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Rock mesh application in highly fractured basalt rock cutting in Western Ring Road widening project Melbourne – A case study

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ABSTRACT

Widening of Melbourne's M80 Western Ring Road carriageway required a significant vertical rock cut leading to a new fill embankment for the Moonee Ponds Creek crossing. The rock comprised highly fractured and variably weathered Newer Volcanics basalt. Excavation in other sections of the project in similar fractured basalt had led to significant overbreak. For a conventional concrete faced soil nail solution, similar overbreak in this rock cut was considered to create an appreciable budget overrun not only due to the additional volume of concrete required to fill in the overbreak, but also for the additional steel volume in the nails to support the weight of the thicker concrete facing. To overcome this, a combined rock nail and rock mesh retention system was adopted where a composite action provided restraint for both global and local face stability. Detailed assessment was necessary to determine the interaction between the local rock mesh facing support and the global support afforded by the nails and considerable effort was made to develop an installation procedure allowing construction of the system to be undertaken safely and efficiently. The final rock nail/rock mesh solution minimised the amount of steel and concrete required to support the rock giving a more sustainable solution than originally proposed.

The construction of the rock cut involved the installation of about 400 rock nails and was completed in 2 months at the end of 2011.

Keywords: rock mesh, Newer Volcanics basalt, rock mass, rock nail, face stability, global stability

1 INTRODUCTION

The Western Ring Road (M80) was constructed in stages between West Gate Freeway/ Princes Highway and Greensborough starting in 1989 before being completed in 1997. The section between the Tullamarine Freeway and Sydney Road included a battered rock slope leading towards a bridge over Moonee Ponds Creek and was completed in 1992. Traffic growth soon resulted in this section of road becoming a bottleneck and widening of this road section by adding 4 eastbound lanes to the north of the alignment (Figure 1) commenced in 2009. This required cutting into the toe of an existing 17 m high 2H:1V batter slope creating a 300 m long vertical rock face of up to 9 m in height with the upper batter remaining untouched. The finished cut is faced by precast fascia panels which also provide the necessary safety barrier at the crest.

This upgrade project was undertaken between 2009 and 2012 under an Alliance partnership between VicRoads, Thiess, Parsons Brinckerhoff and Hyder Consulting (the TullaSydney Alliance). Douglas Partners (DP) was requested to provide geotechnical design services to the Alliance.

