than EOD cannot be accepted.

Australian Standard AS2159 allows the geotechnical strength reduction factor (ϕ_g) to vary with the amount of testing performed. Based on the above quantum of testing ϕ_g=0.8 was adopted for the heaviest loaded piles in the group and ϕ_g=0.7 for the remainder of the piles in the group. The maximum pile test load to be achieved was 3966kN and all piles were intended to be driven to achieve this capacity.

6. Pile damage and integrity testing

Representative piles S2-1 (i.e. Pile No. 1 of Pier S2) and S2-10 were installed and PDA tested at EOD. The PDA equipment calculates an integrity factor called BETA. This is based on the change of impedance from one section of the pile to another and is an indication of the reduction in cross sectional area of the pile (Webster, 1996). The PDA data showed that the piles had BETA values of 57% and 78% respectively. The BETA value alone was not the sole indication of pile integrity but was rather used as a guide together with reviewing the force-velocity curve that a pile may be undergoing bending and subsequent loss of integrity at a given point during the driving process. These BETA values together with a review of PDA data indicated that the piles were broken and damaged respectively. PDA testing on these piles was applied after the pile had been driven some

distance into the founding stratum and so the stage of driving where damage initiated was not identifiable.
For the remainder of the piles PDA testing was initiated early in the drive whilst the pile was still being driven through the upper weak soil layers. This was undertaken to allow real time monitoring of the pile integrity and inferred capacity as the pile toe penetrated the founding stratum. For each pile the force-velocity curve of the dynamic impact was viewed for each blow and an assessment of the pile integrity was made as the pile was being installed.
Additional boreholes were drilled at Pier 2 to further investigate the ground conditions and to gain understanding of the bedrock conditions across Pier 2. The subsurface profile was identified to generally consist of weak upper soil layers followed by a rapid transition into high strength bedrock.
Following a review of the borehole information and the PDA data on initial S2 piles it was assessed that these piles may have been damaged as a result of a tendency for the conical driving shoe to slip down the surface of the bedrock prior to penetrating the founding stratum. Bending is likely to occur in the pile between the free tip and the pile being held within a guide frame at ground surface level. It was assessed that there was a high level of risk of piles in Pier N2 and S2 groups being subjected to this mechanism and remedial action would be necessary to enable the pile to drive vertically into the founding stratum.
The original conical driving shoe was not suitable to allow the pile to vertically penetrate such a distinct bedrock surface. A steel rock driving shoe was introduced which would encase the pile toe and provide a pile base surface that would minimise the opportunity for the pile toe to deviate from its position caused by following the contours of the bedrock.
The conical toe of all piles thereafter was cut off to suit the expected founding level. The purpose of this was to remove any unnecessary length of pile that may "whip" during driving caused by the effect of the hammer impact and the low resistance in the weak upper soil layers. It was not clear that pile whip was affecting the pile capacity but the piles were cut as short as possible to minimise potential whip effects. The piles were then subsequently retrofitted with a pre-fabricated steel rock driving shoe which comprised of a steel sleeve and a stiffened steel end plate fitted to the end of the pile using epoxy resin. Figures 3 and 4 show section and end detail respectively of the modified rock driving shoe.

