

# Application of soil nail wall to roadway widening using GFRP rebars as per Australian design guidelines

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## ABSTRACT

This paper presents the application of soil nail wall technology to roadway widening. An arterial road upgrade project in Melbourne consists of widening the freeway in the northern part of Melbourne, including massive cuts into a hillside on the southern side of the existing freeway. Cuts up to 12 meters necessitated the use of retaining walls at 1H:10V batter to stay within the right of way. A soil nail retaining wall was adopted to facilitate reduced excavation, less impact on the existing slope, and improved construction speed with a top-down process. The soil nail retaining wall is 520 m long with a maximum height of approximately 12 meters, including the undercut for pavement and drainage. The ground comprises clay fill overlying residual clay and subsequent weathering profiles of basalt from Newer Volcanics group, partly overlying Brighton Group sediments. Eleven boreholes were drilled sufficiently below the bottom of the wall. Laboratory tests were conducted to estimate the soil and rock strength, including triaxial compression tests with pore pressure measurement to determine effective strength parameters for Brighton group clayey soil. The design employed Glass Fibre Reinforced Polymer (GFRP) bars to enhance the work efficiency by removing encapsulation of steel bars, the durability of which was reviewed for the 100 year design life with the proven data provided by the manufacturer. The soil nail retaining wall was designed as per AS5100.3 and AS4678 guidelines selectively depending on the importance of the wall with reference to VicRoads Specification Section 683 and FHWA-NHI-14-007.

*Keywords:* retaining wall, GFRP permanent nail, roadway widening, Australian design guidelines for soil nail walls

## 1 INTRODUCTION

An arterial road upgrade project in Melbourne consists of widening the freeway in the northern part of Melbourne, including massive cuts into a hillside on the southern side of the existing freeway. Cuts up to 12 meters necessitated the use of retaining walls to stay within the right of way. The reference design provided a reinforced concrete wall on piles to form the retaining wall, which provokes construction risks due to significant excavation to be committed adjacent to the live road. Comparing various retaining wall options, the detailed design adopted a soil nail retaining wall in light of several advantages over other earth retention systems, including reduced excavation, less impact on the existing slope, and improved construction speed with a top-down process. The soil nail retaining wall is 520 m long with a maximum height of approximately 12 meters, including the undercut for pavement and drainage. The ground comprises clay fill overlying residual clay and subsequent weathering profiles of basalt from Newer Volcanics group, partly overlying Brighton Group sediments.

Eleven boreholes were drilled sufficiently below the bottom of the wall along the wall alignment. Test pits were dug to inform the depth of unsupported cut. Laboratory tests were conducted to estimate the soil and rock strength, including triaxial compression tests with pore pressure measurement to determine effective strength parameters for Brighton group clayey soil. The design employed Glass Fibre Reinforced Polymer (GFRP) rebars as opposed to steel bars, which had been used for most soil nail walls in roadway projects, to

enhance the work efficiency by removing encapsulation of steel bars. The durability of GFRP rebars was reviewed for the 100-year design life with the proven data provided by the manufacturer. The soil nail retaining wall was designed as per AS5100.3 and AS4678 guidelines selectively depending on the importance of the wall with reference to VicRoads Specification Section 683 and FHWA-NHI-14-007. This paper addresses the implication of Australian design guidelines for soil nailed walls using GFRP bars, soil nail bond strength determination, the impact of excavation on adjacent existing structures, and the integration of road barriers into the wall.

## 2 SITE CONDITIONS

### 2.1 Ground conditions

The ground conditions have been inferred from eleven boreholes drilled along/close to the wall alignment up to a depth equal to the wall height below the base of the wall. Six test pits were dug up to 3 m deep, which informs the design of unsupported excavation. A review of geological maps of the area and the available geotechnical information indicates that the soils and rock of the Quaternary age Newer Volcanics are underlain by Tertiary age sediments.

