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Pyrites Creek Bridge Embankments and Piling

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ABSTRACT
The design and construction of a tall twin bridge and associated high embankments across Pyrites Creek, part of the Western Highway Anthonys Cutting Realignment project, presented a number of geotechnical challenges. These were primarily due to the underlying alluvial soils, reuse of local cut materials as fills, construction methodology, program, and a limit imposed on the bridge span to minimise cost. The challenge was exacerbated by the creek not being perpendicular to the road alignment. A staged approach was developed utilising rock fill and stabilised sand for quick construction of the steep embankments, while an innovative design solution was adopted for the abutment piling, utilising the stabilised sand to minimise negative friction. The piling consisted of precast driven piles and bored piles. Bored piles were used where it was necessary to manage bending moments induced by the adjacent large embankments.

Instrumentation was installed prior to, during, and after construction. This was used to monitor ground pore pressures, settlement of the alluvial soils and embankment fill, and movement of the superstructure. Monitoring results were fed back to the design team to check that the behaviour of the ground and fill materials was consistent with those predicted from the numerical modelling.

Keywords: Pyrites Creek Bridge, embankment, driven piles, bored piles.

1 INTRODUCTION

A new section of the Western Highway between Melton and Bacchus Marsh in Victoria passes over undulating terrain, a variety of ground conditions, and a number of waterways. At Pyrites Creek there is a tall bridge flanked by high embankments on alluvial soils. The design of the Pyrites Creek Bridge presented a number of geotechnical challenges. These included the height of the embankments, the limited span of the bridge, alluvial soil conditions, use of local materials for embankment fill, construction program, and overall project objective to deliver value for money.

An innovative engineering solution was devised for the design of the piling system for the bridge. The installation of the piles was staged with the earthworks for the high embankments. Monitoring of the performance of the earthworks and footing system was undertaken during and after construction to validate design assumptions.

The delivery of the project was undertaken by an Alliance between Vicroads, John Holland Group, and AECOM (the Alliance).

2 BACKGROUND

2.1 Site and Highway Alignment

West of the Djerriwarrh Creek Bridge, the new section of the Western Highway passes through a long rock cutting before emerging at the cut face of a quarry. The top of this rock face is almost 50m above a relatively flat alluvial plain below. The new highway crosses the quarry, a minor road (Cowans Road), and then the alluvial plain before rejoining the existing Western Highway alignment.

Pyrites Creek meanders through the eastern portion of the alluvial plain that is used to grow apples. The creek is approximately 20m wide, up to five metres deep, is heavily vegetated, and runs about 15° from the perpendicular to the new highway alignment.

As the highway leaves the rock cut above the quarry it transfers rapidly onto large fill embankments. These embankments gradually reduce in height as the road progresses towards the existing highway to the west, but are still around 22m high where they intersect Pyrites Creek.

2.2 Existing Ground Conditions

Consistent with the process in which it was deposited, the near surface alluvial materials vary significantly over short distances adjacent to Pyrites Creek. However, they could generally be characterised as stiff silty or sandy clays. It is also possible to differentiate a softer layer across most of the site three to six metres below the surface. This was defined as soft to firm silty/sandy clay. The thickness of the softer layer varies, but is generally around four metres beneath the embankment footprints. A relatively well defined deposit of dense to very dense sands and gravels sits below the clays. This continues to depths of over 27m.

An extensive geotechnical investigation was carried out in the area surrounding Pyrites Creek. A total of 26 boreholes and Cone Penetration Tests (CPTs) were performed within a 230m length of embankment by Vicroads and the Alliance.

The additional boreholes identified a seam of coal and less dense organic material at a depth of approximately 21m within the dense sands and gravels. While only around two metres thick, it was inferred to continue horizontally for 65m along the alignment of the new highway, centred about the creek. It was possible the layer may have implications for the design if the bridge's piles.

One feature that was not encountered in the geotechnical investigation was soft spots associated with the orchid trees that occupy the alluvial plain. The trees were individually watered using a drip feed system, which created a dish shaped soft spot beneath each tree. Stripping and grubbing of the surface as construction commenced revealed a grid of soft spots across the proposed embankment footprints. Dynamic Cone Penetration (DCP) tests showed the ground to be effected to over 1.5m beneath the grubbed surface in some locations. The soft spots were not picked up during the initial site investigations as each borehole and CPT was located centrally between the existing tree rows where the watering system had little or no effect.

The groundwater level corresponded roughly with the base of Pyrites Creek – approximately 4m below the surface of the plain.

2.3 Original Concept

The original VicRoads concept at Pyrites Creek was to divert the creek through the new embankment via a large reinforced concrete culvert. However, the local water authority required the existing natural course and profile of the creek to be maintained, something that couldn't be achieved with a culvert. The alternative was to terminate the embankment either side of Pyrites Creek and span over the waterway with two bridges – one for each carriageway.