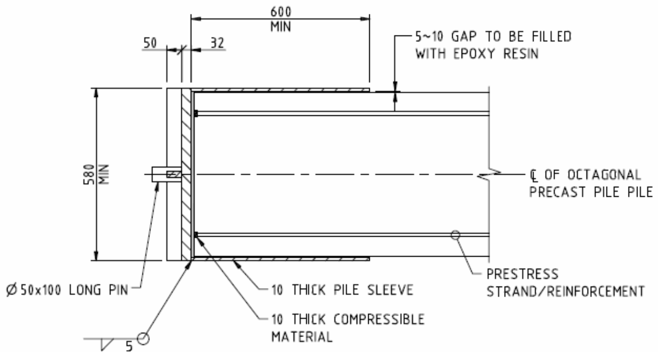

Figure 3: *Rock Driving Shoe Section*

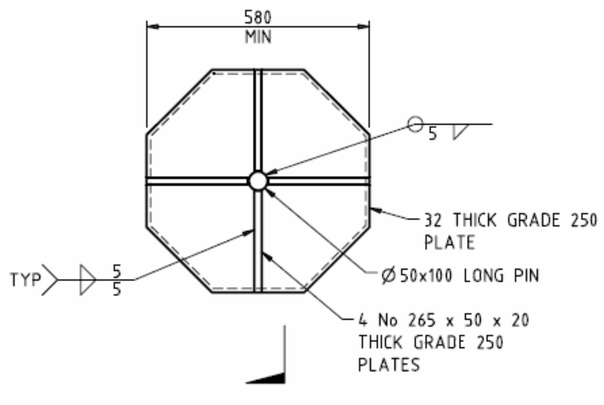

Figure 4: *Rock Driving Shoe Details*

Review of PDA test data during installation and subsequent review of CAPWAP signal matching analysis showed that the integrity of the modified piles was good with BETA values of 100% on all tests. Following adoption of the revised rock driving shoe no loss of integrity was observed in any of the piles.

7. Pile Relaxation

Observations of Relaxation

Pier 3 piles were the first piles to be installed at the bridge and it was at this location that pile relaxation was identified. These piles did not have problems with integrity during driving and were installed using the conical tip. Pier 3 piles were originally driven to just beyond the pile test load and upon RST testing were found to have relaxed below the test load. Piles were then subjected to additional driving to significantly higher capacities in excess of the pile test load to account for the extent of relaxation.

Given the experiences of pile capacity relaxation at Pier 3 the approach for driving Pier 2 piles was adopted such that piles would be over driven to account for the extent of relaxation. In the northbound Pier, piles N2-1, N2-3 and N2-5 were selected for extensive PDA testing over time since their locations span the Pier N2 pile group and provide a reasonable representation of the relaxation behaviour. These piles were driven to final EOD capacities between 4,490kN and 4,670kN. RST testing was performed over a period of up to 63 days which showed that the piles had relaxed then stabilised in capacity. Results of the testing are shown in Fiure 5. The maximum measured relaxation was 840kN. CAPWAP analysis was performed on selected tests and the analyses demonstrated similar capacity to PDA estimates. The results of testing show that relaxation did occur however the final pile capacities were in excess of the individual pile test load. Once it was assessed that piles were relaxing, the piles were driven to achieve a PDA field estimated capacity in excess of the required pile test load. In some cases piles were re-driven up to 1m to achieve the desired PDA inferred capacity which was up to 1500kN above the individual pile test load. Samson and Authier (1986) report a case where piles were re-driven up to 0.3m at RST to regain the required pile resistance.

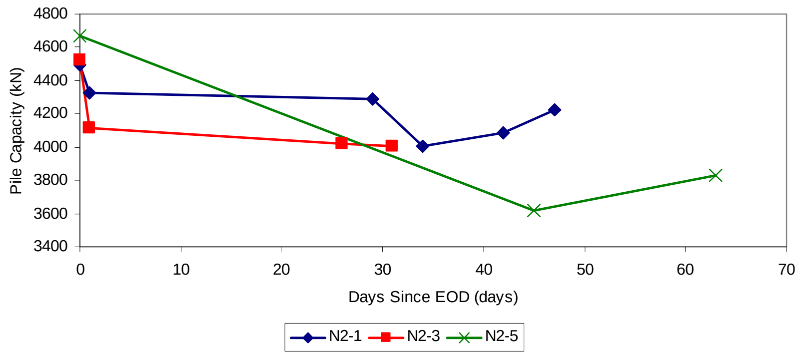

Figure 5: *N2 Relaxation Behaviour*

In the southbound pier, piles S2-5 and S2-10 were selected for extensive testing as they had the lowest PDA capacities at EOD. Piles S2-3, S2-6 and S2-8 were also selected for additional PDA testing over time due to their locations along the pier group and thus providing a reasonable representation of the relaxation behaviour. CAPWAP analysis was performed on selected tests and the analyses demonstrated similar capacity to PDA estimates. The results of testing show that relaxation occurred and that the final pile capacities were in excess of the individual pile test load. Figure 6 illustrates the pile relaxation behaviour of Pier S2 piles and demonstrates the trend for stabilisation of relaxation with time.

