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Enhancing existing rail corridor by track lowering – a case study

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ABSTRACT: Inland Rail is a 1,600km freight rail line that will connect Melbourne and Brisbane via regional Victoria, New South Wales and Queensland. Enhancement sections of the rail corridor would provide the necessary vertical and horizontal clearance along the existing 'Main South Line' corridor to support safe running of double-stacked freight trains. The works include track slews and reconfigurations, track lowering, signaling gantry replacements, level crossing modification, bridge modifications/replacement and associated works. This paper focuses on the track lowering works at the Billy Hughes Bridge in Albury, NSW, where the rail level is being lowered by approximately 1.6 meters to provide the necessary clearance under the bridge. A key feature of the project is the design and construction of a deflection wall between the existing bridge piers and the rail line to safeguard the bridge against potential train collision events. This case study highlights the geotechnical assessment; design challenges and innovations associated with the deflection wall and the mitigation measures implemented to address potential impacts on the existing bridge piers and adjacent gabion walls caused by track lowering works.

KEYWORDS: Inland Rail, rail enhancement, deflection wall, track lowering, track slew

1 INTRODUCTION

Inland Rail is a fast freight backbone that will transform how goods are moved around Australia, generating opportunities for our regions and our economy, now and well into the future.

The Albury to Parkes (A2P) section of Inland Rail involves extensive enhancements to specific sites across the 355 kilometers of existing track running from Albury to Illabo and Stockinbingal to Parkes (Figure 1). The enhancements would provide the necessary vertical and horizontal clearance along the existing 'Main South Line' corridor to support the safe running of double-stacked freight trains up to 1,800 meters long and 6.5 meters high. Billy Hughes Bridge is located on the Wagga Road over the Rail line, within Albury, NSW. The existing track level will be lowered by approximately 1.6 m to allow for the travel of double-stacked freight trains, under the Billy Hughes Bridge.

Figure 1. Inland Rail route map

The Billy Hughes Bridge package forms part of the Albury to Illabo (A2I) section of works, with the proposed track lowering works located approximately between Chainage 634+640 km to Chainage 635+270 km, to enable the Plate Structure Outlines D, H and F2 to Plate D clearance (Plate F2M). In addition to track lowering, track slewing is also required to provide sufficient clearance for Inland Rail traffic (Figure 2 and Figure 3).

In order to maintain the stability of the existing reinforced soil walls and the Bridge structure, new independent deflection walls are required at either side of the track. An interim track alignment is required to enable construction of the deflection and protection walls.

Based on the new track alignment, construction of the new formation, earthworks and a drainage network are also required.

This paper presents the Geotechnical interpretation of the Billy Hughes Bridge site and the summary of the geotechnical design of associated structures. In addition, geotechnical assessment of soil walls to support excavation in front of the existing gabion walls. Geotechnical assessment and design of deflection walls to protect existing bridge piers.

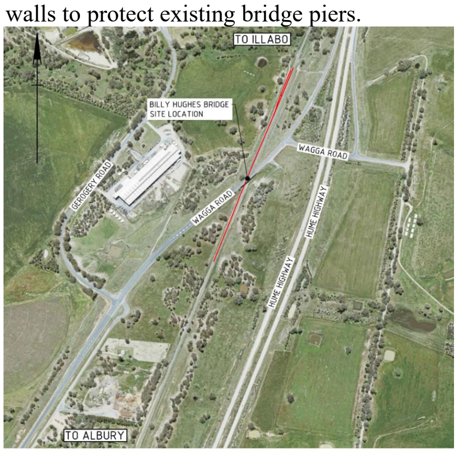


Figure 2. Locality Plan

Figure 3. Billy Hughes Bridge with track section to be lowered

2 SITE CONDITIONS AND GEOLOGICAL SETTINGS

2.1 *Site conditions*

Billy Hughes Bridge is located over 10 kilometres north-east of Albury Railway Station on the border of Table Top and

Ettamogah. Billy Hughes Bridge crosses over the Main Southern Railway Line as an overpass for Wagga Road that connects to on and off ramps for the Hume Highway and residential roads.

2.2 *Geological settings*

NSW Seamless Geology, Version 2.3 (2023) shows that the site is underlain by Quaternary colluvium, which comprises poorly sorted, weakly cemented to unconsolidated colluvial lenses of polymictic conglomerate with medium- to very coarse-grained sand matrix; interspersed with unconsolidated clayey and silty red brown (aeolian) sand layers, modified by pedogenesis.