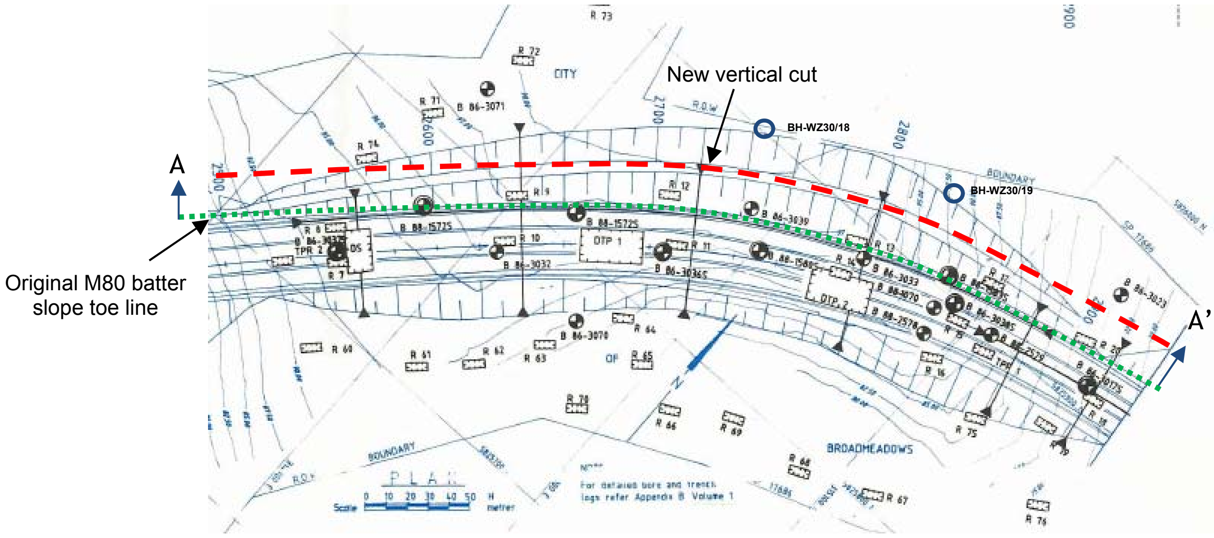

Figure 1. New vertical rock cut and site investigations

2 GEOLOGY AND ROCK MASS CONDITION

2.1 Newer Volcanics

Extensive investigation that mainly consisted of core drilling and exploratory dozer trenches and seismic refraction survey was carried out for the original construction in 1990 with additional work undertaken by the Alliance in 2010. The investigations showed that the new cutting would be mostly within highly weathered basalt as could be observed on the original M80 batters.

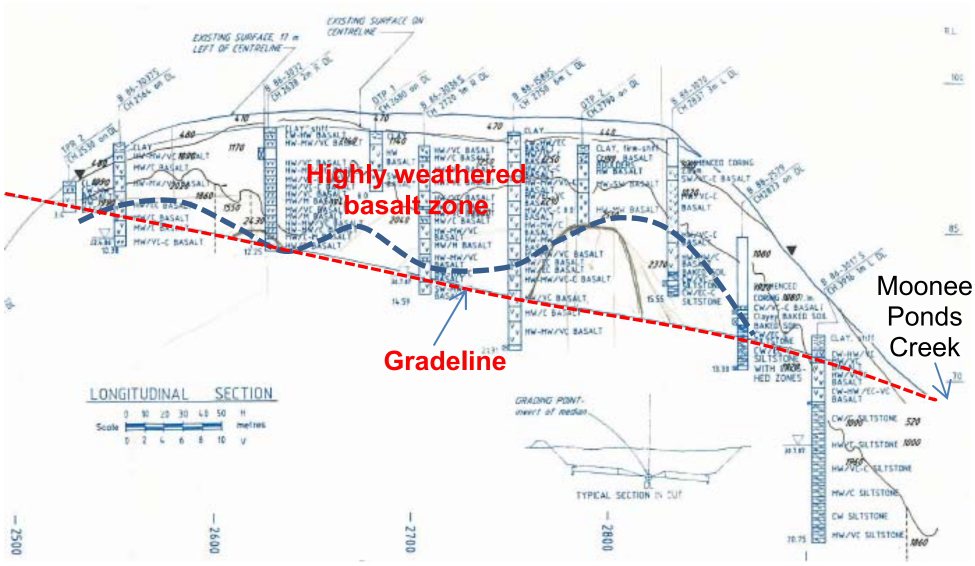

Figure 2. Geological longitudinal Section A-A' showing extent of highly weathered basalt

2.2 Rock Mass Condition

The various investigations showed the basalt to be mainly highly weathered with some moderately to slightly weathered basalt in places. The rock mass is highly fractured and consisted of columnar jointing in several basalt flows with closely spaced sub-horizontal bedding and occasional conjugated 30˚ to 60˚ joints forming a blocky structure ranging in size from 0.01 to 0.3 m^3. The joints were mostly described as being Limonite coated (iron staining), clay coated (smeared) or clay infilled. Some tight and clean joints were also recorded in the boreholes. The joint surfaces were mainly described as either planar and smooth or irregular and rough. The low dipping joints were usually oblique in their dip direction, and the joints were usually curved. Based on this, their persistency was judged as likely to be limited to a few metres. The reported Rock Quality Designation (RQD) values were low with an average RQD for basalt more weathered than ‘moderately weathered’ of less than 10%. The average RQD for the moderately or less weathered basalt was below 50%.

The strength of basalt varied depending upon the weathering condition. Point Load Strength Index (PLI) test results showed $Is_{(50)}$ values of between 0.25 and 4.8 MPa.

3 CONSTRUCTION CONCERNS AND SOLUTION

Prior to the commencement of the design work, the Project construction team had major concern over the concrete faced rock nail face stabilisation method adopted in earlier but smaller cuttings in similar rock conditions where the conventional reinforced concrete facing (shotcrete) was used to provide face stability and had experienced significant overbreak during excavation. The overbreak had not only required additional volumes shotcrete above those anticipated but also additional steel nails to support the weight of the shotcrete until a suitable footing could be established at the base of the wall. The construction team envisaged that similar problems in this large cut would cause appreciable budget and time overruns. To overcome this concern, a composite rock nail/flexible rock mesh solution was proposed and subsequently selected by the Alliance.

