HYBRID RETAINING SYSTEM TO SUPPORT HIGH ABUTMENT APPROACH

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https://doi.org/10.56295/AGJ6033

ABSTRACT

Ison Road Overpass bridge with high approach embankment at each abutment has been proposed to carry the existing Ison Road over Geelong to Melbourne rail corridor, extending the road south towards Browns Road in Werribee, Victoria. The high approach embankment behind each abutment requires a retaining structure to retain its earth fill. At the same time, collision protection shall be considered in accordance with AS 5100.1 for all structures within 20 m adjacent to existing or proposed rail track centrelines. An innovative hybrid retaining system consisting of L-shape wall, reinforced soil structure (RSS) block and required ground improvement works has been progressively developed through value engineering process as part of design development which was then built successfully. This paper commences with a brief description of geological conditions along the proposed project alignment and then discusses the optioneering process to develop the innovative and cost-effective retaining system to support the high abutment approach. The paper moves further to discuss key geotechnical issues and challenges encountered in design, basis of design, design approaches and assumptions, ground improvement works needed, subsequently presents key observational views from construction of the retaining system and finally presents some technical insights for future design and construction of similar retaining system as conclusions.

1 INTRODUCTION

Ison Road Overpass bridge has been proposed to carry the existing Ison Road over Geelong to Melbourne rail corridor extending the road south towards Browns Road in Werribee, Victoria (see Figure 1). The existing rail corridor features two (2) broad gauge tracks maintained by V/Line and one (1) standard gauge track maintained by Australian Rail Track Corporation Limited (ARTC). The proposed bridge consists of northern and southern abutment piers and a centre pier with a retaining structure as a deflection wall at each abutment, a blade wall at centre pier, a high approach embankment (approximately 11 to 12 m in height) behind each abutment to be retained by a retaining structure along the longitudinal direction and with 1V:3H & 3.5H batter slope along the transverse direction, plus minor retaining structures along approach embankments, ground improvement across part of embankment footprints and utility protection over existing utilities with ground improvements.

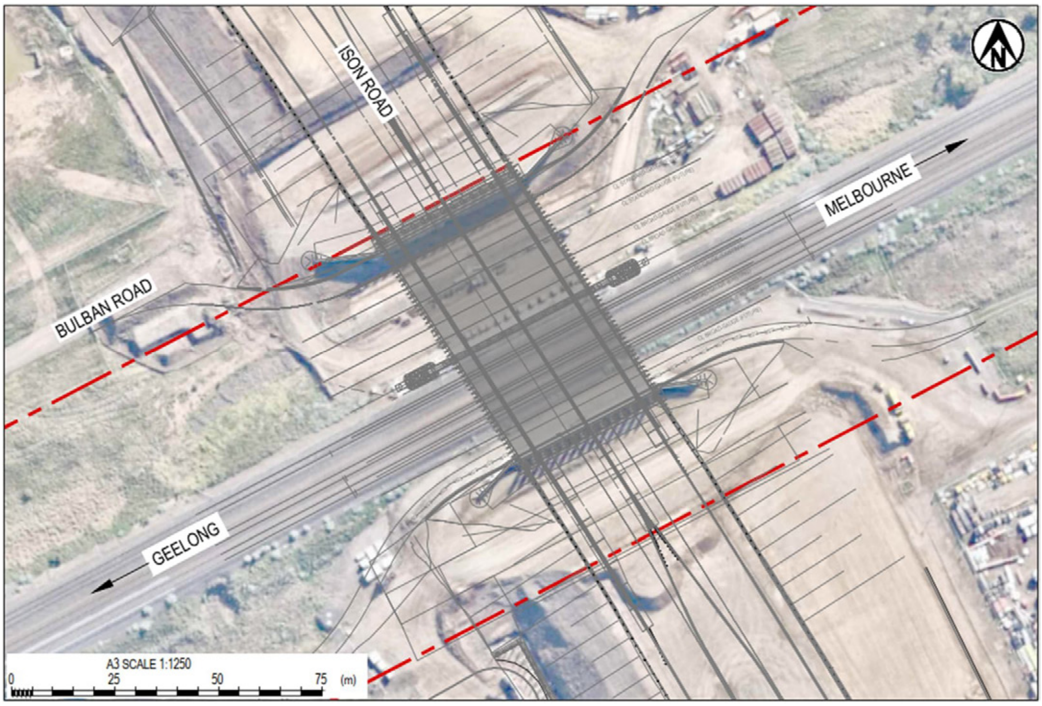

Figure 1: Ison Road Overpass Bridge Location

The project aims to provide a vital link to the developing residential areas of Wyndham West and to provide residents' access to surrounding areas and broader transport networks, to support urban growth and housing development in this high-growth area, to reduce traffic in central Werribee, thereby reducing congestion and improving journey reliability to Princes Freeway and the Werribee Town Centre; and to improve amenity and safety of local streets and Werribee Town Centre. At time of preparing this paper, the overall design has been completed, and construction was under its way, scheduled to complete in April 2025.

The proposed overpass is located approximately 4 km west of Werribee and will connect Ison Road on the north to the future Ison Road Extension on the south. The site, excluding the existing rail embankment, is generally flat in topography, and mostly grassed. Trees were sparsely present along the local access road that runs parallel to the existing rail line on the southern side. An active service corridor runs underground parallel to the rail corridor on the southern side, approximately 17 m behind the abutment piles. The services consist of 30 mm diameter LV (Low Voltage) electrical cables at an average depth of 0.9 m from the existing surface level. A channel is at the toe of the existing rail embankment, where overflow water from Lollypop Creek can backflow and stagnate. Aerial imagery shows ponding of water in this channel from time to time. The new Ison Road Overpass bridge is an integral structure that comprises precast Super-T girders forming the bridge deck of two (2) spans with approximate width of 36 m and lengths of 30 m and 33 m, respectively. Each abutment foundation consists of 900 mm diameter bored piles spaced at 2.65 m centre to centre (c/c). Each pile was sleeved (with a bigger diameter corrugated HDPE pipe) to remain structurally unstrained within the fill abutment. Piles were constructed to be structurally isolated and independent of the rail collision protection wall in front of abutments which is one of the key subjects to be discussed in this paper. The centre pier consists of 900 mm diameter bored piles spaced at 3.04 m c/c.

