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Reinforced soil wall and approach embankment for Cliff Street overpass constructed on stabilized 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 10 m below the existing surface. Groundwater table was measured to fluctuate between 1 m and 3 m. The northern RSS abutment was supported on a 400 mm deep layer of cement stabilized sand raft, reinforced with geogrids and geotextile that was constructed on top of an arrangement of capped reinforced concrete (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 50 mm maximum after two years on completion of the construction. 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 thus far.

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 35 m long with two end spans each approximately 20 m 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 10 mm
- Reinforced soil wall facing panels – maximum
- differential movement 10 mm, and
- Bridge approach fill embankment – maximum differential settlement 50 mm.

4 GEOTECHNICAL INVESTIGATION

A geotechnical investigation was undertaken at this site by VicRoads GeoPave with reference to Australian Standard 1726 (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 field-work are as follows:

- Thirteen (13) investigation boreholes, four of which have incorporated a standpipe for subsequent groundwater monitoring. Four of the boreholes were extended to a depth of 60 m in order to ascertain the strength of the underlying weathered limestone (calcareous sand),
- Eight (8) test pits for the proposed road improvement work,
- Five (5) Cone Penetration Tests (CPT), and
- Four (4) test holes to determine the existing pavement composition 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 1.5 m intervals. Drilling was extended below the soft material until a suitable founding medium was found, typically 60 m 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 penetrometer test vehicle. CPT testing was extended to effective refusal at depths ranging from 17.6 m to 45.6 m.

Figure 1. Typical test pit showing the condition of the uncontrolled fill.

Table 1. Generalized sub-surface conditions.

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

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.

Groundwater monitoring indicated that groundwater was at depths between 1.8 m (North approach embankment) and 1.35 m (Road cutting along the Link Road) below the existing surface. The presence of groundwater at shallow depth needed to be addressed carefully, particularly if remediation measures were 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,

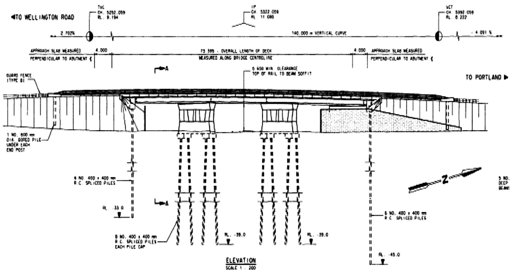

Figure 2. Elevation – General foundation layout.

– Ground improvement work for construction of embankment fill on the North approach.

Figure 3. View of completed reinforced earth wall.

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 35 m and 47 m below the existing surface. The general layout of the overpass is depicted in Figure 2.

5.2 *The RSS and foundation*

The Reinforced Earth Company designed and supplied the RSS and VicRoads GeoPave designed the foundation treatment.

The Reinforced Earth wall design combines galvanized, medium tensile steel reinforcing strips with granular backfill that was compacted against precast concrete facing panels in a single stage operation to create a strong yet flexible retaining wall. Chemical and electrochemical testing confirmed the backfill to be suitable for steel reinforcing strips and a design service life in excess of 100 years. The steel strips are a hot-rolled, ribbed 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 site of the bolted connection to ensure that the capacity of the 45 mm × 5 mm strip 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 facing panels and a tapered coping to the top of the wall (Figure 3). The panels were 2 m × 2 m × 140 mm embossed with 30 mm and 50 mm deep relief in three different patterns. 50 mm × 5 mm galvanized steel tie points were cast into the panels for connecting the ribbed steel strips. The arrangement of the patterned panels formed a random appearance in the finished wall.

Based on the investigation results, it was estimated that long-term settlement in the subsurface stratifications beneath the 8 m high RSS would be in the order of 300 mm at the North abutment location. The differential settlement was expected to be about 100 mm. 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 and 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 × 350 mm 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 2 m square grid. The founding depth of the piles was 15 m below the existing surface (i.e. R.L. −13 m). The design pile capacity was 800 kN (Ultimate Limit State) per pile. To ensure that the RSS loading was distributed as evenly as possible to the piled foundation, a raft consisting of 2 layers of bi-oriented geogrid reinforcing and cement-treated sand was constructed over the piled area. The geogrid was of polypropylene type with a design ultimate tensile strength (max strain 10%) of 40 kN/m in each direction. In addition, a pile cap, 600 mm × 600 mm, was provided for each of the piles for transfer of loads from the raft. The pile layout at the north abutment, geogrid reinforce raft and arrangement at the pile top is shown below (Figures 4 and 5).

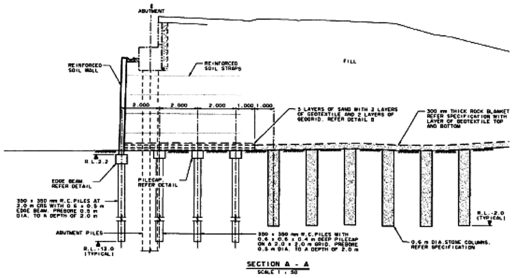

Figure 4. Layout of piled raft foundation for placement of bridge approach embankment fill at North abutment.

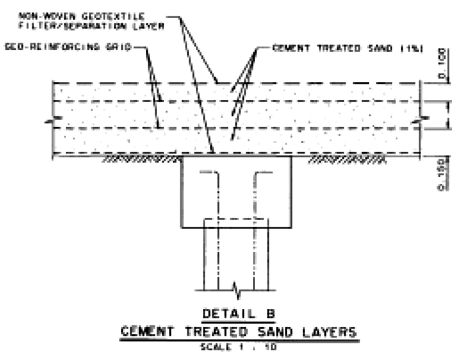

Figure 5. Geogrid reinforced raft and top of pile.