4 DESIGN PHILOSOPHY AND METHODOLOGY

The borehole data indicated that the basalt rock mass to be highly fractured and therefore its global behaviour more likely to be controlled by ‘soil’ type mechanisms (i.e. circular slips and face instability). In addition to the global stability, a local surficial sliding wedge failure of the cut face could be formed due to disturbance during the excavation through detachment of rock joints and undercutting.

4.1 Design Philosophy

The design philosophy adopted was to address the two design concerns (global and face stability) using a single composite retention system. Since the primary global failure mode was identified from the prevailing rock mass conditions as a circular slip failure, the design approach was to first validate the adopted design parameters by evaluating the stability of the existing cut batter and then assessing whether the new cut required stabilisation. Where the new cut did not meet the satisfactory long term factor of safety stabilisation measures in the form of the soil/rock nail system were then adopted.

To obtain appropriate face stability, a flexible rock mesh facing was adopted with the nails acting as support for this face retention system. The adoption of the flexible mesh would allow the anticipated uneven rock face to be followed more closely thereby eliminating the need for an undefinable quantity of shotcrete facing as would be required for a rigid mesh.

As the excavation progresses, stress relief movements would lead to the development of nail head forces requiring such forces to be included in the bar capacity assessment in conjunction with the pre-stressing requirements of the facing mesh.

The spacing of the support nails was defined by the strength of the nail bar under the combined loading of the axial force (i.e. that required to tension the mesh and that developed by the stress relief) and the shear developed by the weight of the sliding blocks of face material transmitted through the mesh. For a given nail spacing, the shearing capacity of the face retention is defined by the need to retain the sliding block and break-out resistance of the mesh pinned by nail against wedge shape sliding block. In addition, the axial force in the nail would generate a face load that the facing would need to handle.

Whilst there are several suppliers of rock mesh retention systems, the system supplied by Geobrugg was adopted for the project. The mesh system comprises a high tensile steel mesh TECCO 65/3 with a yield stress of 1700 MPa draped over the rock face and held in place by a system of ‘spike’ plates with a staggered bolt pattern (see Figure 3). A geofabric was placed behind the mesh to control the potential for small fragments to pass through the mesh.