This paper commences with a brief description of geological conditions along the proposed project alignment and then discusses the optioneering process to develop an innovative and cost-effective retaining system to support high bridge abutment approach. The paper moves further to discuss key geotechnical issues and challenges encountered in the design, basis of design, design approaches and assumptions, ground improvement works needed, subsequently presents some key observational views from construction of the retaining system and finally presents some technical insights for future design and construction of similar retaining system as conclusions.

2 GEOLOGICAL SETTINGS AND OVERVIEW OF GROUND CONDITIONS

The published geological map of the area shows the regional surface geology at the subject site to comprise the late Neogene to early quaternary aged Deutgam Silt (Nxe), underlain by quaternary aged materials of the Newer Volcanic Group (Neo). The location of the site in the context of the regional geology is presented in Figure 2 below. The soil strata associated with the Deutgam Silt typically comprises a reddish-brown, poorly bedded silty and sandy clay with minor sand and gravel. The soil strata associated with the Newer Volcanic Group unit typically comprise clays of high plasticity, often interspersed with cobbles and boulders of the less weathered underlying basalt, known as 'floaters'. Depth to basalt bedrock can be highly variable over relatively short horizontal distances within this unit but generally ranges from 2 m to 6 m with a long section presented in Figure 3 below. From Figure 3, a thin layer of existing topsoil and a layer of uncontrolled fill with varied thicknesses were observed from past and present investigations.

The general existing surface level is approximately between 18.0 m RL and 20.0m RL, generally increasing with chainage (from south to north). The Visualising Victoria's Groundwater (VVG) database indicates that groundwater across the site is generally less than 5 m in depth. Based on the available data from an early investigation prior to award of this project, the groundwater levels were expected to vary throughout the project alignment but typically encountered between RL12 m AHD and RL 15 m AHD, where AHD stands for Australian Height Datum. It has been noted that groundwater was not encountered in any of the boreholes from the additional investigation carried out by the designer during the design delivery phase. It should be noted that during rock coring, due to the drilling technique utilised, groundwater could not be observed, as opposed to during auger drilling in the surficial layers. The design groundwater level has been adopted as 2.0 m below the existing ground level to allow for potential water level rise or seasonal high-water table.

3 HYBRID RETAINING SYSTEM TO SUPPORT HIGH ABUTMENT APPROACH

As discussed above, the high approach embankment behind each bridge abutment requires a retaining structure to retain its earth fill. At the same time, collision protection shall be considered in accordance with AS 5100.1 for all structures within 20 m adjacent to existing or proposed rail track centrelines. Collision loads shall be considered under the ULS condition with a load factor of 1.0. As both south and north abutments are located within 20 m of the existing and/or proposed future rail track centrelines, the proposed retaining structures at both south and north abutments have been designed as deflection walls to resist potential collision load imposed due to impact from derailed trains.

During the conceptual design phase, different retaining/deflection systems were proposed and assessed against with the reference design of a high cantilevered bored pile wall solution provided by the Principal, i.e., Major Road Projects

Victoria (MRPV)'s consultant. The pros and cons from design, construction and cost perspectives have been compared against each other. For the high cantilevered bored pile wall solution, large lateral displacement and induced bending moments / shear forces are most likely expected due to high earth pressure behind the wall. This requires large diameter piles, socketed sufficiently deep into strong basalt rock, and possibly requires in combination with multiple high-strength textile layers placed within the compacted earth fill behind the wall as reinforcement to reduce earth pressure. This option demands a significant construction budget and may result in higher risks and challenges in terms of constructability and durability. In light of this, the development of an alternative innovative solution that would require a lower construction budget, a better engineering performance and achieving a lower risk profile became part of the winning/success strategy during the tender design phase, with further design optimisation to be carried out during the detailed design phase.

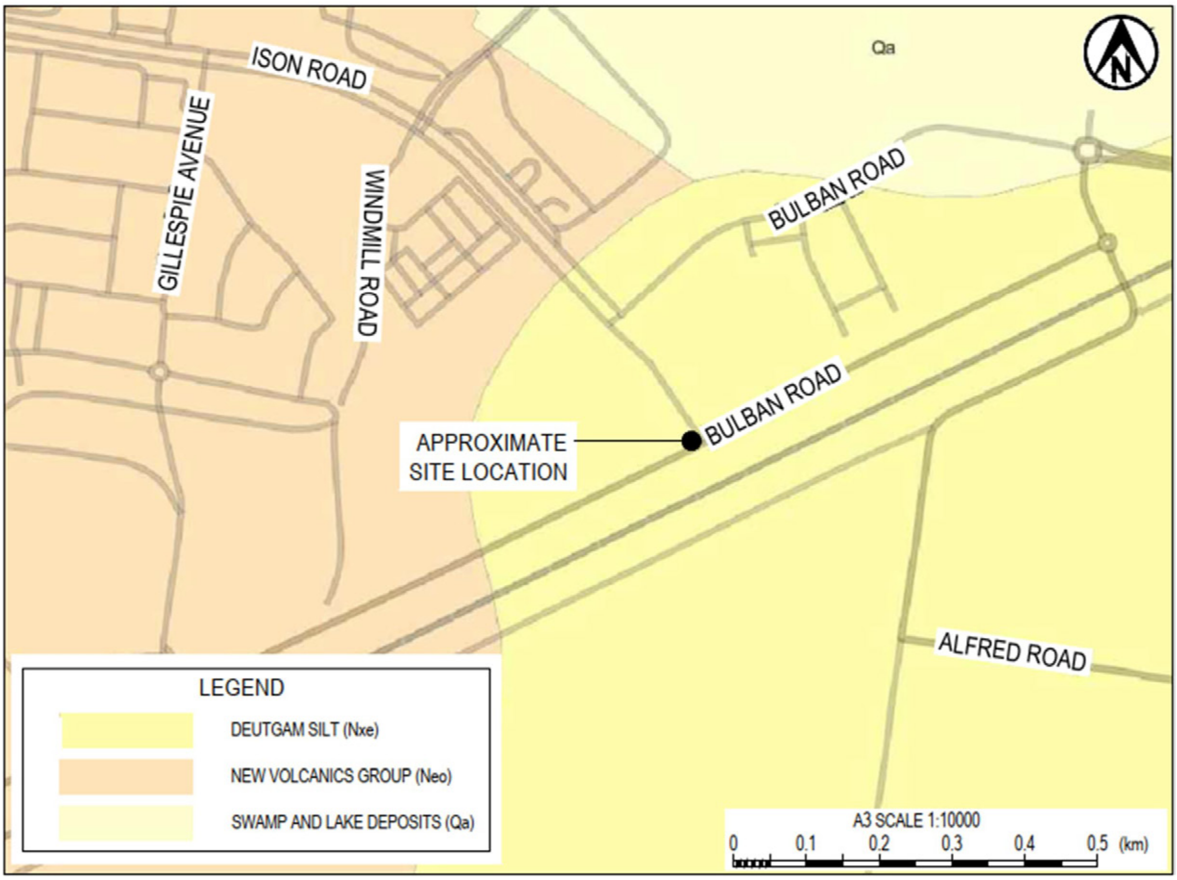

Figure 2: Geological setting as shown on the 1:50,000 geologic maps of the surrounding area