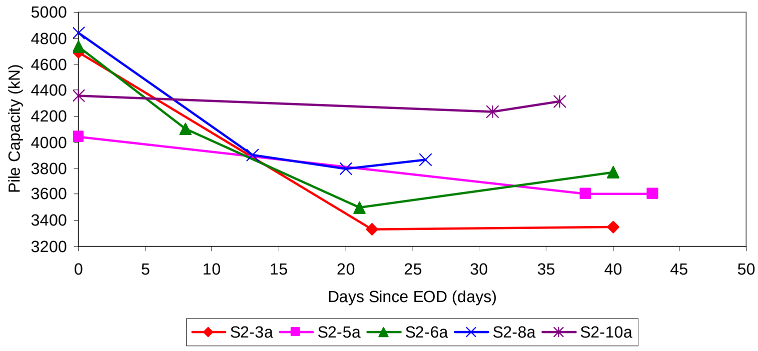

Figure 6: *S2 Relaxation Behaviour*

The available PDA data shows relaxation occurring over approximately 20-30 days before stabilising. The CAPWAP analyses for tested piles suggested that relaxation occurred on the shaft capacity and conversely the base capacity increased with time as shown in Figure 7a and Figure 7b respectively.

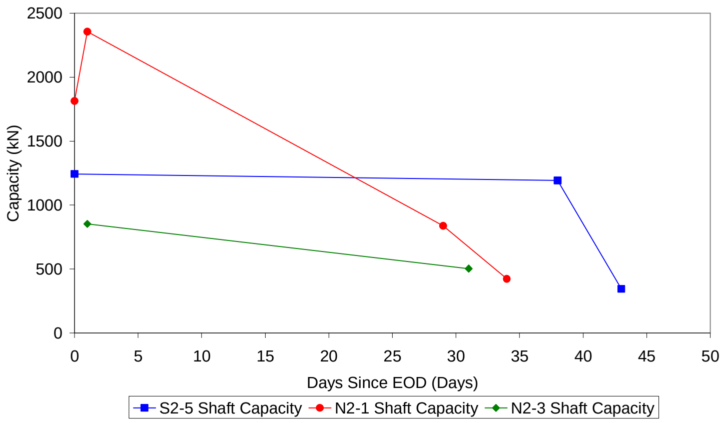

Figure 7a: *Pier 2 Shaft Capacity Change with Time*

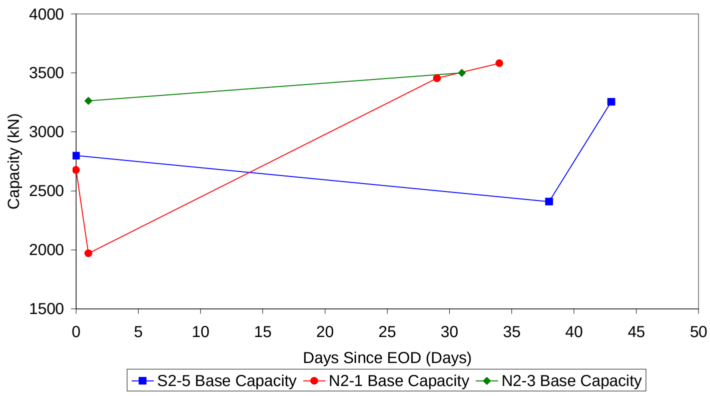

Figure 7b: *Pier 2 Base Capacity Change with Time*

Potential Reasons for Relaxation

An assessment of reductions in pile capacity between PDA EOD and 1st RST testing of pier piles is presented in Table 2. Table 2 illustrates similar relaxation range (as a percentage) for all pier piles. Given that the conical shoe was used in Pier 3 and the rock shoe used in the other piers it appears that the type of driving shoe did not affect pile relaxation.

Moller and Bergdahl (1981) suggested that during driving displacement piles into very dense sand dilation causes negative pore pressures are developed giving a temporary increase in effective stresse causing a temporary increase in capacity and that the effect of relaxation occurs due to the reduction in the effective stress with time as the pore pressures dissipate (York et al. 1994). The founding bedrock at ECC Bridge consists of a fine grained metasandstone and it may be possible that as the piles were driven into the bedrock the material has fractured to some degree and under this change in rock structure development of negative pore pressures may have occurred.