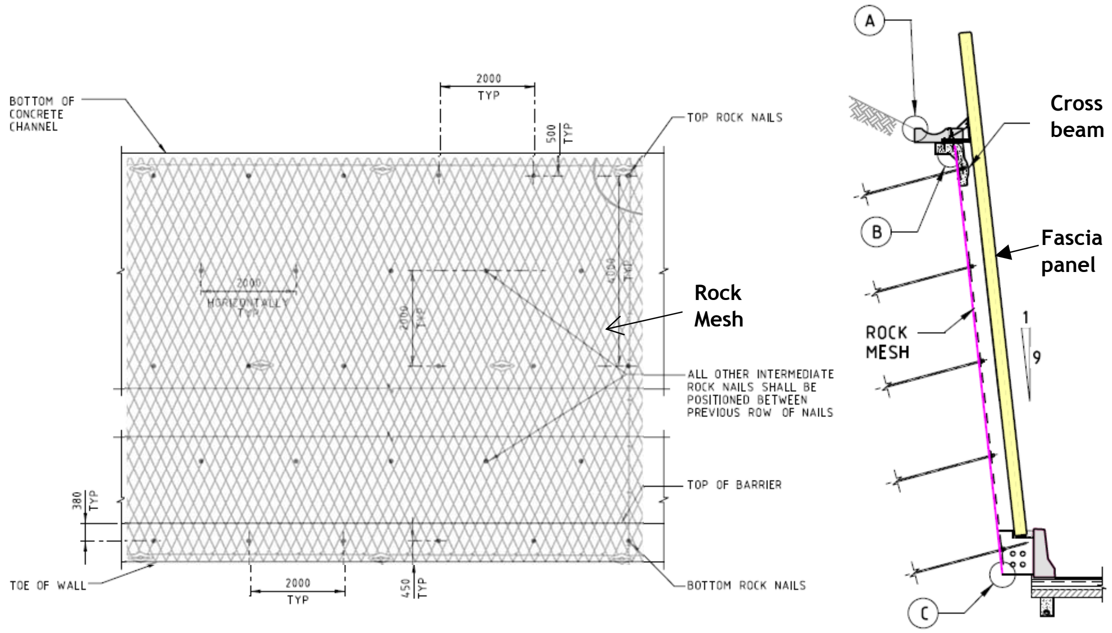

Figure 3. Rock mesh and nail pattern

4.2 Design Methodology

The commercial software program Slope/W (Ver. 2007) was used to perform the numerical global stability calculations. Potential circular failure surfaces were generated using Morgenstern and Price's Simplified Method of Slices. Additional analyses were carried out using the program STARES (University of Sydney). This program employs the modified Bishop method and more easily allowed study of the effects of the limited head restraint that existed with the mesh facing.

For the face stability, the Ruvolum program developed by the mesh supplier was used. The program considers the shallow face failure between the nails (or mesh holding bolts) using a similar failure mechanism assumption to that of wedge theory. Independent assessment was also made to evaluate both circular and wedge failure mechanisms using STARES and force limiting-equilibrium method respectively.

5 DESIGN

5.1 Design Parameters

The adopted rock mass strength parameters were an effective cohesion (c') of 20 kPa and a friction angle (ϕ') of 40˚ based on the General Hoek-Brown failure criterion using the Geological Strength Index (GSI) system (Marinos and Hoek, 2000) for the prevailing rock mass conditions. An ultimate ground-grout adhesion of 200 kPa was used for the nails. A friction angle of 35˚ was adopted for the surficial disturbed rock mass zone in analysing the face retention. The thickness (t) of the disturbed rock mass zone was assumed to be 1 m based on the field observations and sensitivity studies.

5.2 Primary Stabilisation - Rock Nails

The global stability analyses in terms of circular slip failure indicated that the rock cut required 8 m long, 28 mm diameter 500 MPa yield stress bars installed in 125 mm diameter dry drilled holes on a 2 m x 2 m vertical and horizontal grid (see Figure 3). A maximum nail head force in the order of 100 kN was estimated.

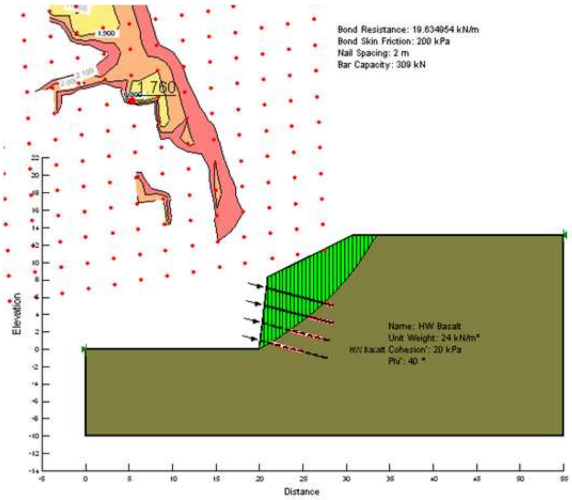

Figure 4. Global stability analysis for 30B cut of highly fractured rock mass

5.3 Face Retention - Rock Mesh System

The face instability involves the superficial slope-parallel slip failure surfaces. The stabilisation treatment system adopted is an active stabilisation approach using high tensile steel rock mesh to retain the disturbed rock mass pinned and secured by the rock nails. The stability of the rock mesh and bolt system requires checking of the bar capacity of rock bolt in shear due to the sliding body (Figure 5a) and break-out of the mesh at the interface with the nails (Figure 5b) due to the force from a wedge shaped sliding body between individual nails (Roduner et al, 2010).