Boreholes drilled on-site demonstrate ground conditions comprising fill varying in depth between 1.5 m and 4.0 m, overlying natural residual soil and basalt of Newer Volcanics Group. The borehole logs long plot is displayed along the soil nail wall alignment in Figure 1. As shown in Figure 1, the boreholes near the

western/eastern end of the soil nail wall indicate that the Newer Volcanics are underlain by cemented sandy clay of Tertiary age sediments, which forms part of the ground behind the wall. The residual clays encountered within these boreholes are of very stiff to hard consistency. The weathering of the basalt varies from highly to moderately weathered with low to high strength. Based on the available geotechnical information, the basalt is identified to be highly jointed and fractured. Existing groundwater data available on-site suggests the groundwater levels are below the base of the soil nail wall.

**2.2 Laboratory soil and rock testing and rock mass quality**

Considering limited information on relationships between effective soil strength parameters and in-situ sounding tests, such as SPT “N” on hard sandy clay of Tertiary age sediments, consolidated undrained triaxial tests with pore pressure measurement were conducted on the undisturbed samples taken from hard sandy clay encountered in boreholes near the western and eastern ends (BH01, 02 and 09). The results of the triaxial tests are displayed in the form of p-q’ plot, as shown in Figure 2.

The effective cohesion tested greater than 40 kPa, indicating the clay is highly over-consolidated throughout the boreholes, with plasticity tests on the samples indicating low plasticity. Observation on rock cores and laboratory test results on basalt core samples indicate the rock mass quality of the basalt pertaining to Class IV Poor rock in Rock Mass Rating by Bieniawski (1989) and Blocky/Disturbed/Seamy to Disintegrated on GSI chart by Hoek (1994) as shown in Figure 3.

Uniaxial compression strength (UCS) of basalt tested between 8 and 35 MPa. The UCS and Point Load Strength (Is50) relationship appeared to be  $UCS = 8 \times Is_{50}$  site-wide.

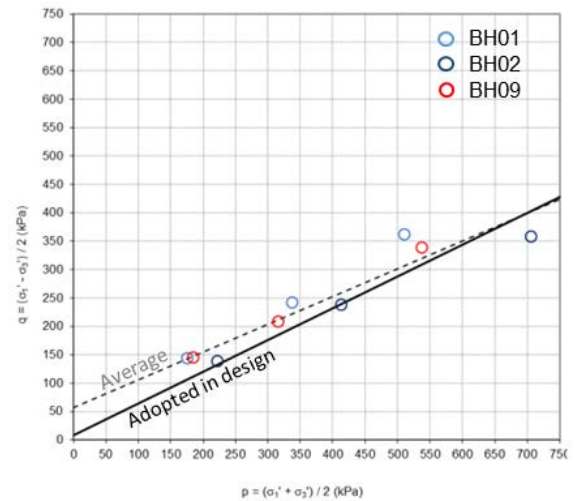


Figure 2. P-q’ plot from CU triaxial tests with pore pressure measurement

GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS	SURFACE CONDITIONS				
	VERY GOOD	GOOD	FAIR	POOR	VERY POOR
STRUCTURE	DECREASING SURFACE QUALITY →				
INTACT OR MASSIVE-intact rock specimens or massive in situ rock with few widely spaced discontinuities	90				
BLOCKY-well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets	70				
VERY BLOCKY-interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets	60				
BLOCKY/DISTURBED/SEAMY -folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity			40		
DISINTEGRATED-poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces				20	
LAMINATED/SHEARED-Lack of blockiness due to close spacing of weak schistosity or shear planes					10

