

# DESIGN OF PILE FOUNDATION FOR AN INTEGRAL BRIDGE IN SOFT GROUND ON HUNTER EXPRESSWAY

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## ABSTRACT

The Hunter Expressway will provide a four-lane carriageway 40 km long between the F3 Interchange at Newcastle and the New England Highway at Branxton, New South Wales Australia. It is due to be opened by the end of 2013. The Hunter Expressway Alliance (HEA), comprising Roads and Maritime Services (RMS), Thiess Pty Ltd, Parsons Brinckerhoff and Hyder Consulting, is constructing the eastern section consisting of 13 km of new freeway and local road adjustments. There are 23 bridges and major culvert structures. This paper describes the foundation design challenges of bridge No. 18 at Buchanan Interchange. In particular, it will discuss foundation design constraints from the bridge structure, 5 m high embankment on soft compressible ground and its impact on pile design, construction constraints and monitoring, and pile testing.

**Keywords:** driven pile, embankment induced ground movements, preloading, integral abutment

## 1 INTRODUCTION

The Hunter Expressway Alliance (HEA) is constructing 13 km of new freeway and local road adjustments, which will become the eastern section of the 40 km-long Hunter Expressway in NSW.

The new expressway will cut the travel time between Newcastle and the Hunter Region by 28 minutes and relieve traffic congestion between Newcastle and the towns of Thornton, Maitland and Rutherford.

There are 23 bridges and culverts, including three viaducts, which are up to 787 m long and 47 m high above the gully floor. Eight of the bridges are subject to mine subsidence due to old mining works.

Zhang and Choi (2012) describe the foundation design challenges and innovative foundation design solutions for one of the viaducts.

This paper presents the foundation design challenges and cost-effective foundation design of the bridge over Surveyors Creek (BW018) on John Renshaw Drive. BW018 is a single span bridge with overall deck around 25 m long. The width between barriers varies from about 18 m at the western end to 20 m at the eastern end. (Figures 1 and 2 show the plan view and elevation of this bridge respectively.) The bridge accommodates a travel lane 3.5 m wide in the westbound direction and two travel lanes 3.5 m wide in the eastbound direction. The ends of the girders are encased in abutment and form an integral connection at the abutment headstocks. This design eliminates the need for any bearings or expansion joints. It is also required to provide an effective dry passage width for fauna movement of 3 m at normal water flow.

Embankment fill up to 5.5 m high and over 40 m long is proposed between BW019 and BW018. Boreholes, and cone penetration tests and dilatometer tests in this zone have identified compressible soil layers at abutment A. Preloading is required to limit the post-construction settlement of this embankment and its potential impact on foundation needs to be addressed.

## 2 BRIDGE DESIGN CRITERIA AND LOADS

BW018 has been designed in accordance with the Scope of Works and Technical Criteria (SWTC), the relevant additional documents from the RTA and relevant Australian Standards, such as RTA Bridge Policy Circulars and RTA Bridge Technical Directions, RTA Standard Bridge Drawings, AS5100 and AS/NZS 1170.

Design vehicular loading on the bridges consists of the SM1600 and the HLP320. BW018 is classified as a main road with high traffic volumes and as a Type II bridge for earthquake design in accordance with AS5100. A site factor ( $S$ ) of 1.0 and an acceleration coefficient ( $\alpha$ ) of 0.11 were adopted for earthquake design in accordance with AS1170.4. The abutments have been designed so that the bridge remains serviceable with the predicted loss of support to the piles due to scour effects during the 1 in 100 year ARI event. In the 1 in 2000 year ARI event, the bridge has been designed to satisfy the ultimate limit state requirements. In accordance with AS5100.2, the horizontal forces acting on the substructure due to debris impact have been considered.