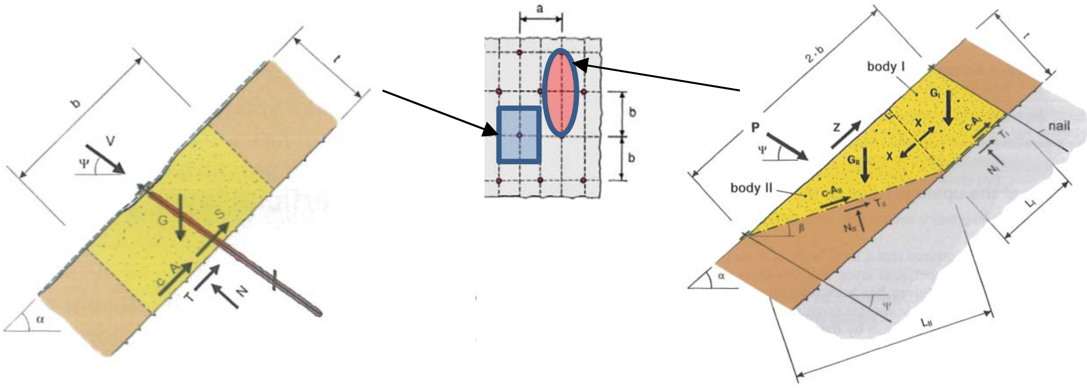

Figure 5a. Forces acting on sliding body

Figure 5b. Sliding of wedged shape body and 'two-body' mechanism

5.3.1 Nail Bar Shear Capacity – Sliding Force

The resultant shear force acting on the bar shared by a single bar is generated from a sliding body with a thickness (t) against the base of stable zone (Figure 5a). The shear force (Sd) can be estimated by:

$$Sd = G\ sin\alpha - V\ cos(\Psi + \alpha) - c'A - [G\ cos\alpha + V\sin(\Psi + \alpha)]\ tan\varphi \qquad (1)$$

where G is the weight of the sliding body, V is the nail pretension force, c' is the cohesion, A is the base contact area, α is the slope angle, Ψ is the nail inclination angle and φ is the friction angle of the disturbed rock mass.

5.3.2 Mesh/Nail Capacity – Wedge Break-out Force

For a single wedge shaped sliding mechanism (Figure 5b), the break-out force (Pd) at the intersection between the mesh and the nail can be determined from:

$$Pd = \frac{G\,[sin\beta - cos\beta\ tan\varphi] - Z\,[\cos(\alpha-\beta) - \sin(\alpha-\beta)\ tan\varphi] - c\prime A}{\cos(\beta+\Psi) + \sin(\beta+\Psi)\ tan\varphi} \qquad (2)$$

where β is the angle of the sliding wedge and Z is the frictional force from the rock mesh against the sliding body.

A two-body sliding mechanism where upper sliding body imposing an active pressure against the lower sliding wedge (Figure 5b) was also considered in the assessment. The contact force (X) from the upper body onto the lower wedge shaped body is:

$$X = GI(sin\alpha - cos\alpha\ tan\varphi) - c'AI \qquad (3)$$

The break-out force (Pd) on the lower wedge shaped body due to the contact force from the upper body is:

$$Pd = \frac{GII\,[sin\beta - cos\beta tan\varphi] + (X - Z)[\cos(\alpha-\beta) - \sin(\alpha-\beta)tan\varphi - c\prime AII}{\cos(\beta+\Psi) + \sin(\beta+\Psi)\ tan\varphi} \qquad (4)$$

The larger Pd force from both mechanisms would be the most critical case.

5.3.3 Results

The most critical load component of the steel mesh system was found to be the shear force imposed on the bar due to the parallel sliding body rather than the break-out force as shown in Figures 6a and 6b.

For the sliding force (Sd) of a body associated with a 2 m grid and a disturbed rock mass thickness of 1 m, the 28 mm nail bar was found to be satisfactory in terms of shear capacity (Figure 6a). The break-out force (Pd) was found to reach a maximum when the thickness of sliding body (t) was 0.6 m.