Low height deflection wall supported on a series of cast in-situ bored piles and reinforced soil structure (i.e., RSS block) wall extending from the top of the deflection wall was initially proposed and submitted as a solution during the tender design phase in lieu of the high cantilevered bored pile wall. This solution was further optimised by replacing the low height piled deflection wall with an L-shape wall as the final solution developed for design and construction. Figure 4 and Figure 5 present hybrid retaining walls at north and south abutments with their respective ground improvement works. As presented in Figure 4 and Figure 5, a 6 m wide and 800 to 1,000 mm thick cast in-situ L-shape concrete wall with a 750 mm deep and 500 mm thick shear key at the rear was designed to support RSS block placed on the top surface of the compacted earth fill behind the L-shape wall at each abutment. The lowest level of RSS block was determined based on likely flood level under a 1% annual exceedance probability (AEP) event with consideration of climate change plus allowing 500 mm freeboard (i.e., RL21.07 m AHD) to eliminate potential detrimental effects of a potential flood on the RSS components. The L-shape wall has three functions, i.e., the first function is to act as a deflection wall, the second function is to act as the lower retaining wall to retain the lower part of compacted earth fill and the third function is to act as a foundation with the compacted earth fill behind to support the upper RSS block. As for the RSS block, it will serve two different purposes, i.e., first is to act as the upper retaining wall to retain the upper part of compacted earth fill behind the RSS block and second one is to support the bridge approach slab and pavement above the RSS block. To meet stringent settlement requirements and stability requirements related to bearing capacity, sliding and global stability, ground improvement works were designed with their details presented in Figure 4 and Figure 5. At north abutment (see Figure 4), "remove and replacement" was proposed as ground treatment solution. At south abutment (see Figure 5), control modulus columns (CMCs) were designed to improve settlement performance for the new retaining system. The reasons for a different ground improvement solution to north abutment were twofold, i.e., one is to match with utility protection scheme using piled slab in conjunction with CMC and secondly, to avoid removal / excavation adjacent to the existing rail track. More details in relation to L-shape wall, RSS wall and the proposed ground improvement works underneath L-shape wall are discussed in the subsequent sections as part of overall design development.