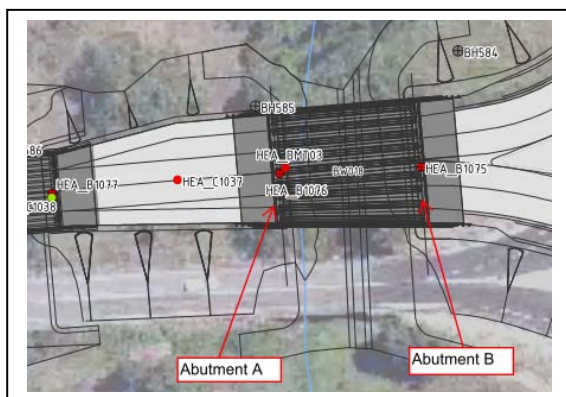


Figure 1: Plan view of BW018

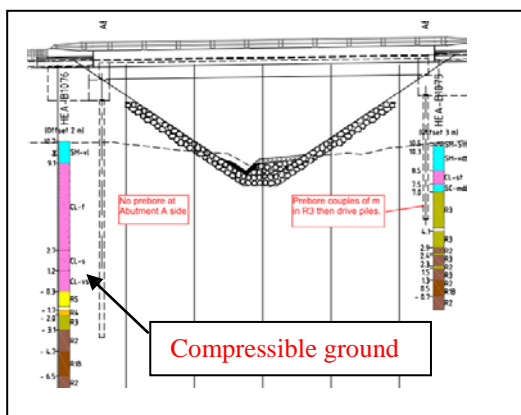


Figure 2: Geotechnical section of BW018

In accordance with C11.2 of SWTC, embankments will be designed to limit settlements and level changes in the associated pavement: a maximum total settlement of 100 mm in 40 years following the construction of pavement; a longitudinal change of grade not greater than 0.3% and a transverse change of grade not greater than 1% at the pavement surface over 40 years. In accordance with C13.25 of SWTC, a minimum factor of safety of 1.2 and 1.5 was adopted for the embankment slope stability during construction and in the long term, respectively.

### 3 FOUNDATION DESIGN CHALLENGES AND COST-EFFECTIVE DESIGN

Foundation design is complicated by the frame effect from integral abutments and the ground movements caused by embankment load on compressible ground. The risks associated with bridge foundation construction include (1) overstressing of pile foundation due to combined bridge structural loadings and post-construction ground movements; (2) loss of soil support due to scour; (3) piles overstressed due to spill through abutment slope instability; (4) excessive waiting period of preloading before piling. The design methodology and cost-effective engineering solution to eliminate/mitigate these risks are discussed in the following sections.

#### 3.1 SUBSURFACE CONDITIONS

As shown in Figure 2, subsurface conditions typically comprise loose silty sand up to 2.0 m deep, underlain by soft, firm to stiff sandy/silty clay of up to 11.5 m. The bedrock levels between Abutment A and B suggest a sloping rock profile, west to east, comprising typically low to medium strength sandstone, overlying high strength sandstone.

#### 3.2 PRELOADING OF COMPRESSIBLE GROUND

To minimise both the short-term and long-term impact of embankment load on the abutment and foundation piles, preloading of the compressible clayey layer was proposed. The proposed sequence is as follows:

1. build the first lift up to 3.0 m high in 20 days
2. wait for 30 days
3. build the second lift up to 6 m high, including 1.0 m surcharge
4. wait for 60 days
5. remove surcharge and construct piles, abutment headstock and deck.

The preloading design, actual behaviour of this embankment and back analysis are discussed in Zhang *et al.* (2012).

#### 3.3 FOUNDATION TYPE

The following factors have been considered for selecting the pile foundation type:

- portal frame effect from integral abutment subject to
  - longitudinal movements at the ends of the deck due to creep, thermal expansion and contraction, differential shrinkage and differential temperature effects
  - braking forces
  - unbalanced M1600 truck load
- preloading and embankment-induced ground movements after pile installation
- construction sequence and timing of piling work in relation to overall bridge construction program

- constructability and cost, including material cost and mobilisation cost.

A value engineering workshop was held to propose and compare different foundation options. Three foundation options were considered (Table 1) and option 3 was selected as the final design.

Table 1 Comparison of foundation design options

Design option	Type	Number <sup>2</sup> and size	Comments
1	Bored pile	4x750 mm	Attract much more load due to frame effect
2	Driven steel pile <sup>1</sup>	6x350 mm UC	High material cost and mobilisation cost
3	Driven RC pile	10x350 mm square	Low material cost, low mobilisation cost

Notes:

1. Encased in 900 mm diameter pre-bored concrete
2. Pile no per abutment
3. Top 3.0 m of the piles were isolated from the embankment fill by HDPE sleeves.

