Reinforced Soil wall and approach embankment for Cliff Street Overpass constructed on stabilised foundations

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ABSTRACT: The Portland Transport Strategy called for a new bridge to be constructed over the Henty Highway and Portland/Hamilton railway line at Cliff Street, Portland in Victoria. The three span trough beam bridge was to be supported on piles inside Reinforced Soil Structures (RSS) abutments. The foundation for the Northern approach to the new bridge consisted of unconsolidated fill underlain by alluvium consisting of soft to firm clayey/sandy silt to a depth of 10m below the existing surface. Groundwater table was measured to fluctuate between 1m and 3m. The northern RSS abutment was supported on a 400mm deep layer of cement stabilised sand raft, reinforced with geogrids and geotextile that was constructed on top of an arrangement of capped RC piles. The balance of the approach embankment was supported on an arrangement of shallow stone columns constructed using the Dynamic Replacement (DR) technique.

This paper will detail the development of the design from initial investigation through to construction with particular emphasis on the design of the foundation stabilisation works and the interaction with the RSS abutment and verification of the ground improved by DR during construction. The performance of the approach embankment and the RSS abutment is being monitored and the actual overall settlement will be compared to the prediction of 50mm maximum after two years on completion of the construction.

1. INTRODUCTION

The township of Portland was established in 1834 and since then it has become one of the major seaports in Australia. The Port of Portland is a deep-water bulk port strategically located between the capital city ports of Melbourne and Adelaide. It has facilities capable of handling the berthing of all types of bulk and general cargo vessels. The port is well served by a road and rail network. Typical daily truck movements to and from the port are forecast to grow by 225% by 2030 (VicRoads Report 2004). The Cliff Street overpass project consists of a 3-span bridge with prestressed concrete beams as the superstructure. The main span is approximately 35m long with two end spans each approximately 20m long. The Project was approved in 2005 with an estimated cost of $15 million. The construction of the Cliff Street Overpass project was awarded to Akron Construction Pty Ltd.

This Paper presents the findings of the geotechnical investigations at this site. It also provides detailed discussion on the design, construction, and testing of piles, ground improvement and a reinforced soil structure.

2. GEOLOGY

The south side of the site comprises Quaternary age igneous rock consisting of Iddingsite basalt which comprises the volcanic flows of the Portland area (Geological Survey of Victoria). Basaltic materials were not however encountered beneath the actual bridge site. The northern side of the site comprises Quaternary alluvium consisting of flood plain and river terrace deposits. At depth, the entire site is underlain by a sequence of Quaternary age weathered calcareous sands (aeolinites). From historical maps of the site, it appears that a pre-existing drainage channel near to the northern area of the site has been in filled and replaced with a man-made canal.

3. PERFORMANCE CRITERIA FOR CLIFF STREET OVERPASS

- The following performance criteria were specified:
  - Bridge foundation – maximum differential settlement 10mm
  - Reinforced soil wall facing panels – maximum differential movement 10mm, and
  - Bridge approach fill embankment – maximum differential settlement 50mm.
4. GEOTECHNICAL INVESTIGATION

A geotechnical investigation was undertaken at this site by VicRoads GeoPave with reference to AS1726. Because of the expected difficult ground conditions, the investigation sites were carefully selected to ensure that adequate geotechnical information would be available for design and construction purposes. Details of the fieldwork are as follows:

- Thirteen (13 No) investigation boreholes, four of which have incorporated a standpipe for subsequent groundwater monitoring. Four of the boreholes were extended to a depth of 60m in order to ascertain the strength of the underlying weathered limestone (calcareous sand),
- Eight (8 No) test pits for the proposed road improvement work,
- Five (5 No) Cone Penetration Tests (CPT), and
- Four (4 No) pavement dippings at tie-in locations between the proposed road improvement work and the existing road pavement.

The boreholes were advanced by auguring and wash boring. NW casing was required to prevent caving of the boreholes when drilling encountered very loose silty sands and/or soft clays. Field sampling and testing of the soils consisted of Standard Penetration Tests (SPTs) in sandy soils and undisturbed tube samples in predominantly clayey soils. Sampling was undertaken at approximately 1.5m intervals. Drilling was extended below the soft material until a suitable founding medium was found, typically 60m at the piers and north abutment location. Following the finding of the weak layers from the drillings, it was necessary to determine the extent and frequency of these layers. CPT was selected to supplement the drilling investigation. The CPTs were performed using VicRoads penetrometer test vehicle. CPT testing was extended to effective refusal at depths ranging from 17.6m to 45.6m. Test pits performed in the uncontrolled fill, Figure 1, confirmed the presence of weak material and shallow ground water depth. All of the test pit walls collapsed drilling excavation.