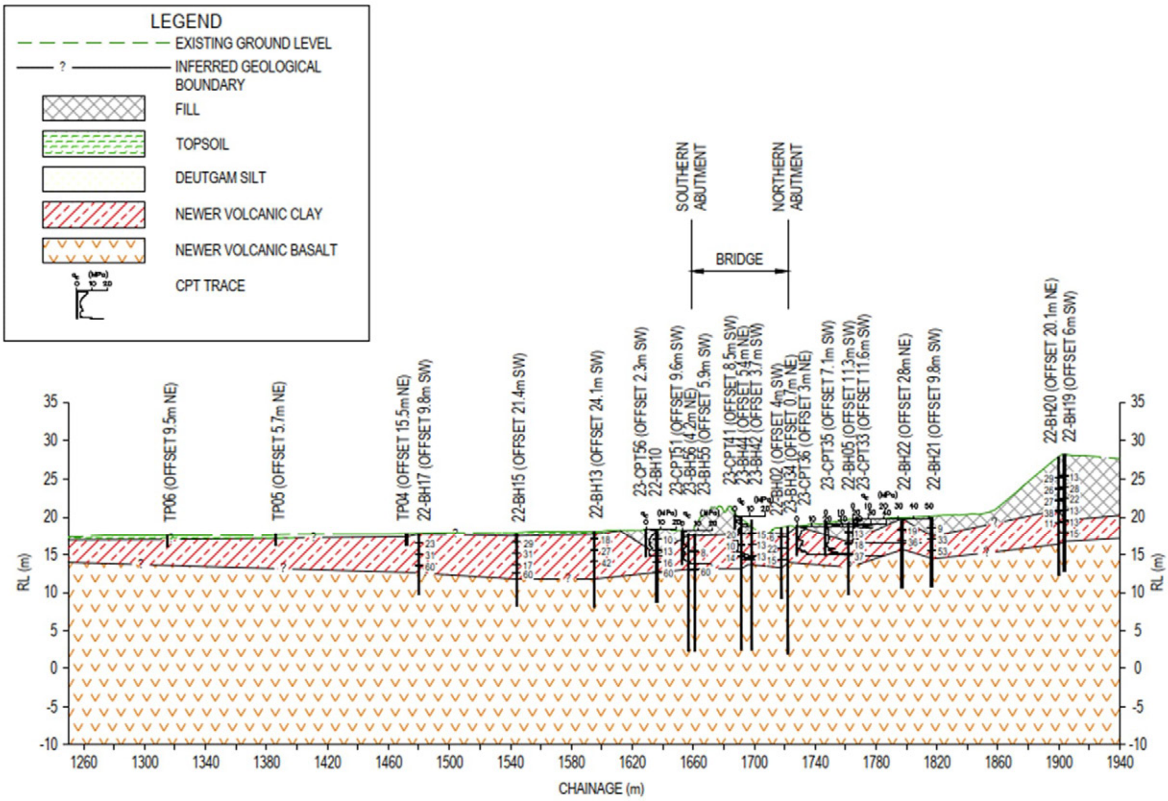

Figure 3: Geological long section along project alignment

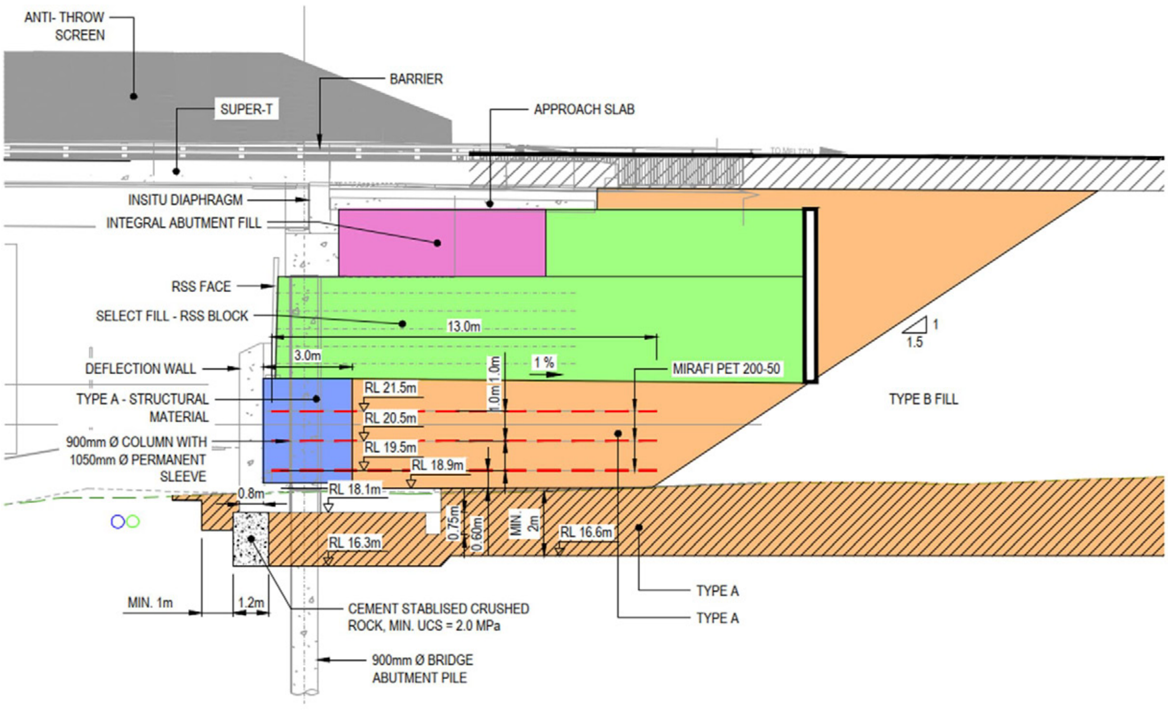

Figure 4: Hybrid retaining wall (L-shape wall + RSS wall) with ground improvement at north abutment

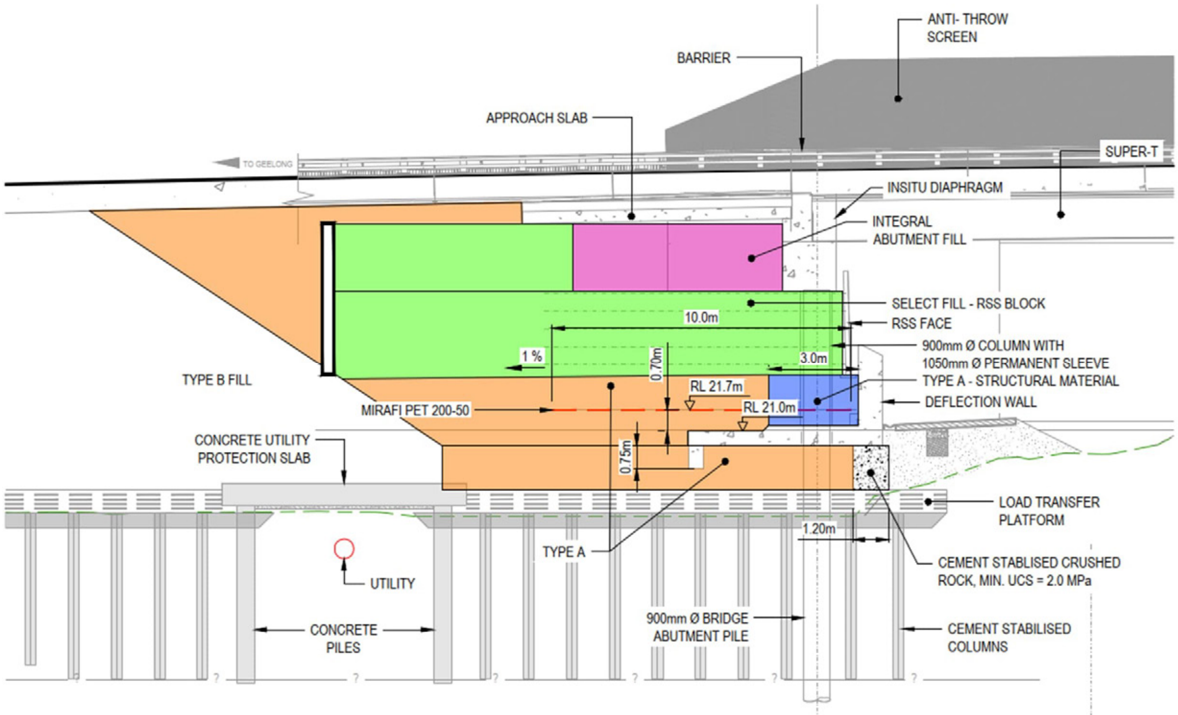

Figure 5: Hybrid retaining wall (L-shape wall + RSS wall) with ground improvement at south abutment

4 KEY GEOTECHNICAL ISSUES AND CHALLENGES

As discussed in Section 3, there are several issues and challenges to be addressed during the design of the proposed hybrid retaining system. The key issues and challenges faced were related to four different aspects, i.e., 1) potential global instability; 2) potential failures of bearing, sliding and overturning; 3) potential excessive displacements; and 4) potential poor interaction and integration between different components.

The proposed hybrid retaining system must be designed sufficiently robust to meet the relevant industry standards and specifications and address the following considerations, i.e., 1) global instability; 2) potential bearing capacity, sliding capacity and overturning moment failure; and 3) displacement limit. The design of the proposed L-shape wall and RSS wall shall also consider a full interaction and integration of the L-shape wall, the RSS wall and the integral bridge abutment system. During the design development, the key stakeholders and their appointed consultant regularly expressed their concerns about the actual performance of the proposed hybrid retaining system, whether the L-shape wall and the RSS wall could work together as a fully integrated retaining system. The following sections discuss how the key geotechnical issues and challenges were addressed by design.