Figure 3. Rock mass quality on GSI chart by Hoek

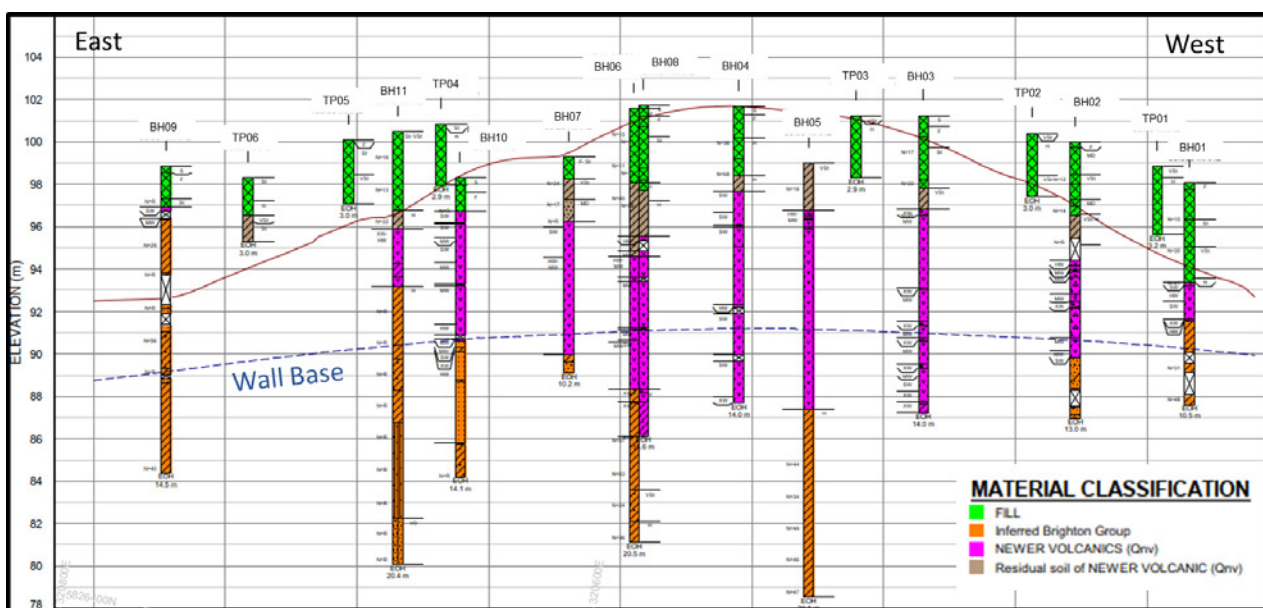


Figure 1. Borehole long plot along the wall

### 2.3 Restrictions Imposed by Adjacent Structures

The wall alignment neighbours existing structures, which imposes constraints, including:

- proximity to the existing transmission tower that is approximately 8 m south of the crest of the proposed soil nail wall for which the asset owner required the design to ensure minimum clearance of 0.5 m between the closest soil nail and the tower footing and maximum vertical tolerance of 5 mm for the transmission tower;
- existing soil nail wall, the face of which needs to be dismantled to tie in with; and
- proximity to the existing bridge at the eastern end of the wall.

Design and construction measures have been made to manage the tolerance of the wall deflection in proximity to the existing transmission tower by complying with the design requirement as per AS5100 and numerically predicting the wall deflection, which has been ensured with the monitoring during the construction. More stringent design requirements out of AS5100 and AS4678 have applied for the design where the wall extends up to 30 m from the existing bridge abutment or existing assets of high importance with reference to VicRoads Road Structure Inspection Manual (2018). The implication of design codes is further discussed in Section 3.2.

In addition, the traffic barriers adjoining the bottom of the soil nail wall shall be designed to act as independent systems or to prove no detriment to the soil nail wall in the event of any collision with the traffic barrier where the barriers and the wall are integrated. To this end, the impact load is considered to be taken by the traffic barrier designed as an independent system to the soil nail wall, hence no damage to the integrity of the wall or load transfer to the wall. This is further discussed in Section 3.3.1.

### 2.4 Other Wall Features

The retaining wall requires a balustrade or other barrier to alleviate the risk of a person falling from the top of it as depicted in Figure 4. The extension at the top of the soil nail wall, which forms a safety barrier upstand, is assumed to be constructed at the same time as the uppermost nails. This barrier upstand is subjected to wind loading and handrail loading from AS5100 CI 12.5 (a-c).