3 DESIGN CHALLENGES

The decision to construct twin bridges over Pyrites Creek introduced a number of challenges for the Alliance. Many of these resulted from the self imposed requirement to limit the length of the bridge by using no more than three spans. The aim of this constraint was to control cost, but the bridges were still required to span Pyrites Creek, the spill through abutment slopes, Cowans Road, an irrigation channel, and land set aside for maintenance access. The issue was exacerbated by the creek not being perpendicular to the road alignment.

The bridge superstructures were to be constructed using tee-roff beams. These would have to be lifted into place using cranes, so their depth was limited to 1500mm to control weight. The maximum distance this beam can span is approximately 31m, restricting the total bridge lengths to 98m when the widths of the two pier crossheads are included.

Due to the heights of the embankments on each side of the creek, the greatest percentage of the bridge length would be taken up spanning over the spill through embankment slopes. To minimise this length, the embankment slopes were designed to be as steep as possible. This meant that the stability of the embankments was a concern, particularly given the proximity of the creek and the underlying ground conditions. In addition, the high embankments were expected to induce considerable settlements in the alluvial clays above the dense sands and gravels.

Each bridge had one pier on either side of the creek, within or adjacent to the toes of the embankments. These would be supported with piles, as would the abutments. The high and steep embankments were expected to induce lateral soil loads on the bridge piles and possibly its piers and superstructure. It would also be necessary to address the negative friction on the piles induced by the settlement of the soils beneath the embankments.

Installation of the abutment piles was considered a challenge. If precast piles were adopted, they would have to be driven through approximately 18m of embankment fill plus up to nine metres of alluvial soil before reaching the dense sands and gravels. As high quality fills would be used in the embankments for their stability, the piles would be expected to refuse well above the required embedment depth. Holes that are slightly smaller than the pile width to provide full lateral support could be predrilled, but over this length it would be difficult to keep the holes straight enough to avoid high bending moments being induced in the piles during installation. There would also be a high probability that the predrills would collapse before installing the piles.

Installing bored piles through the embankment fill, alluvial clays, and into the dense sands and gravels would also pose challenges. Casing would need to be very long and the base of the piles would have to be installed under polymer due to the presence of groundwater. Temporary casing would be difficult to extract and permanent casing would be very expensive.

Bored piles would also be expected to attract considerable negative friction due to their large, rough surface area within the embankment and underlying settling soils. The bridge piles were to be designed using the new piling standard AS2159–2009 and tested in accordance with the standard and the requirements of VicRoads specification Section 606. This would involve applying a sufficient test load to mobilise the portions of the abutment piles within the stable zone in order to assess their ultimate geotechnical strength. If skin friction through the embankment was high, it may be difficult and costly to source equipment capable of generating this load and applying the load without damaging the pile.

4 THE SOLUTION

4.1 Bridge / Embankment Configuration

The side slopes of the embankments on either side of Pyrites Creek were inclined at a combination of 1V in 2H and 1V in 2.5H. However, the space restrictions meant that the embankment batters beneath the bridge were to be constructed at 1V in 1.5H. At this slope, it was necessary to construct the embankments with high quality fills to maintain stability. Type A fill was used adjacent to structural elements such as the abutments, piers, and piles. Rock fill was used elsewhere, because it could be readily sourced from the rock cuts excavated for portions of the new alignment to the east.

Despite the superior quality fill, a single bench (a minimum of five metres wide) was required in each slope to improve stability. Cowans Road was moved from ground level onto the bench on the eastern side to utilise this space. While this provided additional space, it had the effect of moving the toe of the eastern embankment closer to the creek bank.

Initially there was an aim to keep the abutments and piers of both bridges aligned with each other. This would enable a simpler design as well as construction, particularly as a launching gantry that could crab between both decks was being considered to build the bridges. However, when it was confirmed that the existing natural course and bed profile of the creek had to be maintained, it was necessary to stagger the bridges so that the embankment fill did not encroach into the creek. The subsequent decision to lift the pier and deck components into place with cranes negated the launching issue.

4.2 Modelling and Construction Staging of Embankments to Maintain Stability

Plaxis was used to model the embankments and their interaction with the various bridge components. One generic long section was generated that was representative of the worst portions of the embankments. For example, it incorporated the steepest embankment slopes, the deepest creek bed profile, and the smallest toe/bank offsets. While the Mohr-Coulomb material model was used for most materials, a soft soil model was used for the soft to firm clay layer.