5 INTERACTION AND INTEGRATION OF THE L-SHAPE WALL, THE RSS BLOCK AND THE BRIDGE

As discussed in the previous section, the bridge abutment consists of an integral abutment system (System A) and an earth retention and collision protection system (System B). The abutment was designed as cast-in-situ reinforced concrete crosshead supported by 900 mm diameter columns, placed at a spacing of 2.65m. The reinforcement of the column extends into the abutment sill beam, forming a secure connection. The structural capacity of this rigid connection was checked by the structural designer and confirmed to be adequate. The abutment sill beam then becomes a part of the integral connection between the superstructure and the abutment by introduction of in-situ diaphragm. The back side of the diaphragm also provides the step on which the bridge approach slab subsequently friction slab will sit onto. The foundation for each abutment consists of 900 mm diameter bored piles with an average rock socket depth of 2.7 m, which was installed before constructing the base slab for the L-shape retaining wall. These piles were socketed into the bedrock approximately 5 to 8 m below the existing ground level. To accommodate the piles, a 1,050 mm diameter hole was incorporated into the L-shape retaining wall's base slab. This arrangement creates an average 75 mm structural gap, filled with compressible foam for separation purposes. The actual structural gap nominated was 100 mm for the abutment side to accommodate movement induced by earth pressure and 50 mm for the rail side to accommodate movement induced from potential train impact load. The compressible foam has a maximum stiffness of 5 kPa/mm (Class L) for a 50 mm thick layer. The part of the pile column that extends above the base slab and under the abutment was enclosed with a sleeve, isolating the piles from the surrounding backfill material. The sleeve comprises a 1,050 mm diameter corrugated HDPE pipe with a rated capacity against a working lateral pressure of 150 kPa. This design approach ensures flexibility and unrestricted movement within the piles due to bridge dynamics ensuring that any movements within the piles are independent of the RSS walls, the L-shape wall and the backfill behind the L-shape wall. Up to the underside of the abutment sill beam and behind the abutment diaphragm / sill beam, Type A structure fill (i.e., a superior quality material complying with the requirements of Table 204.041 of Section 204 of VicRoads Standard Specification and used principally as capping, selected material, structural material and/or verge material) was placed. The backfill will be securely held in position by a combination of the crosshead (i.e., abutment sill beam plus diaphragm), a reinforced earth wall, i.e., the RSS wall, and the L-shape retaining wall.

The bridge interacts with the backfill at the abutment beam-embankment interface. A portion of the lateral earth pressure is taken by the integral abutment system, i.e., System A. The forces exerted by backfill are transferred from abutment beams to abutment columns, which, do not directly interact with the backfill due to isolation provided by sleeves. From the abutment wall base downwards, these columns become piles as they are bored into the soil and socketed into moderately to slightly weathered basalt rock. The abutment retaining wall (i.e., L-shape wall) has been detailed with a pre-penetration in the base slab to be independent of the bridge piles as discussed above. In addition, the bridge piles themselves are independent of the backfill behind the L-shape wall (and RSS block above). In the event of a train collision with the abutment wall (i.e., L-shape wall), the load will not impact the bridge pile and the bridge superstructure. The RSS wall is provided with a compressible separation layer between its foundation and the impact wall (i.e., L-shape wall). Therefore, there is no load path directly between the impact wall (i.e., L-shape wall) and the RSS block. Further assessment has been undertaken to confirm that in the event of a train collision, the maximum lateral displacement of the abutment wall (i.e., L-shape wall) would be less than 20 mm at the top of the wall and 30 mm at the base of the wall, which are within the 50 mm thickness of the compressible layer provided and the 50 mm offset allowed on the rail side, respectively. A geotechnical assessment of sliding induced from the active side has also been undertaken. The analysis undertaken indicates that the maximum sliding movement is less than 55 mm from the start of the construction to 100-year design life. Therefore, the 100 mm offset provided on the abutment side comfortably accommodates the expected lateral movements.

The L-shape concrete wall as discussed above is a continuous wall with a base slab that extends into the approach embankment in both longitudinal and transverse directions. The L-shape concrete wall extends further to form wingwalls for each abutment. The RSS block is supported by the engineered backfill behind the L-shape wall. This backfill, while being contained by the L-shape wall, transfers the loads exerted by the RSS block to the L-shape wall. The RSS block, on the other hand, is placed between the abutment beam and the L-shape wall. It is subjected to vertical and horizontal loads exerted by the embankment. The design of the RSS wall internal system has been formulated and documented by an independent supplier specialising in RSS wall systems which is not part of this paper. The upper half of the soil is supported by the RSS wall, managing the lateral forces along with a vertical load on the retaining wall. The lower half of the soil bears the vertical surcharges and pressure, with the retaining wall providing support.

6 DESIGN BASIS OF HYBRID RETAINING WALL

The hybrid retaining wall was designed on the basis of the following industry standards and specifications: i.e., Part 2 of Australia Standard of AS5100-2017 (+A1) (i.e., Design Loads); Part 3 of AS5100-2017 (+A1) (i.e., Foundation and Soil-Supporting Structures of Bridge Design); Part 4 of AS 1170-2007 (R2018) incorporating Amendments Nos 1 and 2 (i.e., Earthquake actions in Australia); VicRoads Standard Specification Section 204 - Earthworks; VicRoads Standard Specification Section 210 - Geotextiles in Earthworks; Australia Standard of AS 4678-2002 (Earth-retaining structures); British Standards of BS 8006-1-2010 (Code of practice for strengthened/reinforced soils and other fills); DTP Standard Sections 600 Series - Bridgeworks (Section 682 - Reinforced Soil Structures); and Transport for New South Wales Specification D & C R225 - Concrete Injected Columns. The following two specifications were considered as reference only, i.e., Transport for New South Wale (TfNSW R57, IC-QA-R57-2020): QA Specification R57. Design of Reinforced Soil Walls. Ed2/Rev11; and Transport for New South Wales (TfNSW R59, IC-QA-R59-2020): QA Specification R59. Construction of Reinforced Soil Walls. Ed3/Rev8.

Table 1 below presents a summary of some key technical requirements / assumptions for the design of the hybrid retaining wall.