### 3.4 FOUNDATION DESIGN TO STRUCTURAL LOADS AND GROUND MOVEMENTS

Axial loads, lateral loads and bending moments arising from structural loads were provided from structural analyses. The maximum ultimate axial load and bending moment were 1500 kN and 250 kNm per pile respectively. Negative skin friction (NSF) is expected from long-term settlement. NSF was estimated to occur along the pile shaft from the soffit of the abutment to the rock head. The estimated NSF was about 200 kN per pile in working condition and ultimate NSF was about 240 kN. This was treated as an external load to the pile. The pile spacing is 2.0 m centre to centre at both abutments, which is equivalent to 5.7 times the pile size. The group effects are expected to be negligible.

Rowe & Armitage's (1987) elastic method was used to check the rock socket length. The ultimate bearing capacity was checked against the criteria set out in AS2159-2009  $\phi_g R_{ug} = S^*$ .  $\phi_g$  was taken as 0.75. Dynamic pile load testing was conducted on four piles (19% of all the piles) to verify the pile-bearing capacity.

It was expected that at Abutment A piles could be driven through extremely low to low strength rock and stop at the top of medium strength rock. However, at Abutment B medium strength rock was found below the alluvial soils. It was expected the piles could not penetrate such rock, so 400 mm diameter, 2.0 m long pre-drilling into medium strength rock was proposed. Low-strength grout was used to fill the hole before pile driving.

Shaft resistance was ignored in the design. Ultimate base resistance of 25 MPa (for medium strength rock) was used. Two piles at each abutment were given PDA tests at the end of driving (EOD). Restrike (RSK) on all the four piles was carried out on the following day. Figure 3 shows the piles installed.

Observations from CAPWAP analyses are as follows:

- At Abutment A, the EOD shaft resistance accounts for only 13% of the total resistance; it increases to 17% (RSK) after about 24 hours; the mobilised base resistance ranges from 18 to 21 MPa in both EOD and RSK. The design pile bearing capacity is achieved.



Figure 3: Piles installed at BW018

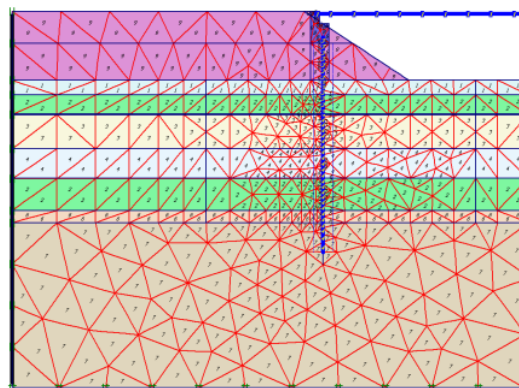


Figure 4: 2D plane strain FE model

- At Abutment B, the EOD shaft resistance accounts for only 2% of the total resistance; it increases to 7% (RSK) after about 17 hours; the mobilised base resistance ranges from 21 to 24 MPa in both EOD and RSK, which demonstrated the design pile bearing capacity was achieved.

Another important issue for pile design is the impact of post-construction horizontal movements. This impact was analysed by 2-Dimensional (2D) finite element analysis (FEA). The FEA using Plaxis 2D was carried out to assess the short-term and long-term effects of settlement and horizontal movement of the existing ground under the new embankment loading and the resulting effects on the bridge piles.

The piles (350 mm<sup>2</sup> piles with 2.0 m centre to centre spacing) were simulated as a wall using smeared bending stiffness EI and axial stiffness EA. This tends to overestimate the pile movements and forces because in the field the soil can move in between the pile gaps and the piles move less than the soils.

In Plaxis, to simulate the creep behaviour of the compressible soils the layer of soft to firm clay was modelled using a soft-soil creep (SSC) model where consolidation parameters were used in the analysis. The other subsurface layers where long-term creep is negligible were modelled using a traditional Mohr-Coulomb model. The FE model is shown in Figure 4 below and design parameters shown in Tables 2 and 3 respectively.