3 DEFLECTION WALL

A deflection wall will be constructed between the bridge piers and rail line in order to protect the bridge from a potential train collision event. The proposed wall alignment is shown in Figure 4.

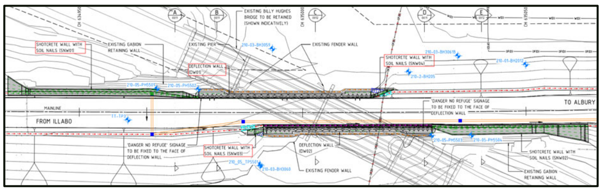


Figure 4. Deflection wall and soil nail wall alignment at the Billy Hughes bridge site

The deflection wall comprises 600mm diameter bored piles spaced at 1.5m centers. The first six piles at each end of the wall are spaced at 1.0m centers. The piles are connected with a 1.5m wide (edge) or 1.0m wide (center) and 1.3m deep capping beam. Figure 5 shows a sketch of the typical pile arrangement. 2m height backfill (above the formation level) should be placed behind the deflection wall to provide additional passive resistance to the cantilever portion of the piles.

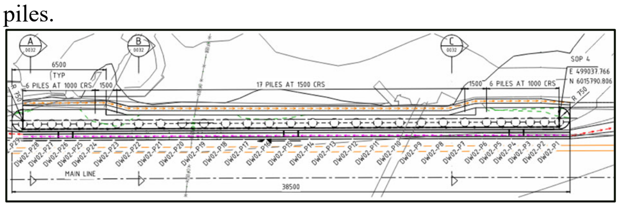


Figure 5. Typical deflection wall Arrangement

3.1 *Design methodology*

A summary of the adopted design methodology is presented in Table 1.

Table 1. Summary of geotechnical design methodology for foundations

Design Element	Description
Geotechnical strength reduction factor (φg)	The estimated design average risk rating (ARR) in accordance with AS2159-2009 is 3.1. The geotechnical strength reduction factor (ϕ_g) = 0.56 has been adopted in the current design, corresponding to a high redundancy system where more than 4 piles are used to resist the loads.
Design loading	Head-on collision load equal to 4000kN parallel to the wall alignment and 1500kN perpendicular to the wall has been adopted for the design of end piles in each deflection wall. The head-on collision load is applied over a 2m long and 0.5m deep area of the capping beam, 2m above the track level. In addition, a critical collision loading case of 12,000kN, parallel to the end wall alignment has also been analyzed. Middle sections of the deflection walls that are not subjected to head-on collision load were designed for the minimum collision load equal to 1500kN perpendicular to the wall.
Design methodology – Deflection wall piles	Per AS2159-2009 requirements, pile lengths were proportioned such that the design geotechnical strength ($R_{d,g}$) was not less than the design action effect (E_d), with $R_{d,g}$ calculated as the design ultimate geotechnical strength ($R_{d,ug}$) multiplied by a geotechnical strength reduction factor (ϕ_g), as follows: $R_{d,g} = \phi_g . R_{d,ug} \geq E_d$ The general procedure for the deflection pile design was as follows: Pile reactions were obtained using Plaxis 3D, incorporating the prevailing geotechnical conditions, Ø600mm piles and applying the ULS train collision loading at the end of the deflection wall. Passive resistance due to the backfill was considered for the pile behavior perpendicular to the wall. Passive resistance was neglected in the direction parallel to the wall. Pile lengths were proportioned to satisfy the limit state design requirements per AS2159-2009. Pile actions were sent to structural engineers to confirm the structural capacity of the piles. The pile design is governed by pile shaft structural capacity.

Geotechnical design is carried out using Plaxis 3D with the modelling of the adjacent bridge piers. The adopted model is shown in Figure 6 below.

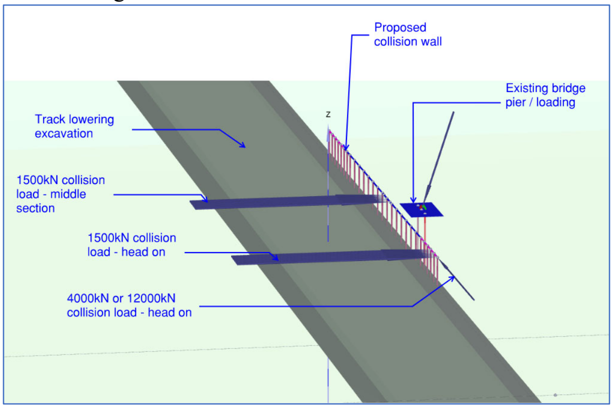


Figure 6. PLAXIS 3D Model for Collision Wall Assessment

3.2 *Design outcome*

Summary proportioned piles are presented in Table 2.