7 DESIGN OF HYBRID RETAINING WALL

This section discusses the design geotechnical model at each abutment, relevant geotechnical design parameters, proposed ground improvement works and presents a summary of some key design outcomes. Based on the available historical and additional site investigation data, including CPT data, in situ field vane shear test data, borehole logs, test pit data, measured groundwater table, and laboratory test results, ground models, and strength / elastic stiffness related design parameters were derived as presented in Table 2a and Table 2b for north abutment and south abutment, respectively. Tabe 3a and Table 3b present stress and time-dependent consolidation related design parameters. Table 4 below presents strength related design parameters for compacted Type A fill, Type B fill (as per Section 204) and cement stabilised crushed rock materials.

Table 1: Key technical requirements

Item	Key technical requirements
Design life	• Design life of retaining walls including reinforced soil walls - 100 years.
Design approach	• Limit state design approach in accordance with AS5100 applied for both L-shape wall and RSS wall.
Settlement criteria for pavement	• The project specifications require that the Contractor assess the need for ground improvement in ground considered prone to settlement or under high embankments. • Placement of pavement material: The Contractor must not commence the placement of pavement material until the residual total predicted secondary settlement (or creep) is less than 20 mm on completion of such placement. • Sealing of pavements: The Contractor must not commence sealing of pavement until the residual total predicted secondary settlement (or creep) is less than 15 mm on completion of such sealing. • Defect Liability Period, i.e., 24 months after the Date of Practical Completion - total settlement: 20 mm. • Within the period of 10 years after the Date of Practical Completion - Maximum Grade Change: 0.4%, measured over a distance of 5 metres or less (1 in 250 or 20 mm over a 5 m distance) within 20 m of any structure. • Over the period of 10 years following the Date of Practical Completion - total settlement: 50 mm.
Settlement criteria for RSS wall and L-shape wall	• Total settlement limit for earth retaining structures is 20 mm, and the differential settlement limit is 10 mm. These limits will be measured from commencement of construction elements on top of the completed wall backfill during the defects period. • A differential settlement limit of 0.5% for the RSS block in both parallel and perpendicular directions to the facing panels has been nominated by the proprietary RSS wall designer.
Lateral displacement criterion for RSS wall and L-shape wall	• Total horizontal movement limit is $H_w/125$ during Defect Liability Period, where H_w is the height of the retaining wall projecting from the base of the wall (i.e., finished surface) to the top of the wall.
Other criteria for RSS wall	• Effective cohesion of the reinforced fill shall be taken as zero (c' = 0). The wall facing shall lean backwards (minimum 1(H):40(V) verticality).
Global stability criteria for bridge approach embankment, RSS wall and L-shape wall	• Long term case: The global stability for as constructed embankments to achieve a factor of safety (FOS) of 1.6 or greater in accordance with the requirements of the Guide to Road Design (AGRD) Part 3 of Austroads Standard. Note: Additionally, a strength reduction factor approach has been considered in accordance with AS5100.3 for abutment global stability check (i.e. target long-term equivalent FOS of 1/0.55 = 1.81). • Short term case: FOS $\geq$ 1.3 * • Seismic case: FOS $\geq$ 1.1 * • Flood case: FOS $\geq$ 1.2 * • Rapid drawdown case: FOS $\geq$ 1.2 *

Note: * - Reference to typical industry practices.

Geotechnical design assessments have been carried out which included global stability assessment using commercial software Slope/W, bearing capacity, sliding capacity and overturning were assessed using an in-house excel spreadsheet, and soil-structure interaction assessment using commercial finite element software, PLAXIS 2D. To account for potential earthquake impact, an estimated peak ground acceleration (PGA), for 1 in 1,000 AEP of 0.15g as per AS1170.4-2018 was adopted in the design. For settlement assessment, the design water level has been adopted as 2.0 m below ground level (BGL) to allow for potential water level rise or seasonal high-water table despite a lower water level was observed during the course of the additional site investigation. Rankine active earth pressure was adopted in the design assessment against bearing capacity, sliding capacity and overturning. The earth pressure profiles obtained from PLAXIS 2D modelling were also reviewed against the design criteria as additional secondary checks.

Table 2a: Simplified geotechnical model and design parameters (strength) adopted at north abutment

Material	Top of layer RL (m AHD)	Thickness (m)	Unit weight γ_t, (kN/m^3)	Effective cohesion, c' (kPa)	Effective internal friction angle, ϕ' (Degree)	Undrained shear strength, s_u (kPa)	UCS* (MPa)	Drained Modulus, E' (MPa)	Poisson's Ratio, ν
Stiff to Very Stiff Clay	18.8	3.3	17.0	8	27	70-80-100	-	15 – 22	0.3
Very Stiff Clay	15.5	2.9				110	-	24	
Basalt, High Strength	12.6	-	25.0	400	42	-	20	8,000	0.2

Note: * - UCS is unconfined compressive strength.

Table 2b: Simplified geotechnical model and design parameters (strength) adopted at south abutment

Material	Top of layer RL (m AHD)	Thickness (m)	Unit weight γ_t, (kN/m^3)	Effective cohesion, c' (kPa)	Effective internal friction angle, ϕ' (Degree)	Undrained shear strength, s_u (kPa)	UCS (MPa)	Drained Modulus, E' (MPa)	Poisson's Ratio, ν
Very Stiff Clay	18.2	0.6	17.0	8	27	150	-	33	0.3
Stiff Clay	17.6	3.1				60	-	13	
Very Stiff Clay	14.5	1.9				110	-	24	
Basalt, High Strength	12.6	-	25.0	400	42	-	20	8,000	0.2

Table 3a: Simplified geotechnical model and design parameters (consolidation) adopted at north abutment

Material	Top of layer RL (m AHD)	Bottom of layer RL (m AHD)	Unit weight γ_t, (kN/m^3)	Undrained shear strength, s_u (kPa)	Pre-consolidation Pressure, p_c'	Compression Ratio, CR	Recompression Ratio, CRR	Creep coefficient, $C_{\alpha e(OCR)}$	c_v* (m^2 / Year)
Clay 1	18.8	18.2	17.0	100	550	0.15	0.0225	0.00175	3.6
Clay 2	18.2	16.2		80	440				
Clay 3	16.2	15.5		70	385				
Clay 4	15.5	12.6		110	605				

Note: * - c_v is coefficient of vertical consolidation.