Table 1 summarizes the subsurface conditions at this site and the strength parameters adopted in the pile design. The overlapping of layer thickness indicates the variability of stratification across the site.

<table>
<thead>
<tr>
<th>Thickness</th>
<th>Description</th>
</tr>
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<tbody>
<tr>
<td>0 – 4.0m</td>
<td>Uncontrolled FILL.</td>
</tr>
<tr>
<td>0 – 5.6m</td>
<td>Extremely weathered BASALT (south side only)</td>
</tr>
<tr>
<td>2.0m – 15m</td>
<td>Firm to stiff silty CLAY,</td>
</tr>
<tr>
<td>8.5m – 40m</td>
<td>Loose to very dense calcareous SAND.</td>
</tr>
<tr>
<td>26m – 36m</td>
<td>Very soft clay (North side location only)</td>
</tr>
<tr>
<td>36m – 60m</td>
<td>Medium dense to very dense calcareous SAND.</td>
</tr>
</tbody>
</table>

Groundwater monitoring indicated that groundwater could be expected at depths between 1.8m (North approach embankment) and 1.35m (Road cutting along the Link Road) below the existing surface. The presence of groundwater at shallow depth would need to be addressed carefully, particularly if remediation measures are required to treat the uncontrolled fill. This will be discussed later in this paper.

5. DESIGN CONSIDERATIONS

Based on the investigation results, the following issues were addressed to ensure that satisfactory performance of the structure would be maintained over its entire design life. These issues were:

- Bridge structure foundation,
- Reinforced soil structure foundation,
- Ground improvement work for construction of embankment fill on the North approach.

5.1 Bridge Structure Foundation

Various foundation types, including steel shell piles were considered. Based on costs and availability considerations, reinforced concrete driven piles were considered the most suitable foundation type at this location. In accordance with AS 5100.3 Foundation, a material strength reduction factor of 0.5 was adopted in the geotechnical strength design. The design computations were based on “Pile Design and Construction Practice”, 4th edition, M.J. Tomlinson. In order to satisfy the performance criteria for the bridge foundation, the piles were designed to be driven into the weathered limestone to founding depths between 35m and 47m below the existing surface. The general layout of the overpass is depicted in Figure 1.

Figure 1. Typical test pit showing the condition of the uncontrolled fill.
5.2 Reinforced Soil Structure and Foundation

The RSS design was undertaken by the Reinforced Earth Company whilst VicRoads GeoPave designed the foundation treatment.

The Reinforced Earth wall design combines galvanized, medium tensile steel reinforcing strips with granular backfill to create a strong yet flexible retaining wall. The steel strips are a hot-rolled, deformed flat bar with minimum tensile and yield strength of 520 MPa and 355 MPa respectively at 22% elongation. The strips are rolled with a localized thickening at the connection to ensure that the strip capacity is not controlled by the connection strength. With the bridge loads directly supported on piles the peak bearing pressure at the base of walls was 230 kPA.

The project architects specified a complex finish to the precast concrete wall panels and a tapered coping to the top of the wall (Figure 3). The panels were 2m x 2m embossed with 30mm and 50mm deep relief in three different patterns. These patterns were then arranged in three orientations to provide a random patterned appearance in the finished wall.

Based on the investigation results, it was estimated that long-term settlement in the subsurface stratifications beneath the 8 metre high reinforced soil structure (RSS) would be in the order of 300mm. The differential settlement was expected to be about 100mm. Therefore, the risk of rotational and vertical movement of the RSS would be very high. As a consequence, the bridge abutment piles would likely be subjected to excessive lateral/vertical (down drag) loading induced by movements of the RSS (Stewart 1999).