Table 2: *Reductions in Pile Capacity Between PDA EOD and 1st RTS Testing*

Pier	EOD capacity	1st RST capacity	Relaxation (%)
1	4075kN to 5142kN	3000kN to 4500kN	9 to 30 (average of 20)
2	4057kN to 5157kN	3066kN to 4920kN	4.6 to 26 (average of 13)
3	4125kN to 4821kN	2959kN to 4316kN	1 to 32 (average of 14)

Samson and Authier (1986) presented two case studies where reduction in capacity of toe bearing piles on shale bedrock has been observed. These piles were driven through soft strata into hard rock which is a similar stratigraphy to that encountered at this bridge.
Low shaft friction and high end bearing may be ground conditions that favour pile relaxation.

8. Pile group design revision

The 2 representative piles on Pier S2 were disregarded from the pier group after it was confirmed that the piles were damaged beyond acceptable contribution to the group. Consequently a new pile layout was adopted by flipping the original layout plan to enable installation of 10 piles (S2-1 to S2-10) into new locations whilst not being obstructed by the 2 disregarded piles. The new configuration of Pier S2 is shown in Figure 8.

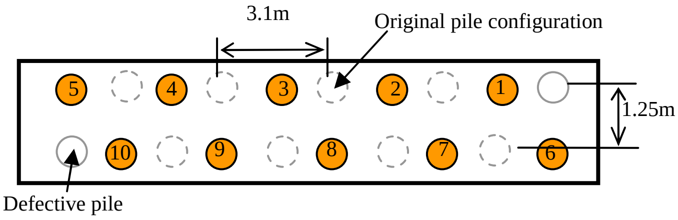

Figure 8: *Revised Pier S2 Pile Configuration*

Relaxation made achieving the original pile test load difficult. Continually redriving and retesting the piles was taking time and costing money. There was a high risk that the piles would not achieve the required pile test load and as such an additional two piles were incorporated into each pile group to reduce the individual pile loads. The final pile configurations for Pier S2 and Pier N2 are as shown in Figure 9a and Figure 9b respectively.

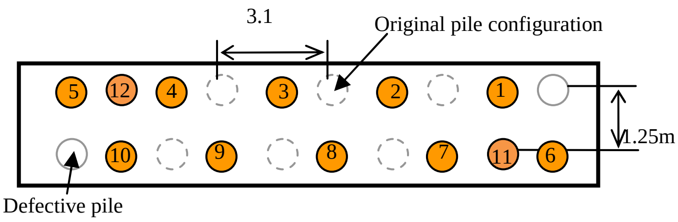

Figure 9a: *Revised Pier S2 Pile Configuration with 12 Piles*

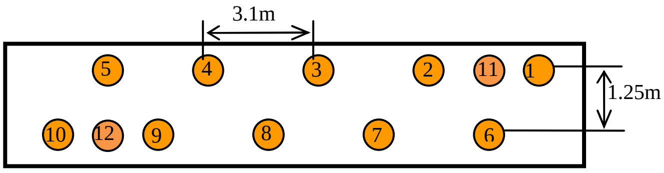

Figure 9b: *Revised Pier N2 Pile Configuration with 12 Piles*

Foundation designs for the revised pile group configurations were carried out. Table 3 shows the revised ultimate limit state loads and the associated individual pile test loads (i.e. maximum ultimate limit state load/appropriate geotechnical reduction factor).

Table 3: *Revised Pile Test Loads for Pier 2*

Pier N2			Pier S2		
Pile No.	ULS Load (kN)	Pile Test Load (kN)	Pile No.	ULS Load (kN)	Pile Test Load (kN)
N2-1	2800	3500	S2-1	2850	3563
N2-2	2330	3329	S2-2	2580	3225
N2-3	2230	3186	S2-3	2320	2900
N2-4	2080	2971	S2-4	2350	3357
N2-5	2420	3457	S2-5	2740	3425
N2-6	2420	3457	S2-6	2740	3425
N2-7	2080	2971	S2-7	2350	3357
N2-8	2230	3186	S2-8	2320	3314
N2-9	2330	3329	S2-9	2580	3225
N2-10	2800	3500	S2-10	2850	3563
N2-11	2420	3457	S2-11	2850	3563
N2-12	2420	3457	S2-12	2850	3563

9. Acceptance of pile capacity

Pile Capacity Acceptance Procedure

The procedure for accepting piles was developed according to the following sequence:

i) Revised pile test loads were allocated to each individual pile based on the pile group analysis. Individual pile test loads are shown in Table 1;

ii) Selected piles were extensively RST tested over a period of time to establish the extent of relaxation, that relaxation ceases over time and to show that the pile capacity at the lower bound of relaxation is above the individual pile test load;

iii) Spot checks were performed by RST testing other selected piles to demonstrate the pile capacity is in excess of the individual pile test load;

iv) For piles without RST testing the lower bound pile capacity has been represented by nearby piles which have similar EOD capacities to those which have been extensively RST tested.