Table 2. Proportioned pile elements

Pile location	Pile diameter (mm)	Pile spacing (m)	Pile Top RL (m AHD)	Top of capping beam RL (m AHD)	Pile length (m)
Deflection walls – End piles	600	1.0	218.7	219.7	15.0
Deflection walls – Middle piles	600	1.5	218.7	219.7	10.0

Summary of estimated maximum pile actions are presented in Table 3.

Table 3. Assessed pile actions for collision ULS load case

Location	Pile length (m)	Max. Axial load (kN)	Max. bending moment (kNm)	Max. shear force (kN)	Remark
1	10.0	100 (compression)	261	345	Stability satisfactory with 14mm movement
2	10.0	500 (tension)	395	639	Stability satisfactory with 42mm movement
3	15.0	939 (tension)	590	689	Stability satisfactory with 91mm movement
4	10.0	136 (compression)	560	941	Stability satisfactory with 61mm movement

Location 1: Deflection walls – Middle piles under 1500kN Collision
Location 2: Deflection walls – End piles under 4000kN and 1500kN Collision
Location 3: Deflection walls – End piles (Pile No.1) under 12000kN and 1500kN Collision
Location 4: Deflection walls – End piles (Pile No.5) under 12000kN and 1500kN Collision

3.3 *Impact to existing bridge pier*

An analysis of impact of the deflection wall construction (including installation of the pile wall and excavation for track lowering), and in the case of collision, on the existing bridge structure has been carried out. The cross section illustrating the interface is shown in Figure 7 below.

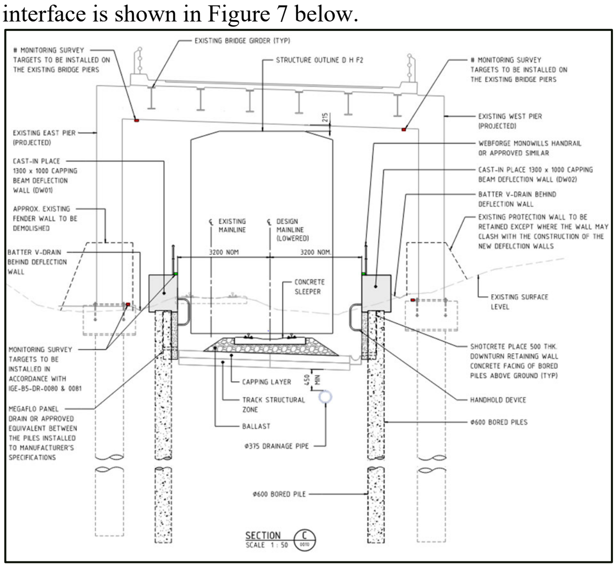


Figure 7. Section through bridge pier

3.3.1 *Finite Element Modelling Inputs*

The assessment was conducted based on finite element modelling using commercially available software Plaxis 3D as illustrated in Figure 6.

The following construction stages are adopted:

a) Initial condition
b) Construction of the bridge pier and bridge piles
c) Application of the bridge loads
d) Construction of the collision wall
e) Excavate to construct the proposed track lowering
f) Application of the collision loads

In the assessment the flexural stiffness reduction is applied to 40% to account for the cracked concrete sections in long-term.

3.3.2 *Finite Element Modelling Results*

The assessment has involved 1) the assessment of impact of bridge loading on the collision wall, and 2) the assessment of the collision loading impact on the bridge pier during a collision event.

The impact of the bridge loading on the collision wall is shown in Figure 8. As might have been expected, the induced bending in the collision wall due to the bridge loading is far less than that during a collision event and the assessed collision wall movement is less than 0.5mm. Hence it is concluded that the impact of the bridge loading on the collision wall is insignificant.