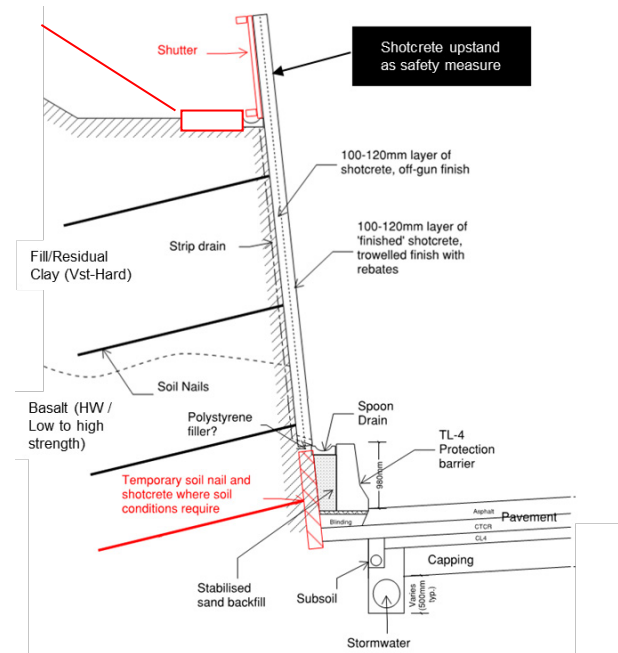


Figure 4. Typical cross-section of soil nail wall featuring barrier and shotcrete upstand

## 3 DESIGN CONSIDERATIONS

### 3.1 Design soil/rock parameters and bond strength

Geotechnical material parameters adopted for design have been derived based on the available geotechnical information, including both in-situ and laboratory tests, and practically accepted correlations in literature (e.g. Burt 2005), as summarised in Table 1. The highly weathered (Hw) to moderately weathered (Mw) basalt strength parameters are determined based on the empirical relations between the RMR class and Mohr-Coulomb failure criterion strength parameters and Mohr-Coulomb strength parameters equivalent to Hoek and Brown strength for rock mass.

The soil nail bond strength was determined from the lesser of empirical bond stress presented in literature such as CIRIA report C637 (CIRIA 2005) and effective stress based estimation employing the effective strength design parameters in Table 1.

Table 1: Geotechnical design parameters

Material	Unit Weight, kN/m <sup>3</sup>	Effective Cohesion, kPa	Effective Friction Angle, degrees	Undrained shear strength, kPa	Young's Modulus, kPa	Soil Nail Ultimate Bond Stress, kPa	Bond stress based on
Existing Fill	19	5	28	75	20	50	Average normal effective stress
Newer Volcanics Residual Clay (Very stiff to hard)	19	10	28	100	30	60	Average normal effective stress
Basalt – Hw to Mw	22	80	32	-	300	300	Empirical adhesive strength
Inferred Brighton Group – cemented sandy clay (hard)	19	15	35	200	60	80	Average normal effective stress

The adhesion between the nail grout and Hw basalt for the rotary-drilled hole was determined with reference to empirical literature charts such as those in FHWA (FHWA 2003). The nail verification pull-out tests conducted during the installation of nails have verified the design ultimate bond stress to be adequate.

**3.2 Soil Nail Wall Design as per AS4678 and AS5100 for Soil Nail Retaining Walls**

The soil nail wall design has been completed predominantly relying on the stability of soil nail walls, which has been analysed using the 2D limit equilibrium software Slide2 by Rocscience and the 2D finite element software PLAXIS 2020 with the strength reduction method employed.

Static stability design has primarily adopted the limit state method with material strength partial factors and load factors adopted in accordance with AS 4678-2002 as summarised in Table 2 and Table 3, achieving a minimum global “factor of safety” (“FoS”) of 1.0 in both short-term (i.e. during construction) and long-term static conditions.

Table 2: Partial Factors as per AS4678

Partial Factor Items	Existing Fill	In-situ Material
Effective friction angle, $\phi_\phi$	0.9	0.85
Effective cohesion, $\phi_c$	0.75	0.7
Undrained shear strength, $\phi_{uc}$	0.5	0.5

Table 3: Load Factors as per AS4678

Partial Factor Items	Factor adopted
Dead load behind wall	1.25
Dead load ahead of wall	0.8
Traffic load behind wall	1.5

The following design reduction factors have been considered for the global stability analysis in as per AS4678-2002 or other standards as stated below:

- Tendon structural capacity reduction factor = 0.4 as per VicRoads Section 683 (06.b.vii);
- Pull-out (bond) resistance reduction factor for rock nails =  $\phi_k \times \phi_b = 0.56$ ;

where:

- Importance category reduction factor,  $\phi_k = 0.8$  (AS4678-2002 Table B1);
- Minimum material reduction factor  $\phi_b = 0.7$  (AS4678-2002 Table B2) for the bond between rock and grout;
- Pull-out resistance reduction factor for soil nail =  $\phi_n \times \phi_b = 0.63$ ;

where:

- Structure classification design factor,  $\phi_n = 0.9$  for structure classification C (AS4678-2002 Table 5.2); and
- Minimum material reduction factor,  $\phi_b = 0.7$  (AS4678-2002 Table B2) for the bond between soil and grout;

The bond capacity within the first 1 m length of nail has been ignored in the design as per VicRoads Section 683.

For the design sections in proximity to important structures, the design has adopted the working stress

method as per AS 5100.3-2017, achieving a minimum global FoS of 1.8 in long-term

Condition. The important structures sections include:

- located within 30 m of the existing road bridge
- located within 30 m of the existing transmission tower.

Seismic stability has been analysed based on the working stress method, achieving a minimum global FoS of 1.2. The stability analysis has taken into account possible construction staging with 0.5 m excavation below each row of nails prior to installing the subsequent row of nails and 1.5 m over-excavation below the pavement level at the end of the excavation included. The short term analysis accounting for ‘during-construction cases’, including those subject to the build-up of temporary water pressure and seismic loading, has been performed employing undrained soil design parameters, which have been switched to effective stress design parameters for the long term stability analysis.

As shown in Figure 5, the long term analysis has been carried out excluding the bottom row of soil/rock nails behind the road protection barrier that is considered sacrificial for collision so that the wall stability is independent of the bottom row soil/rock nails. As a result, the minimum global “FoS” has been calculated to be between 1.1 and 1.4 for the sections designed as per AS4678 and 1.9 for those as per AS5100 in long term conditions.

On the other hand, the strength reduction method in the finite element models has generated the minimum global FoS slightly higher than that from the conventional limit equilibrium method. The working stress method analysis against all the sections designed as per AS4678 or AS5100 has indicated the minimum global FoS greater than 1.5 specified as the design criteria in FHWA-NHI-14-007 and CIRIA report C637.

The nails have been laid out at spacings of 1.5 m in soils and 1.8 m in Hw to Mw basalt with lengths up to 0.9 H at maximum. The hole diameter has been set to 125 mm in soils and 105 mm in Hw to Mw basalt.

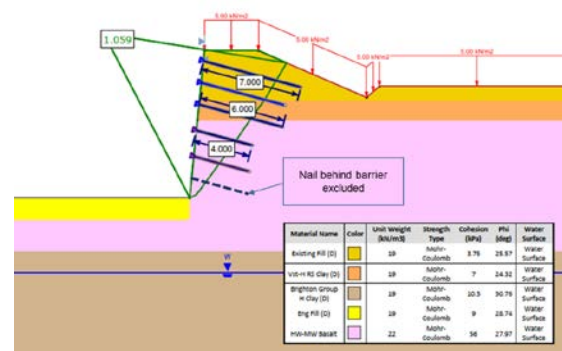


Figure 5. Soil Nail Wall Stability Analysis with the bottom nail excluded

**3.3 Deformation Analysis Considering Interaction with Adjacent Structures**

The Finite Element analysis undertaken has also assessed the deformation of the soil nail walls, including the influence of collision loading and its impact on the existing structures in proximity. The maximum lateral wall deflection has been estimated to be less than 0.3 % of the wall height, which results in insignificant settlement

behind the wall in terms of serviceability. Particular attention has been paid to whether the soil nailing induced ground deformation adversely impacts the structure adjacent to the wall.

### 3.3.1 Collision loading impact assessment

As per the VicRoads regulation on external loads acting on a retaining wall, the road protection barriers in front of the soil nail wall are to be designed to be structurally independent of the soil nail wall from the collision loads. The geotechnical information indicated that the materials behind road barriers consist of Hw to Mw weathered basalt or highly over-consolidated sandy clay. No structural component was required as part of the retaining wall to support the ground after excavation. The facing was assumed to be redundant (i.e. no replacement required) behind the barrier in the event of a vehicle collision. A joint placed in the facing behind the top of the barrier split the facing into two pieces structurally. A 25 mm thick compressible layer was also included to prevent the load transfer from the barrier to the shotcrete. The road protection barriers have been placed without an extended foundation, piled or slab footing where the assumed design ground condition is present.