Table 3b: Simplified geotechnical model and design parameters (consolidation) adopted at south abutment

Material	Top of layer RL (m AHD)	Bottom of layer RL (m AHD)	Unit weight γ_t, (kN/m^3)	Undrained shear strength, s_u (kPa)	Pre-consolidation Pressure, p_c'	Compression Ratio, CR	Recompression Ratio, CRR	Creep coefficient, $C_{\alpha e(OCR)}$	c_v (m^2 / Year)
Clay 1	18.2	17.6	17.0	150	825	0.15	0.0225	0.00175	1.5
Clay 2	17.6	16.0		60	330				
Clay 3	16.0	14.5			300				
Clay 4	14.5	12.6		110	605				

Table 4: Design parameters adopted for Type A fill, Type B fill, and cement stabilised crushed rock

Material	Unit weight γ_t, (kN/m^3)	Effective cohesion, c' (kPa)	Effective internal friction angle, ϕ' (Degree)	Undrained shear strength, s_u (kPa)	Drained Modulus, E' (MPa)	Poisson's Ratio, ν
Type A Fill	21.5	5	34	-	60.0	0.3
Type B Fill	20.0	8	28	125	32.6	0.3
Cement stabilised crushed rock	21.5	15	40	1,000	100.0	0.3

A trial-and-error approach was adopted to optimise the required dimensions and performance of the L-shape wall (see Table 5a, Table 5b, and Table 5c) and the RSS block (see Table 6) through introducing high-strength geotextile to enhance overall global stability of the hybrid retaining system; implementing appropriate ground improvement works to improve bearing capacity and sliding capacity of the L-shape wall base foundation as well as to reduce potential excessive total and differential settlements of the L-shape wall foundation. A shear key has been designed and installed at the rear of the base slab to achieve an optimum base slab width.

Table 5a: Summary of estimated base slab bearing pressure & lateral displacement of L-shape wall^

L-shape wall location	Base slab width / shear key embedment (m)	Base slab length (m)	Maximum wall height, H_w (m)	Estimated ground bearing pressure within localised CTCR^^ & * (kPa)	Estimated ground bearing pressure within Type A fill base footprint^^^ (kPa)	Deflection limit of H_w/125 during DLP** (mm)	Estimated lateral displacement during DLP (mm)	Estimated lateral displacement from start of construction to the Date of Practical Completion (mm)	Estimated lateral displacement from start of construction to end of DLP (mm)	Estimated lateral displacement from start of construction to 100-years (mm)
North abutment	6.00 / 0.75	37	5.3	300 - 420	225 - 320	42	2 - 3	25 - 29	28 - 31	28 - 34
North wingwall	4.50 - 6.00 / 0.50	11	5.0	< 400	120 - 330	40	0, negligible	32 - 38	32 - 38	34 - 41
South abutment	6.00 / 0.75	37	2.8	120 - 425	80 - 235	22	5 - 6	38 - 40	43 - 46	43 - 53
South wingwall	4.50 - 6.00 / 0.50	11	2.8	< 300	40 - 250	22	3 - 4	33 - 34	37	38 - 46

Notes: ^ - L-shape wall has adopted compressive strength of concrete of 40 MPa; ^^ - Estimated geotechnical strength ($R_{d,g}$) against end bearing failure > 500 kPa; ^^^ - Estimated geotechnical strength ($R_{d,g}$) against end bearing failure > 400 kPa; * - CTCR = cement stabilized crushed rock panel; ** - DLP = defects liability period & the deflection limit refers to the relative tilting of the L-Shape wall.

Table 5b: Summary of estimated maximum induced structural actions in L-shape wall

Location		Induced maximum bending moment (kN.m/m)	Induced maximum shear force (kN/m)	Induced maximum tension (kN/m)	Induced maximum compression (kN/m)
North Abutment	Wall	318	145	0	59
	Base	321	130	-151	24
	Shear key	71	148	-3	25
North Abutment - Wing Wall	Wall	98	94	0	26
	Base	98	71	-88	12
	Shear key	20	45	-4	15
South Abutment	Wall	134	92	0	45
	Base	138	64	-85	0
	Shear key	42	82	0	17
South Abutment - Wing Wall	Wall	99	74	0	23
	Base	99	56	-57	30
	Shear key	11	19	0	12

Table 5c: Summary of estimated base movement and geotextile layers meeting global stability requirements

Location	L-shaped wall slab lateral movement (mm)		Number of geotextile layers (Mirafi 200-50, equivalent or better)	Geotextile vertical spacing*	Minimum geotextile length** (m)
	Due to earth pressure	Due to collision load^			
North Abutment	34	14	3	First layer 0.6 m; subsequent layers 1.0 m	13
North Abutment - Wing Wall	41	16	NIL	NIL	NIL
South Abutment	53	29	1	0.7 m	10
South Abutment - Wing Wall	46	27	NIL	NIL	NIL

Notes: ^ - The following collision loads on the deflection walls were adopted based on AS 5100.2 Cl. 11.4.2.3, i.e., (a) 4000 kN parallel to rails; (b) 1500 kN normal to rails, and loads (a) and (b) have been applied simultaneously at 2 m above rail level, distributed over a length of 2.0 m by 0.5 m.; * - Vertical spacing from the top of L-shape wall slab; ** - Length from the L-shape wall back facing.

At the north abutment (see Figure 4), a minimum of 2 m "remove and replacement" was assessed to be a cost-effective ground treatment solution. As for the south abutment (see Figure 5), where "remove and replacement" method was not possible due to proximity to the existing rail line, installation of 360 mm diameter control modulus columns (CMCs) spacing at 1,440 mm c/c in a square pattern to a full depth at rock level. The CMC has a minimum concrete compressive strength of 40 MPa and has been assessed as a robust and a low-risk ground improvement solution that will improve settlement performance for the new retaining system. The base of the L-shape wall sits on top of a load transform platform (LTP) which is approximately 2 m higher than the base of the north L-shape wall. This LTP was installed above the CMCs to facilitate the distribution of load from the embankment to the underlying inclusion members. It comprises a well-compacted granular fill with 4 layers of geotextile reinforcements (i.e., 2 layers in the longitudinal direction and 2 layers in the transverse directions). The purpose of the implementation of the 4 layers of high-strength geotextile is to enable an arch effect to be developed, to help distribute forces and to minimise the potential shear and bending forces to the CMCs. Based on the assessment, a 750 mm thick LTP comprises Type A fill material with four layers of geogrids Tencate Miragrid GX 200/50 or equivalent is sufficient for the purpose. A localised 1.2 m thick mass panel using cement stabilised crushed rock with a minimum UCS of 2MPa was proposed and installed immediately underneath the base of the L-shape retaining wall to manage stress concentration attributed to the overturning effect due to high earth pressure.