The specification for the geogrid 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 geogrid 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 50 mm and 50 mm 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 behavior of the 4 m thick uncontrolled fill

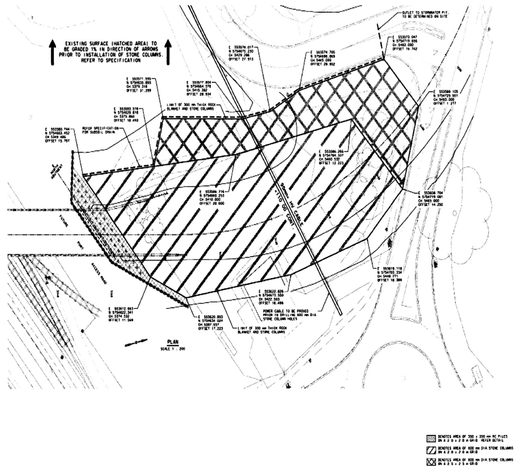

Figure 6. Area required DR ground improvement.

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 3,500 m^2 approximately.

Ground improvement using the conventional surcharge method was not considered due to the tight construction time frame. Several ground treatment options including lightweight fill were considered. The lightweight fill option was not favored 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 since it was considered to have the least risk of causing excessive ground vibration and noise during construction. However, 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 DR proposed by the Contractor is as follows:

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

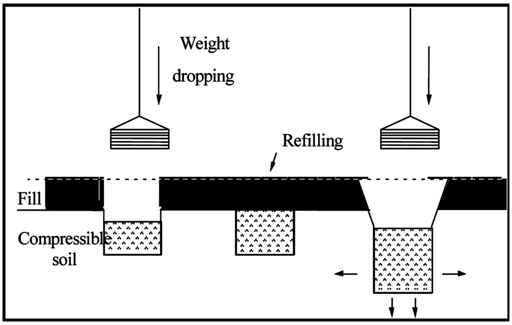

Figure 7. Schematic principle of method.

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

5.3.2 *Dynamic replacement description and history*
DR is a method in which columns of large diameters are formed with granular material based on the techniques developed for Dynamic Consolidation (DC) in highly compressible and weak soils. This technique is similar to DC 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 DC 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 DR method is illustrated in Figure 7.

The equipment used for DR is similar to the DC 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 400 tm 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):

$$D = \alpha\sqrt{W.H} \qquad (1)$$

where:

D = maximum depth of improvement in metres;
α = damping factor (varies 0.3 to 0.7)
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 DR operation on site are similar to those of DC and include heave penetration tests, measurement of volume of stone used, number of drops per print and overall platform settlement. Once the DR 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 5 m × 5 m grid).

The equivalent radius of the model is 2.82 meters and the DR 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 40 MPa was retained.

Finally a service load of 10 kPa 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 (measured cone resistance) value of 4 MPa was found between pillars and after improvement this represents an improvement of q_c values of 50 to 100% (refer figure 8). 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 8 shows the CPT results before and after improvement.

Improvement can be found up to depths of 6 to 7 meters. Considering the weight and height used of 12 tonnes and 17 meters respectively we find an $\alpha = 0.5$ approximately (after Mitchell & Gallagher).

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

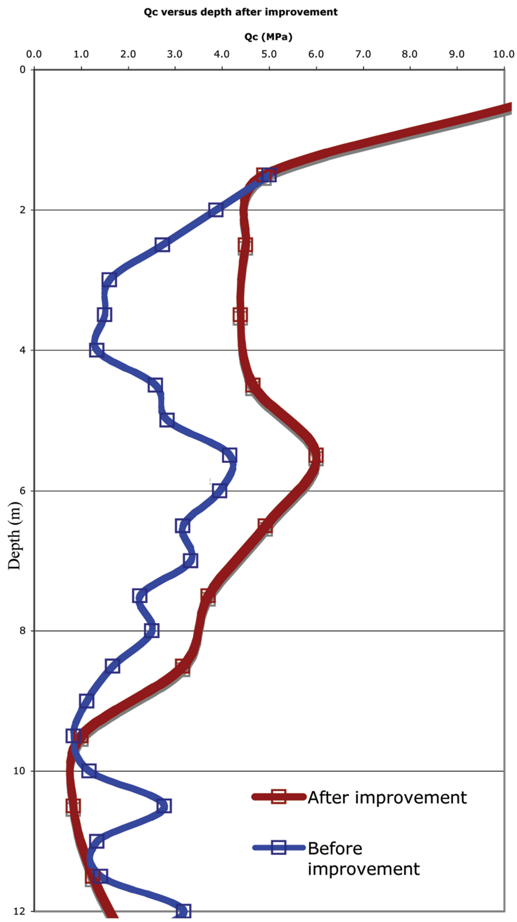

Figure 8. Estimated q_c before and after DR works.

was employed to drive the piles. Hammer drop heights for the bridge foundation piles ranged from 600 mm to 800 mm and typically 600 mm, whereas the drop height was 150 mm 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 5 m (4 m was required).

6.3 *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 50 m for the bridge foundations. A piled raft foundation was provided for the RSS wall and the DR 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 thus far.

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