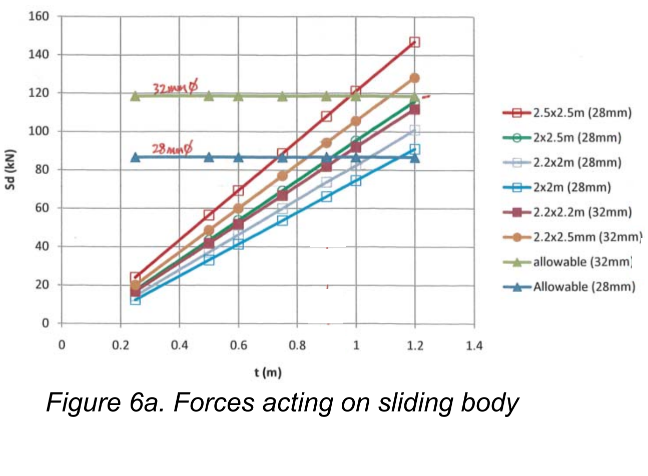

Figure 6a. Forces acting on sliding body

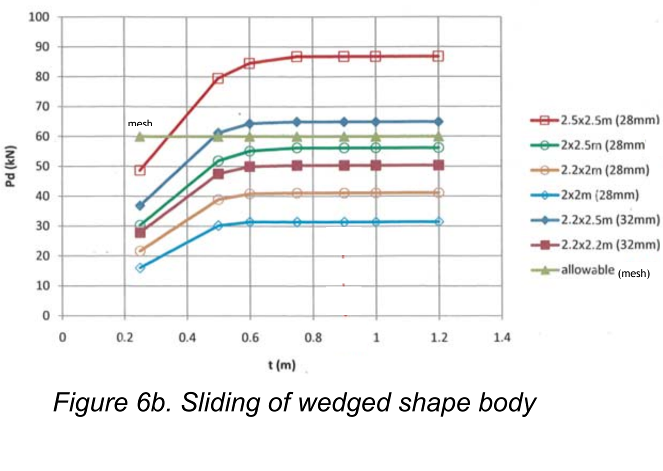

Figure 6b. Sliding of wedged shape body

In order to allow a head force to be developed and to retain the mesh, a head plate is required to bear against the batter face. Whilst the mesh supplier recommended for the retention bolt to be tensioned to between 30 kN and 50 kN the additional loads developed due to mass movement of the slope during excavation the head plates were designed for a head force of 150 kN.

6 CONSTRUCTION

Considerable effort was put into construction staging to minimise the need for working at height and the associated risks. As the final face was to be covered by the precast architectural panels, a support beam was needed at the crest of the cut as a fixing point and it was used to provide an upper fixing point for the mesh (Figure 7a). By constructing this beam when the excavation cut was about 1 m deep, it was possible to roll up the mesh and hang it from the beam and subsequently unrolling the mesh and attaching it to the face progressively at the cut deepened.

The full excavation and installation of about 400 nails and the mesh was completed in 8 working weeks using an average of 2 drill rigs at one time (Figure 7b).

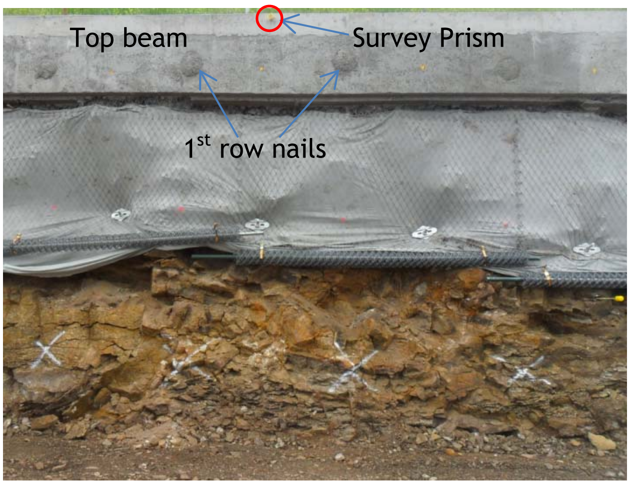

Figure 7a. Top beam with rock mesh secured and pinned with a geotextile underlay


Figure 7b. Final excavation level

Wall displacement was monitored at the top of the wall (Figure 7a) from the commencement of construction primarily for construction control. The maximum measured displacement was recorded as 10 mm or less than about 0.1 % of the wall height which was well within the expected limits.

Testing by pull-out tests on 6 sacrificial nails and proof tests on 2% of the production nails were carried out prior and during the construction respectively.

7 CONCLUSIONS

The application of the combined rock mesh/nail system was found to be suitable for the highly fractured rock mass conditions and the anticipated failure modes. The rock mesh system for face retention of fractured rock mass was successfully integrated into the primary stabilisation using rock nails for global stability. Implementation of the rock mesh presented a safer working environment than would be for a more conventional shotcrete faced wall by eliminating the need for large areas of unsupported rock face prior to shotcrete application. Careful preplanning developed a construction strategy that eliminated the need for expensive working at height safety measures.

8 ACKNOWLEDGEMENTS

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