Stability analyses of the embankment slopes beneath the bridges indicated they had a satisfactory long term factor of safety. The analyses were performed using both Plaxis and Slide. However, the stability of the embankments using undrained parameters was not as high, which was unusual. There was concern that if the embankment was built up too rapidly excess pore pressures would develop in the alluvial clays, compromising the stability of the embankments.

A consolidation analysis was performed using Plaxis to model the increase and subsequent dissipation of excess pore pressures in the alluvial deposits after fill for the embankments was placed. The model showed that if the excess pore pressures were not given sufficient time to dissipate the calculated factor of safety would decrease to well below 1.3 – the limit adopted for the temporary construction case. As a result, a staged construction sequence was created for the bridge embankments. Each stage involved constructing the embankments up to the next defined level at a maximum rate and with a minimum interval until the next stage could commence. This permitted some freedom to the construction team to adapt their program as required to cope with unforeseen events while not causing delays to embankment construction.

Vibrating wires piezometers (VWPs) were installed in 10 locations to record pore water pressures in the alluvial clays as the embankments were being constructed. The VWPs indicated the pressure increases to be much smaller than anticipated, which meant that it was not necessary to slow down fill placement to allow pore pressures to dissipate. The small pressure build ups were attributed primarily to sand lenses within the alluvial deposits increasing its bulk permeability.

4.3 Settlement of Embankment

An appreciable amount of settlement was expected due to the size of the bridge embankments and the depth of the alluvial clays overlying the dense sands and gravels. A preliminary estimate of 510mm was derived for the primary settlement beneath the eastern bridge embankment. Fifteen settlements plates were installed to monitor vertical movements of the in situ soils as the embankments were constructed.

A fortunate side effect of the embankment construction staging, which was instigated to control stability, was that much of the settlement would occur prior to the bridge piles and other structural components being installed. However, the embankment could not be constructed to its full height before the installation of the abutment piles, because the remaining fill would induce negative friction and lateral soil loads onto the piles as it was placed. Our modelling indicated that these loads would have a significant influence on pile design and testing requirements. As a result, an additional stage was incorporated into the construction sequence that consisted of forming a surcharge mound over the abutment hardstand to induce much of the primary settlement that would otherwise have occurred after the abutment piles were installed. The surcharge mound was designed to match or exceed the weight of the fill that would be placed after the abutment was constructed.

4.4 Pier and Abutment Pile Design

It was originally proposed to use vertical precast driven piles to support the bridge piers. They were vertical to avoid attracting large bending moments from the settling soils. However, the combined actions generated by the soil and structural loads were too high and bored piles were required. Bored piles were also adopted to support the abutments, primarily due to the problems associated with installing driven piles through the embankments mentioned previously. All of the piles would be embedded into the dense sands and gravels beneath the alluvial clays.

The lateral soil loads and negative friction drag forces for each pile were derived with the assistance of Plaxis. A pile group analysis was performed using Repute to assess the pile actions under the various structural loads cases. The loads were then combined in accordance with the new piling standard AS 2159–2009. Despite the extensive site investigation, method of analysis adopted, and high level of construction control, it was difficult to justify an average risk rating of less than 2.5 at the site. As a result, pile testing was required in order to adopt a geotechnical reduction factor above 0.4. A higher factor was required to limit pile lengths and not approach the seam of coal and less dense organic material identified beneath the creek. In addition, the VicRoads specification requires that bored piles are tested to confirm their integrity and design geotechnical strength.

Designing, documenting, and testing the piles for the ultimate compression loads in accordance with AS 2159–2009 was quite straight forward. The ultimate test load is,

$$P_{g1} = E_d/\phi_g, \tag{1}$$

where E_d = design action effect and ϕ_g = geotechnical reduction factor.

For piles subject to negative friction, the guidance in the standard for the serviceability test loads is also clear. The serviceability test load is,

$$P_s = E_{ds} + 2F_{nf}, \tag{2}$$

where E_{ds} = design serviceability load and F_{nf} = action due to negative friction.

However, the settlement of the piles subjected to negative friction is controlled by their embedment into a "stable zone". Therefore, this stable zone is required to have a sufficient ultimate geotechnical strength, and this strength needs to be confirmed by testing.

The ultimate test load applied to piles subject to negative friction needs to be sufficient to overcome the shaft resistance that will occur through the ground above the stable zone during the load test and mobilise the portion of the piles embedded within the stable zone. To confirm the required geotechnical strength, the load applied to the tops of the portions of the piles in the stable zone should be a minimum of $(E_{ds} + 1.0F_{nf})/\phi_g$. Therefore, the ultimate test load for piles subject to negative friction would be,

$$P_{g2} = (E_{ds} + 1.0F_{nf})/\ \phi_g + F_{m,s}, \tag{3}$$

where $F_{m,s}$ = allowance for shaft friction in the settling ground.