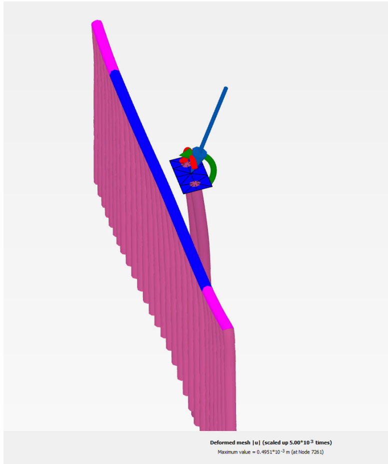
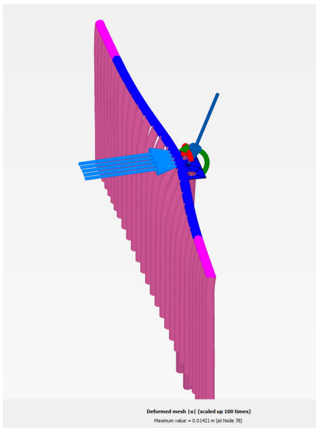
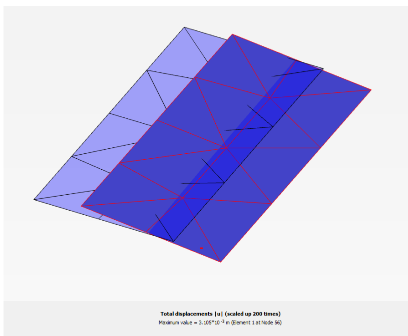
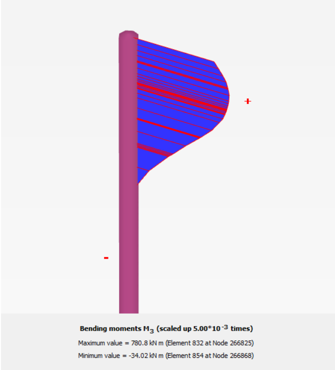


Figure 8. Impact of Bridge Loading on the Collision Wall

The impact of the collision loading on the bridge substructure is shown in Table 4.

Table 4. Summary of impact of the collision loading on the bridge substructure

Illustration	Results of assessment
	Deformed shape showing the potential impact of collision load on the bridge pier.
	The induced bridge pile cap is assessed to be 3.2mm, less than the assessed movement that the cap has experienced due to bridge loading (13mm)

	The induced bridge pile bending is assessed to be 780.18 kNm which is less than the pile capacity of 820 kNm.

3.4 *Excavation induced movement*

An analysis of excavation for the track lowering and the impact on the deflection wall and existing bridge structure has been carried out.

3.4.1 *Finite Element Modelling Inputs*

The assessment was conducted based on finite element modelling using commercially available Plaxis 3D as illustrated in Figure 6.

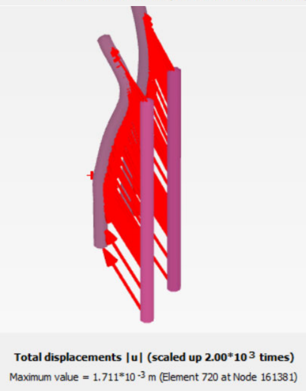
The following construction stages were adopted:

a) Initial condition
b) Construction of the bridge pier and bridge piles
c) Application of the bridge loads
d) Construction of the deflection wall
e) Excavate to construct the proposed track lowering

3.4.2 *Finite Element Modelling Results*

The displacement in front of the collision wall and existing bridge structure is shown in Table 5.

Table 5. Summary of track lowering excavation impact on the deflection wall and existing bridge structure

Illustration	Results of assessment
	The excavation induced movement on the deflection wall is 14.8mm.
	The excavation induced movement on the existing bridge pier is 1.7mm.

The results of the assessment indicate that the excavation induced movement on the deflection wall and the existing bridge pier is considered to be within acceptable limits.