To reduce the potential for movement of the RSS, the RSS was designed to be supported on piles. Based on ease of construction, 350 x 350mm RC driving piles were selected as the foundation for the RSS. The entire footprint of the RSS was designed to be supported on piles installed at a 2m square grid. The founding depth of the piles was 15m below the existing surface (i.e. R.L. -13m). The design pile capacity was 500kN (ULS) per pile. To ensure that the RSS loading was distributed as evenly as possible to the piled foundation, a raft consisting of geo-grid reinforcing and cement-treated sand was constructed over the piled area. In addition, a pile cap, 600mm x 600mm, was provided for each of the piles for transfer of loads from the raft. The pile layout at the north abutment, geo-grid reinforce raft and arrangement at the pile top is shown

![Figure 4. Layout of piled raft foundation for placement of bridge approach embankment fill at North Abutment.](image)

![Figure 5. Geo-grid reinforced raft and top of pile](image)

The specification for the geo-grid reinforced raft was as follows.
1. Ultimate tensile strength (max strain 10%): 40 kN/m in both longitudinal and transverse directions
2. Loaded at 2% strain: 14 kN/m in both longitudinal and transverse directions
3. Loaded at 5% strain: 28 kN/m in both longitudinal and transverse directions
4. Junction Strength: 95% of (i) to (iii) above.
5. The geo-grid reinforcing was of Polypropylene type
6. Shall have properties to inhibit attack by UV light
7. Shall be unaffected by all chemicals, including acids, alkalis and salts, and shall not be affected by micro-organisms in the soil.

5.3 Embankment Fill Foundation

5.3.1 Ground Treatment Options

The design criteria required that the total settlement and differential settlement must not exceed 50mm and 50mm respectively, over the design life of the structure, nominally 100 years. Based on investigation results, it was considered difficult to satisfy these design criteria as it would be difficult to predict the behaviour of the 4m thick uncontrolled fill when it is subjected to the weight of the bridge approach embankment fill. It was decided that ground improvement would be required in order to satisfy this design requirement. The total area to be improved was 3,500m$^2$ approximately.

As the construction of this Project was given a very tight time frame, ground improvement using the conventional surcharge method was not considered. Several ground treatment options including lightweight fill were considered. The lightweight fill option was not favoured as it required construction of containment structures. Other options such as dynamic compaction, dynamic replacement, grout injection and stone columns were also considered (Arulrajah & Abdullah 2002a, b, Arulrajah et al. 2004). The stone column option was selected as it was considered this option would have the least risk of causing excessive ground vibration and noise during construction. However, Dynamic replacement (DR) was accepted as an alternative solution after the contractor demonstrated that ground vibration and noise could be managed and minimized with appropriate construction controls.

The specification for Dynamic Replacement proposed by the Contractor is as follows:
1. Targeted depth of improvement is 4m minimum
2. DR columns shall not be spaced greater than 3m on a square grid.
3. Impact hammer weight 8 to 25 tons, dropped in free fall from 15 to 25m
4. Noise level: less than 75db at a distance 50m from the source of impact
5. Vibration level: less than 3mm ppv at a distance 70m from an impact source of 168 tonne-metres.
6. Material to be granular fill with D(max) < 100mm and percentage (by mass) of fines passing the 75 micron sieve to be less than 10%.

Figure 6. Area required DR ground improvement

Ground improvement was undertaken by Austress Menard Pty Ltd using the dynamic replacement technique.

5.3.2 Dynamic Replacement description and history

Dynamic Replacement (DR) is a method in which columns of large diameters are formed with granular material based on the techniques developed for Dynamic Consolidation in highly compressible and weak soils. This technique is similar to Dynamic Consolidation however the pounding is used to form large diameter granular pillars through the material to be improved. The columns of granular material formed are called “pillars”. This method combines the advantages of Dynamic Consolidation with those of Stone Columns whilst providing an economical edge since excavation of the weak soil is avoided. Also, high internal shearing resistance is provided within the pillars. These pillars also act as large vertical drains and induce a reduction in the consolidation period. Schematic principle of the Dynamic Replacement method is illustrated in Figure 7.
The equipment used for Dynamic Replacement is similar to the Dynamic Consolidation equipment i.e. heavy rigs and pounders. However, usually pounders with smaller areas are used to facilitate the penetration capacity (Menard Soltraitement 2006). Heavy Dynamic Replacement (HDR) columns are made with boulders and cobles using energies exceeding 400tm per blow. The relationship between the effective depth of attained improvement, the pounder weight and the height of the drop is expressed as reminded in equation (1):

\[ D = \left(0.3t0.7\right)^{\frac{W.H}{W}} \]  

(1)

where:
- \( D \) = maximum depth of improvement in metres;
- \( W \) = falling weight in metric tons;
- \( H \) = height of drop in meters (Mitchell & Gallagher 1998).