Shown in Table 4 is a summary of the developed categorization of piles according to the acceptance criteria listed above.

Table 4: *Categorization of Pier 2 Piles with respect to Acceptance Criteria*

Category	Description	Pier N2	Pier S2
A	Piles subjected to RST testing which demonstrate stabilisation of relaxation above the individual pile test load	N2-1 N2-3 N2-5	S2-3 S2-5 S2-6 S2-8 S2-10
B	Piles which were subjected to RST testing a similar time after EOD to those in Category A to demonstrate pile capacity in relation to individual pile test load	N2-4 N2-8 N2-9 N2-12	S2-1 S2-2 S2-9 S2-11
C	Piles which were only subjected to EOD testing at final drive of which are represented by nearby piles in Category A	N2-2 N2-6 N2-7 N2-10 N2-11	S2-4 S2-7 S2-12

Northbound Pier (N2) Piles

Piles N2-4, N2-8, N2-9 and N2-12 were selected for RST testing between 24 and 63 days post installation. The piles were driven to final EOD capacities of between 4,530kN and 4,860kN. Results of the RST tests show that the piles relaxed a maximum 830kN to level in excess of their individual pile test loads. The amount of relaxation on these piles was shown to be less than the maximum relaxation demonstrated by Category A piles. These piles were not subjected to ongoing RST testing but demonstrated that over time the extent of relaxation is similar to Category A piles.

The remaining piles in the group were subjected to EOD PDA testing only. These piles were driven to EOD capacities of between 4,570kN to 4,770kN and were considered to be represented by the nearby Category A piles which were driven to a similar yet lower EOD capacity. The extent of relaxation shown by piles in Category A demonstrated that with similar relaxation the final capacities of the above piles were in excess of their individual pile test load.

Southbound pier (S2) piles

Piles S2-1, S2-2, S2-9 and S2-11 were selected for RST testing between 8 and 46 days since installation. The piles were driven to final EOD capacities of between 4,700kN and 4,830kN. Results of the RST tests show that the piles relaxed a maximum of 1,100kN to levels in excess of their individual pile test loads. The RST tests performed on these piles showed that the amount of relaxation was less than the maximum relaxation demonstrated by Category A piles, their behaviour over time was similar to Category A piles and that their capacities were greater than the individual pile test loads.

The remaining piles in the group were subjected to EOD PDA testing only and were driven to EOD capacities of between 4,850kN and 4,910kN. These piles were represented by the nearby Category A pile S2-8 which was driven to a similar yet lower EOD capacity of 4,840kN. Findings of the extensive testing on pile S2-8 showed that the relaxation has stabilised at a level above the individual pile test load for these 3 piles.

12. Concluding remarks

Piles were driven through challenging ground conditions that created problems with pile relaxation and attainment of pile capacity. Piles with precast conical driving shoes are not suited to being driven through low strength soil deposits onto a high strength sloping rock surface. Under these conditions the retrofitted ribbed flat plate driving show was found to allow the pile to be installed without being damaged.

Ground conditions where limited shaft capacity and a strong fine-grained founding stratum exist appear to be associated with pile relaxation. The mechanism for pile relaxation is not clear but may be associated with dissipation of negative pore pressure around the base of the pile.

The type of driving shoe did not appear to affect the magnitude of relaxation.

To avoid retesting every pile over a period of time acceptance criteria have been developed to relate piles tested at EOD to the performance of piles tested over a longer time period. However, many piles require testing over a long period in order to develop confidence that the less tested piles will achieve the design intent.

Continual redriving and retesting of piles to demonstrate capacity in relaxing ground is a time consuming process. In these circumstances installation of additional piles to reduce the pile test load provides the fastest method to achieve design criteria. The time savings are reflected in cost savings.

Acknowledgements

The authors would like to acknowledge the Roads and Traffic Authority of New South Wales, Australia for permission to publish data from the Ballina Bypass.

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