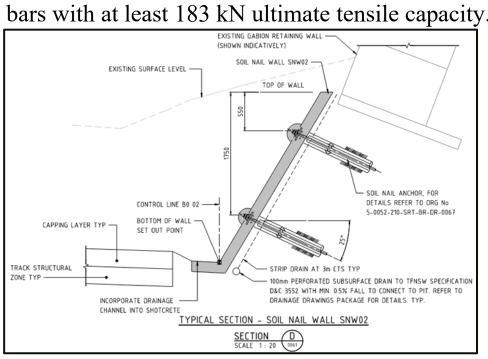
4 SOIL NAIL WALL

4.1 *Soil nail wall below gabion wall*

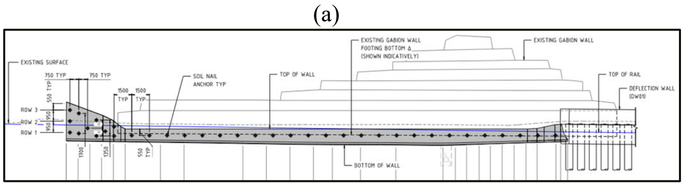
Two Gabion walls are located on both south and north sides of the bridge approach as shown in Figure 4. The lowering of track levels in front of the Gabion walls requires a retaining system to support the base of the existing gabion wall and to maintain the stability of the overall slope. The extent of the soil nail wall is also shown in Figure 4.

The arrangement of proposed soil nail wall is as follows. Figure 4 7a and Figure 4 7b provide sketches of the soil nail configuration.

- Maximum excavation depth = 2.5 m.
- Two rows of 10m long soil nails.
- Horizontal spacing between nails = 1.5m
- Vertical spacing between nails = 0.8m to 1.2m
- Nail inclination 25 degrees.
- Nail type – minimum bond diameter of 170mm and soil nail bars with at least 183 kN ultimate tensile capacity.



(a)



(b)

Figure 9. Configuration of soil nails – (a) Cross section, (b) Front elevation

4.1.1 *Design methodology*

Table 6. Design methodology

Design Element	Description
Construction method	The excavation should be carried out in two stages to maintain the short-term stability during construction. The overview of construction staging is as follows. • Excavate 200mm below existing concrete footing of the gabion wall • Install first row of soil nails at 200mm depth below existing concrete footing of the gabion wall • Apply shotcrete facing • Excavate to final depth • Install second row of nails • Complete the shotcrete facing/ Rail embankment and drainage.
Geotechnical strength and	The soil nail wall was designed using working stress method. Adopted Factor of Safety (FoS)

Factor of Safety	requirements for different design cases are presented in Table 7.
Design loading	Long term – 20 kPa traffic surcharge was adopted 2m away from the top of the Gabion wall Short term construction loading - 20 kPa traffic surcharge + 5kPa construction surcharge between the road and the top of the Gabion wall Seismic loading – 0.04 Horizontal acceleration + 12 kPa traffic surcharge Rapid drawdown – Maximum water level of 1/3 of the embankment height was adopted + 10 kPa traffic surcharge.
Design Methodology	Design was carried out using GeoStudio Slope/W software. Material parameters for embankment fill of the bridge approach were determined using a back analysis assuming long term FoS of 1.5. The soil nail wall design was carried out using the estimated backfill parameters. 24mm Grade D500N threaded bar with minimum grout diameter of 170mm was adopted. The grout-soil bond strength was taken as 50 kPa through the full length of the nail. A geotechnical strength reduction factor of 0.5 was adopted for pullout resistance, in all design cases, based on AS 4678-2002. A structural strength reduction factor of 2.7 was adopted by the structural engineers for soil nail tensile capacity accounting for Importance class and material strength reduction, based on Clause 7.3.3 of AS5100.3.

4.1.2 *Design outcome*

Table 7 summarizes the assessed global factor of safety of the soil nail wall.

Table 7. Assessed FoS of the soil nail wall

Load case	Assessed FoS	*Required FoS*
First excavation during construction - Short term undrained condition	1.54	1.2
Second excavation during construction after installation of first row of anchors - Short term undrained condition	1.46	1.2
Short term after construction	1.7	1.2
Long term – Drained condition	1.50	1.5
Short term – seismic load	1.55	1.2
Short term – rapid drawdown condition	1.47	1.2

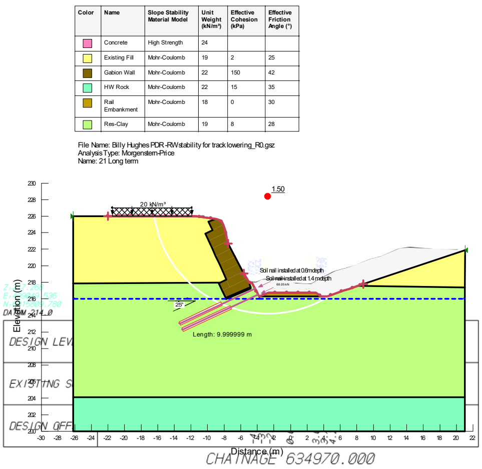


Figure 10. Assessed long term factor of safety of the soil nail wall

4.1.3 *Impact of the soil nail wall construction on the gabion wall*

The soil nail wall will be constructed directly below the footings of the existing gabion wall. Finite Element Modelling (FEM) has been undertaken to assess the impact of the staged excavation and installation of soil nails on the gabion wall and vice versa. This assessment is discussed in the sections below.