On the Cliff street project, pre-excavation was performed down to 2 meters in order to penetrate the hard top layer and to allow for the installation of deeper columns especially in the clayey materials encountered.

Construction control methods of Dynamic Replacement operation on site are similar to those of Dynamic Consolidation and include heave penetration tests, measurement of volume of stone used, number of drops per print and overall platform settlement. Once the Dynamic Replacement has occurred conventional soil investigation can be performed such as CPT, SPT and pressure meter tests (PMT) (Robertson and Campanella 1988).

5.3.3 Model

In order to estimate the settlement of the improved layer, a finite element analysis was performed using the software Plaxis.

The concept is to perform a settlement calculation over an axi-symmetrical model representing one column and its surrounding soil over one cell: (ie. column and soil over a 5m*5m grid) refer figure 3 in following section.

The equivalent radius of the model is 2.82 meters and the dynamic replacement pillar was found to be 1.5 meters in diameter after in-situ measurements. The pillars were assumed to extend 4.5 meters deep and a young modulus parameter of 40MPa was retained.

Finally a service load of 10kPa was taken in consideration.

5.3.4 Soil improvement results

A total of 81 CPT tests were performed after completion of the DR works between the DR prints. The compilation of these soil parameters highlights the consistent improvement throughout the treated layer. An average \( q_c \) value of 4MPa was found between pillars and after improvement this represents an improvement of \( q_c \) values of 50 to 100% (refer figure 9). CPT tests were performed 2-3 weeks on average after the installation of the DR pillars and the improvement measured is likely to account for some consolidation due to the increase in horizontal permeability as well as the improvement obtained by means of compaction. Figure 9 shows the CPT results before and after improvement.
According to Mitchell & Gallagher of 0.5 approximately. Thirteen SPT tests were also performed within the DR prints and they showed consistent improvement down to depths of 4 meters with N SPT results between 25 and 35 illustrating the good quality of the DR prints obtained.

6. CONSTRUCTION

6.1 Piling Installation.

Wagstaff Piling Pty Ltd was the piling contractor for installation of the piles. A six tonne hydraulic hammer was employed to drive the piles. Hammer drop heights for the bridge foundation piles ranged from 600mm to 800mm and typically 600mm, whereas the drop height was 150mm for all of the RSS foundation piles. No major construction issue was reported during construction. In general, piles were driven to depths as expected although relatively significant variation in pile toe levels were experienced between bridge support locations. The difference in pile penetration depths within the same pile group was judged likely due to a reduction of the thickness of the weak layers at that location.

6.2 Ground Improvement

Ground vibration monitoring results indicated that the vibration level was within the contract specification. Noise level was within the acceptable level with no complaint having been received from the local community during construction. The required ground improvement was generally achieved. At the DR columns, the required density was confirmed by the drilling with SPT values generally above N20 (N15 was required) through to a depth of 5m (4m was required).

Reinforced Soil Structure Wall

The RSS wall was constructed by Akron Construction Pty Ltd with assistance from Austress Freyssinet Pty Ltd as a specialist subcontractor. During construction of the wall there was no evidence of panel misalignment or distress due to settlement of the piled foundation support.

7. CONCLUSION

Various geotechnical designs were provided to suit specific foundation requirements for the overpass structure such as the bridge supports, the earth retaining abutments and the high embankment fill. The geotechnical designs utilized reinforced concrete piles driven to a depth in excess of 50m for the bridge foundations. A piled raft foundation was provided for the RSS wall and the Dynamic Replacement ground improvement technique was adopted to remedy the existing uncontrolled fill prior to placement of the bridge approach fill embankment. The ability to provide appropriate geotechnical design is attributed to a good understanding of the subsurface conditions at this site. Therefore, it may be concluded that an appropriate geotechnical site investigation is vital in ensuring that geotechnical issues can be addressed adequately during the detailed design stage.

Cliff Street overpass opened for traffic in January 2007. There has been no report of cracking of the road pavement constructed on the approach embankment fill. However, the long-term performance of the DR ground improvement and the piled raft foundation for the RSS wall will require further assessment.

Reference


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