To avoid having two ultimate test loads, P_g was defined in the drawings as the greater of the two cases (P_{g1} and P_{g2}). In this way, the results of the dynamic pile testing would allow for the assessment of the piles to support each scenario. At Pyrites Creek Bridge, P_{g1} was critical for the pier piles while P_{g2} was greater for the bored piles supporting the abutments. The ultimate test load for the pier piles was almost 11,000kN, so a large hammer, purpose built for bored piles, was required to perform the PDA testing. Similar test loads were required for the abutment bored piles to overcome the shaft resistances generated by the piles as they pass through the deep embankments and alluvial deposits.

5 EVOLUTION OF DESIGN DURING CONSTRUCTION TO FINAL SOLUTION

5.1 Fill Used to Construct the Embankments

Prior to construction commencing, it was anticipated that material suitable for use as Type A fill would be readily available on site. As a result, a large percentage of the embankments at Pyrites Creek were to be constructed using Type A. However, as the cuts for the new road alignment progressed it became clear that material would need to be imported to meet the project-wide Type A need. So the design evolved over a number of iterations to include less Type A fill and more rock fill.

As construction of the embankments commenced, Type A fill was still required around the various structural components, including the piles. The Type A in the abutment pile zones formed 2m wide blocks within the embankments that were otherwise rock fill. Creating these blocks was inconvenient as the Type A needed to be spread in layers not exceeding a compacted thickness of 200mm, while the rock fill could be placed in 600mm lifts. A possible solution was put forward to use 3% cement stabilised sand instead of Type A fill in the pile zones. This would allow the rock fill to be placed in homogenous layers across the entire embankment. An excavator could then be used to form the slot into which the stabilised sand would be poured, while the plant used to spread the rock fill could be used elsewhere. Despite the added cost of the cement stabilised sand, it would be more cost effective for embankment construction to use it in the pile zone instead of Type A fill. However, this solution had implications for the abutment piles.

5.2 Pier and Abutment Piles

An added benefit of the cement stabilised material was that it would be self supporting during the installation of the bored piles, so casing would no longer be required through the embankment.

However, there was concern that friction between the bored piles and stabilised sand would be much greater, resulting in significantly higher drag forces. Incorporating the stabilised sand into the models confirmed this. The length and size of the bored piles would need to be increased for the stable zone to have sufficient geotechnical strength. Much higher test loads would also be required to mobilise the stable zone, and hammers capable of generating these loads were not readily available.

An innovative approach was then conceived for the piling of the abutments. The Alliance would go ahead with the proposal to place 3% cement stabilised sand within the pile zone instead of Type A fill. Each pile location would then be predrilled down to just above the water table. After drilling was complete, precast driven piles could be lowered into the holes and the annulus filled from the surface with uncompacted sand; in this case the sand from the predrills. The piles would then be driven to effective refusal into the underlying dense sands and gravels like standard precast piles.

A predrill diameter of 750mm was used for the 400mm square driven piles. This allowed some margin for the holes to be out of plumb and for the annulus to be filled without clogging up. Finishing the piles just above the water table ensured they would not refuse before reaching the stable zone and minimised the chance of significant collapse of the predrills. Minor over breaks in the alluvial soils would not concern the driven piles.

The original pile design consisted of four 900mm diameter bored piles at each abutment. This was converted into two rows of 400mm square precast driven piles 1m apart – four piles in the back row, five in the front. Two pile rows were required so a portion of the bending moments was carried in push-pull and to provide some tolerance to eccentricity due to out of position piles. Indicative pile lengths were similar for each pile type.

The installation process went smoothly, and in most cases, the piles were driven to just above the indicative toe levels shown on the construction drawings. Some compaction of the annulus fill was expected during driving. PDA testing of the abutment piles indicated very little skin friction over their prebored sections. The annulus sand was providing full lateral support, but would not transfer a high percentage of the negative friction to the piles, so the installation methodology resulted in a very large reduction drag forces.

The installation of the bored piles for the piers was completed without major incident. As anticipated, casing and polymer were required to support the hole, and the final toe levels all finished within 0.5m of the indicative values provided on the construction drawings. Two representative pier piles underwent dynamic pile testing. The results confirmed their geotechnical strengths were sufficient.

6 CONCLUSION

An integrated solution was developed for the design and construction of the Pyrites Creek Bridge and its adjacent embankments. The solution addressed numerous engineering and project challenges, and included an innovative piling solution that was cost effective and facilitated faster and easier construction of the embankments. The unique solution was facilitated through the strongly collaborative culture and leadership of the Alliance.

Construction was near completion at the time of preparing this paper. The road is now open and in use by the public.

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