The assessment was based on finite element modelling using commercially available software Plaxis 2D.

The following were adopted for the numerical model:

- Two-dimensional plane strain model.
- Mohr-Coulomb constitutive model for the soil units Continuous plate elements to model the shotcrete
- Soil nails modelled as embedded beam row elements with hinged connection to the shotcrete.

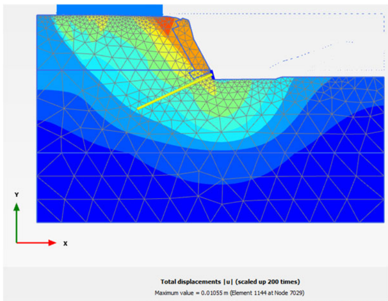
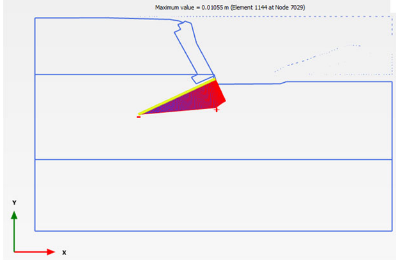
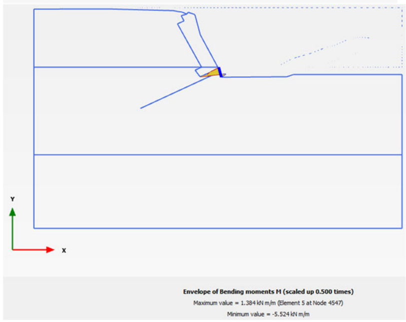
The section of the wall for analysis was selected to represent the most onerous conditions for the wall support system.

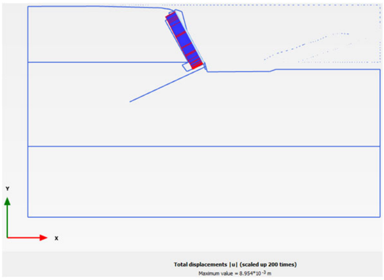
The following construction stages are adopted:

a) Initial condition (Gravity loading procedure)
b) Construction of the gabion wall
c) Excavate to 200mm below the gabion wall footing
d) Install 1st soil nail, and shotcrete
e) Excavate to 2nd soil nail row
f) Install 2nd row of soil nails and shotcrete
g) Flexural stiffness reduction (to 40%) to account for the cracked concrete sections in long-term.
h) Phi-c reduction stage

The results from the FEM analysis are summarized in Table 8.

Table 8. Impact due to construction of the soil nail wall

Illustration	Results of assessment
	Assessed maximum ground movement during the excavation for the soil nail wall is 10.6mm.
	The assessed maximum nail force = 182.9 kN in an Ultimate Limit State
	The assessed bending moment in the shotcrete is 5.5kNm/m x 1.5 (load factor) = 8.3kNm/m. The assessed shear force is 19kN/m x 1.5 (load factor) = 29kN/m



The assessed maximum gabion wall movement is 9mm during the final excavation stage for the soil nail wall.

Monitoring of the existing gabion wall is proposed to enable the actual wall deflections to be compared to the design predictions.

The factor of safety from the phi-c reduction phase is 1.55, which agrees well with the slope stability analysis shown in Figure 10 which satisfies the minimum requirements of 1.5 based on PSR Identifier No. IR-SR-A2I-456.

4.2 *Soil Nail Wall – Deflection Wall Transition*

The 2H:1V batters on either side of the deflection wall have short transitions which require a retaining system to maintain the stability of the overall slope. The extent of the soil nail walls at the deflection wall transitions are shown in Figure 11.

The arrangement of the proposed soil nail wall is as follows. Figure 12(a) and Figure 12(b) provide sketches of the soil nail configuration.