The impact of collision loading has been assessed using PLAXIS 2D for the design sections. The road protection barrier was modelled with concrete properties embedded 100 mm into the finished road level. The result has shown no sign of local instability and a negligible increase in either ground deformation or working load in nails.

### 3.3.2 Asset impact assessment

A 220 kV transmission tower has been identified where the wall height becomes the highest. With the proposed wall slope of 1H in 10V, the closest tower leg is approximately 8 m behind the crest of the soil nail wall. The available information on the tower has informed that each foot of the transmission tower is founded on a circular trapezoid pad footing with the upper surface diameter of 1.1 m and the base diameter of 1.4 m at 2.6 m below ground surface at the time of construction. Due to proximity to the soil nail wall, the transmission tower might be subject to ground movement induced by the excavation during soil nail wall construction. The asset owner of the existing transmission tower requested the conditions described in Section 2.3 to be met. The PLAXIS 2D model developed for asset impact assessment is presented in Figure 6. The material underneath the tower footings is assumed to be residual soil clay with very stiff to hard consistency. Two transmission tower footings with 6.7 m apart centre to centre are included in the analysis.

Due to the uncertainty of the transmission tower loads on the footing, an allowable bearing pressure of 440 kPa on residual clay has been taken as the footing foundation load. The upper row of soil nails is 6 m long at an inclination of 15°, maintaining sufficiently more than 500 mm from the tower footing, which has been accepted by the asset owner.

The PLAXIS analysis result has shown the maximum differential movement under the footings less than 3 mm across the neighbouring footings, which is within the tolerance of 5 mm.

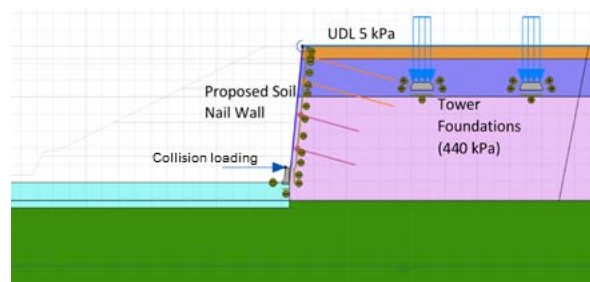


Figure 6. Soil Nail Wall Stability Analysis with the bottom nail excluded

### 3.3.3 Interference with Existing Nails

The presence of existing soil nails was expected to introduce interface constraints with the soil nails proposed at the section tying in to the existing soil nail wall. PLAXIS 2D analysis was carried out to assess the impact of removing the existing soil nail walls based on the information shown on the historical design drawings. The existing soil nails exposed within the excavation for each lift were assumed to be cut prior to the installation of the new soil nails, with the existing soil nail behind the new wall remaining in place without ground disturbance and a new row of soil nails installed.

The critical section comprising the tallest heights of the retaining walls (existing retaining wall height of 6.6 m and new retaining wall height of 8.8 m) was modelled, as shown in Figure 7. The result of the impact analysis using PLAXIS 2D has indicated a low likelihood of disturbance to the stability of the soil nail walls by the removal of the existing nails in a sequential manner.

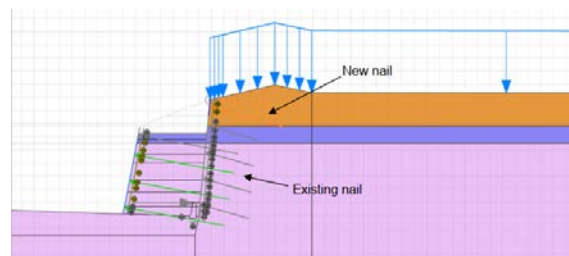


Figure 7. Soil Nail Wall Stability Analysis with the bottom nail excluded

## 3.4 Facing Design

The wall facing comprises double shotcrete layers - structural facing overlain by aesthetic facing. The structural facing has been designed in accordance with FHWA-NHI-14-007. The working stress-based PLAXIS analysis outcomes have informed the structural capacity assessment with the employment of a lumped load factor of 1.5 as per AS5100.3.