Table 2: Design parameters for compressible materials

Soil type	Unit weight (kN/m <sup>3</sup> )	Permeability k (m/s)	Cu (kPa)	c' (kPa)	φ' (degree)	λ*	κ*	μ*	OCR
Soft clay	16	1.0e-8	20	0	25	0.05	0.01	0.002	1.2-2.0
Firm to stiff clay	18	1.0e-8	30-60	3	25	0.03	0.006	0.001	2.5-6.0

Notes:

1. The creep model parameters were derived using laboratory index test results from samples taken from BH HEA-B1076 and in-situ test results from DMT03.
2.  $C_c = PI/74$ ,  $C_r = C_c/5$ ,  $C_\alpha = C_c/25$ .  $\lambda^* = C_c/(2.3(1+e_0))$ ;  $\kappa^* = C_r/(2.3(1+e_0))$ ;  $\mu^* = C_\alpha/(2.3(1+e_0))$ .
3. Undrained shear strength Cu was estimated from field vane shear test, DMT and/or CPTU with  $C_u = (q_c - \sigma_{v0})/N_{kt}$ ,  $N_{kt} = 15-17$ .
4. Coefficient of permeability was estimated from formula  $c_h = kE_{oed}/\gamma_w$ , where coefficient of horizontal consolidation  $c_h$  was calculated from CPTU pore pressure dissipation test result, constraint modulus  $E_{oed}$  (M in DMT) from DMT test,  $\gamma_w$  the unit weight of water.

Table 3: Design parameters for other materials

Soil type	Unit weight (kN/m <sup>3</sup> )	Permeability k (m/s)	c' (kPa)	φ' (degree)	E' (kPa)	v'
Backfill	20	1.0e-6	1	35	30000	0.3
MD sand	20	1.0e-6	1	35	30000	0.3

Note: design parameters were derived from reviewing BH logs with SPT N values and DMT results.

Due to the embankment load, the ground experienced both vertical and horizontal movements. The predicted maximum total settlement was about 360 mm, which is comparable to about 300 mm calculated from the DMT constrained modulus using conventional 1-D compression theory. The predicted maximum total horizontal movement was about 70 mm. The predicted maximum post-construction settlement and lateral soil movement is about 60 mm and 18 mm respectively. The predicted maximum post construction pile deflection was about 8 mm and the maximum SLS bending moment approximately 70 kNm.

Instruments were installed behind Abutment A to monitor the embankment settlement, pore pressure and horizontal displacement at batter slope toe. These instruments were designed to control the fill rate and short-term slope stability. Zhang *et al.* (2012) discuss the details of instrumentation and monitoring results and back analysis. No instruments were proposed for the spill through embankment due to the difficulty of installation and safety concerns for the monitoring personnel who would take the readings. However, three prisms were installed at the abutment headstock to monitor the horizontal displacement during construction.

By the time the deck was installed, the maximum measured horizontal movement from the three prisms was within 5 mm, which indicated the design prediction is reasonable and the piles have not been adversely affected.

The forces due to structural loads and those from preloading effect were superimposed to check the resultant forces. Shortening effects from creep and shrinkage in the deck result in pile moments similar in magnitude but opposite in

sign to the moments induced by residual creep effects of embankment preload. Generally, the pile bending moments from soil creep movements are relatively small compared with the pile's capacity.

## 4 OTHER GEOTECHNICAL DESIGN ISSUES

### 4.1 SCOUR PROTECTION FOR SPILL THROUGH ABUTMENT

Scour assessment of Surveyors Creek was designed to a 200 year ARI standard to account for wave action and model sensitivity. Construction scour was estimated and appropriate rock size and class provided to protect the abutment embankments from scour effects. The estimated scour depth was about 0.6 m and 1.2 m in 200 and 2000 year ARI respectively.

Spill through abutment slopes of 1.5 H:1V were provided. Scour protection, consisting of high strength rock armour in-filled with smaller sized rocks to minimise excessive undulations on the slope extend along the full length of the batters and to a point beyond where acceptable velocities occur for a grassed embankment.

### 4.2 SLOPE STABILITY FOR SPILL THROUGH ABUTMENT

Slope stability of the permanent batter was checked in the short term (during construction), in the long term, under flood and earthquake loads. The stability of the slope satisfied the design requirements for all cases.

## 5 CONCLUSIONS AND RECOMMENDATIONS

Foundation design challenges, constraints of the Bridge over Surveyors Creek (BR018) on John Renshaw Drive have been discussed. A value engineering workshop was held to compare three foundation design options and a cost-effective foundation design was proposed. To address the short-term and long-term risks of pile foundation, instrumentation and monitoring was implemented during construction. The monitoring results indicate the design is reasonable and adequate. PDA tests on representative piles were made to verify the design assumption; the CAPWAP analyses show the pile capacities have been achieved.

## 6 ACKNOWLEDGEMENTS

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