- Maximum excavation depth = 1.5 m.
- Two rows of 5m long soil nails.
- Horizontal spacing between nails = 1.0m
- Vertical spacing between nails = 0.5m to 1.0m
- Nail inclination 15 degrees.
- Preliminary nail type – minimum bond diameter of 170mm and soil nail bars with at least 260kN ultimate tensile capacity.

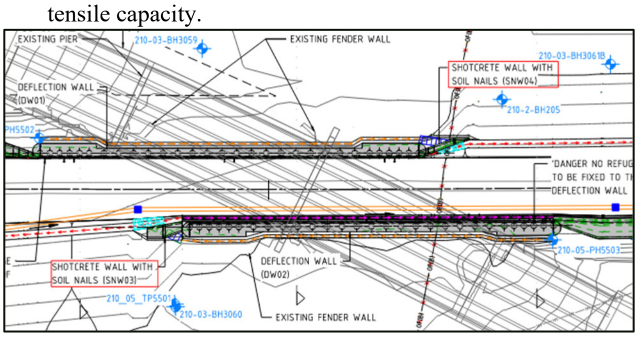
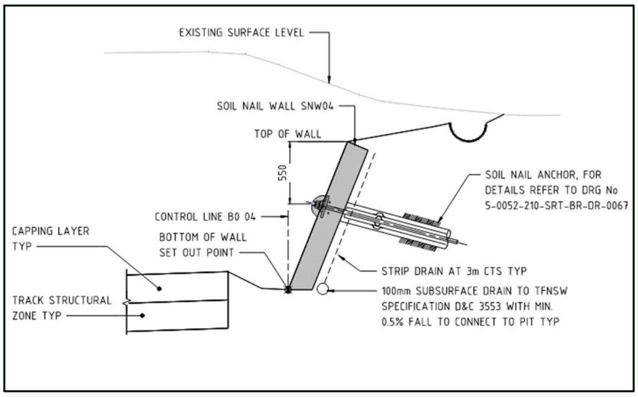
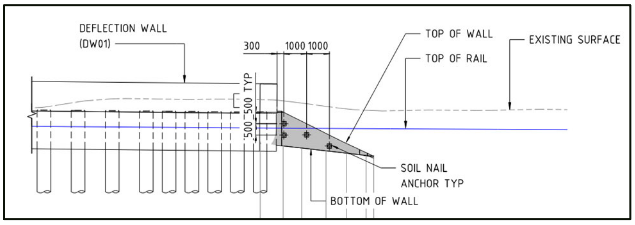


Figure 11. Soil nail wall - deflection wall transition alignment at the Billy Hughes site



(a)



(b)

Figure 12. Configuration of soil nails - (a) Cross Section, (b) Front Elevation

Long term slope stability of the proposed soil nail wall was assessed, and the outcome of the assessment is shown in Figure 13. It is considered the soil nail wall is stable in the long term and satisfies the required FoS outlined in the design requirements.

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Existing Fill	Mohr-Coulomb	19	2	25
	Rail Embankment	Mohr-Coulomb	18	0	30
	Res-Clay	Mohr-Coulomb	19	8	28

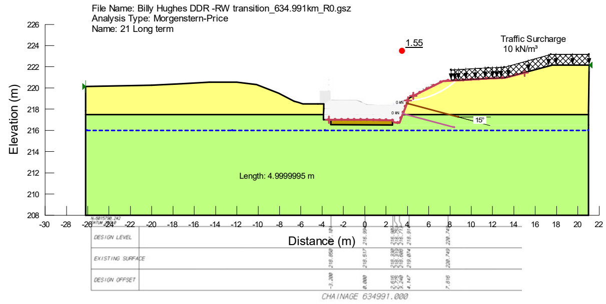


Figure 13. Assessed long term factor of safety of the soil nail wall

5 CONCLUSIONS

This paper presented the geotechnical design and assessment of the proposed deflection wall at Billy Hughes Bridge site. Design methods of deflection wall were outlined, and design findings were summarized. The impact assessment of proposed deflection wall to existing bridge pier was also undertaken. In addition, reinforcing the stability of existing gabion wall by soil nail wall was designed and its impact on the gabion wall was also assessed.

6 ACKNOWLEDGEMENTS

The authors acknowledge the collaborative work between Martinus Rail and Aurecon and BG&E DJV colleagues who have contributed to the development of design work. The authors are grateful for their permission to publish the paper from Martinus Rail and ARTC/Inland Rail.

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