## 4 USE OF GFRP BARS

### 4.1 Longevity of GFRP Bars

The lightweight properties of GFRP rebars make handling and installation easy and safe compared with conventional steel rebars. Their inherent non-corrosive properties have additional benefits from a durability perspective, as no additional corrosion protection is required (CIRIA 2005). Although the application of GFRP rebars to permanent soil nail walls is yet to be long

enough to attest to the longevity over a 100-year design life, indicative testing conducted by others has suggested that vinyl ester resin GRP should have a long life in a cementitious environment (Johnson 1998). Mechanical testing to inform the 100-year design life is usually based on creep rupture testing. This testing is used to estimate indicative sustainable loads over a period of 106 hours (>114 years) assuming the linear degradation of the ultimate tensile strength (UTS) of the GRP due to creep. This testing is undertaken in the absence of embedment material around the soil nails. The indicative performance is therefore based on an allowable applied load on the GRP bars of less than the creep rupture strength estimates at 100 years and the indicative performance of the Vinyl Ester resin GRP in the cement mortar bedding.

The vinyl ester resin GRP Bluey BluGeo GRP60-20 mm bar was identified in the testing performed by Mohamed and Benmokrane (Mohamed and Benmokrane 2018) to have an indicative 106 hour creep rupture strength of 58% of the original UTS. Theoretically, where the load is applied less than the 106 hour creep rupture strength and the mortar and soil nail bar bond is maintained over the design life, the properties of the GRP are not considered unduly affected by the environment, which the soil nail should then have adequate durability for the nominated 100-year design life. In addition, according to the manufacture information, the product should comply with VicRoads Section 683.

#### 4.2 Implication of GFRP bars on Soil Nail wall design

For the sake of comparison between GFRP and conventional steel nails as to their implication to the design of soil nail wall, PLAXIS2D and SLIDE models have been analysed over one of the design sections with respect to the minimum global FoS, maximum horizontal wall displacement, and maximum axial nail force adopting the parameters tabulated in Table 4.

Table 4 Properties of GRP and Steel

Items	BluGeo GRP60	DSI GEWI® Threadbar
Diameter (mm)	25	20
Young's Modulus (GPa)	60	200
Ultimate Tensile Strength (kN)	350	157
Unit Weight (kg/m)	0.9	2.47
Structural Reduction Factor	0.40 <sup>(1)</sup>	0.648 <sup>(2)</sup>

<sup>1</sup> As per VicRoads Standard Specification 683

<sup>2</sup> As per AS 4678-2002 Earth-retaining structures

The use of GFRP nails in lieu of conventional steel nails results in the following:

- decreases in axial nail force up to 30% due to lower elastic modulus of GRP nails than that of steel nails.
- marginal increases of soil nail wall horizontal displacement up to 7% due to the lower elastic modulus of GRP nails than that of steel nails.
- no change in the factor of safety against wall instability.

## 5 CONCLUSIONS

A soil nail wall up to 12 m high has been successfully designed and constructed adjacent to one of the busiest arterial roads in the Greater Melbourne area as part of a freeway widening project. The application of soil nailing utilising GFRP rebars to the retaining wall has provided significant benefit to the project from either the design or construction perspective, including:

- Meeting the design requirements with innovative material replacing conventional steel rebars
- Relieving burden on the design concerning corrosion of conventional steel rebars
- Minimising disruption to the adjacent live arterial road by reducing temporary earthworks, which is facilitated by top down staging of soil nailing
- Speeding up the nailing installation progress, leading to construction time saving
- Enhancing work efficiency by reducing the burden to handle heavy steel products and encapsulating steel rebars

Lastly, although the case study presented demonstrates the applicability of GFRP soil nails for use in retaining walls with significant benefit design and construction-wise, the use of GFRP nails will be better convinced with further research on the long term performance in the future. Monitoring the wall profile at regular intervals over a long period may help understand the long term behaviour of the wall reinforced with GFRP rebars

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