Table 6: Summary of minimum dimensions of RSS block and estimated lateral displacement

RSS wall description	Founding material	Maximum height (H, m)*	Minimum embedment (D$_m$, m)	Facing height (H$_1$, m)**	L (m)***	Estimated lateral displacement during DLP (mm)
North abutment	Type A fill	6.9	0	4.0	10.0	< 5
North wingwall - 1		6.5	1.8	3.7	7.5	
North wingwall - 2		6.5	2.5	6.3	7.5	
South abutment		6.7	0	3.5	9.0	< 6
South wingwall - 1		6.1	1.8	3.2	7.5	< 5
South wingwall - 2		6.1	2.2	6.0	7.5	

Notes: * - Height from finished level at toe to finished pavement level at top of wall. Inclusive of run-on slab beam/approach slab for abutment, and backslope height for sidewalls; ** - Height from finished level at toe to finished top level of wall; *** - Minimum RSS block width (i.e., reinforcement length.

From Tables 5a to 5c, the required width of the base slab decreases along the length of the L-shape wall away from the abutment with a shallower shear key embedment due to a lesser surcharge applied. This is because the face of the RSS block terminates at the either corner of the abutments and turns approximately 90 degrees, parallel to the embankment alignment with a 1V:3H batter slope gradually transitioning to the embankment behind the RSS block.

Based on the design assessment, it has been noted a major portion of displacements would take place during construction stage. This is because the subsoils are in an unsaturated condition with ground water table at a relatively low level. As discussed in Section 5, the 100 mm compressible gap provided on the abutment side allowing for installation of the abutment piles has been assessed to be sufficient to accommodate the expected lateral movements. The wall lateral displacement within the DLP were predicted to be quite small because a significant portion of the overall wall displacement would have occurred during the earth filling stage.

Localised high stress concentration found immediately underneath the vertical wall base was managed by introducing a localised CTCR panel. The panel will improve local bearing capacity as well as to avoid potential softening due to long term environmental impacts, such as elevated groundwater level, flooding, surface scouring etc. Within the L-shape wall base slab at the north abutment, tensile force was assessed to be as high as 151 kN/m which is attributed to high sliding force driven by earth pressure against the wall that is transferred down to the base slab. The shear key provides sliding resistance developed between the base slab and the underneath compacted Type A fill. Additionally, the localised cement stabilised crushed rock panel and the passive resistance on the abutment side contribute to the sliding resistance.

The estimated maximum induced structural actions (i.e., bending moment, shear force and axial force) of L-shape wall, base and shear key were issued to the structural designers for structural design. An independent structural modelling was also performed to simulate the coupled behaviour of the retaining system and the abutment under various loading conditions.

To improve the factor of safety (FOS) against potential slope instability, 3 layers of high strength geotextile were introduced for the north abutment approach embankment and 1 layer for the south abutment approach embankment.

The detailed design aspect of the CMCs, including the predicted structural actions and the load transfer platform under various loading conditions will be published in a separate paper. The paper will focus on the design of utility protection and the relevant ground improvement works.

The RSS blocks located above the L-shape wall have been assessed to be adequate based on a width and maximum height ratio of greater than 1.0. The predicted settlement profiles along the RSS block bases and the L-shape wall base slabs were found to be quite even which indicate the high stability and compatibility of the proposed hybrid retaining system. The assessment results also indicate sufficient FOS values against any potential global instability, bearing failure, sliding failure and overturning failure. The estimated RSS wall movements are also well within displacement limits. Table 6 above presents a summary of the RSS block design and the estimated lateral displacement.

8 KEY PERFORMANCE OBSERVATIONS FROM CONSTRUCTION

Instrumentation and monitoring schedules have been developed and implemented on site to verify and validate the design assumptions and to ensure adequate factors of safety can be maintained throughout the construction. The instruments installed on this site consist of 44 settlement plates and six horizontal in-place inclinometers along the alignment, one vertical inclinometer in front of each abutment, two vibrating wire piezometers under each approach embankment and a number of prisms on both the L-Shape wall and the RSS walls. The instruments that are directly relevant to the construction of the L-Shape walls and the RSS walls include one vibrating wire piezometer behind each L-Shape wall, the prisms at the different heights of the walls, and one vertical inclinometer in front of each L-Shape wall. The rest of the installed instruments were located away from the L-Shape wall for different purposes. Due to limited space, the detailed instrumentation and monitoring plans / schedules are not included here.

At the time of finalising this paper, backfilling of the L-shape wall, the construction of the RSS walls and the embankment backfilling had just been completed (see Figure 6).

Figure 6: Ison Road flyover with completion of L-Shape walls, RSS walls and abutment backfilling (photo taken in February 2025)

The movements from the installed prisms on the faces of both north and south L-shape walls and RSS walls recorded up to 14 February 2025 indicate that the walls deform laterally as rigid bodies with a maximum lateral displacement value of approximately 20 mm, which is about 50% of the maximum design predicted values (i.e., 40 mm for the L-Shape wall presented in Table 5a).

The predicted value, which is relatively high (i.e., 40 mm), accounts for the saturation effect in the ground by a potential future rise in the groundwater table, as suggested by historical flood record. However, the current ground was observed to be unsaturated (i.e., dry) with a low groundwater table typically recorded at a depth of 5 m below the existing ground surface. This may have contributed to the lower movements as measured during the recent construction phase.

In addition, the approach embankments behind the L-Shape walls were built to the base levels of the RSS walls and even higher behind the RSS blocks with a temporary steep but safe batter. This in turn resulted in partial ground movements occurring before the L-Shape walls were fully completed (see Figure 7). This would have reduced the movement impact on the retaining system.

As for the measured settlements, the installed prisms have recorded maximum values of approximately 25 mm and 15 mm for the north retaining system and the south retaining system, respectively. These measurements represent about 35% and 33% of the estimated settlement values, i.e., 70 mm and 45 mm for the north L-Shape wall and the south L-Shape wall at the slab base levels, respectively. Again, the monitoring results are aligned with the designers' expectation given the ground was relatively dry during construction as noted earlier and a portion of settlement might have occurred prior to the prisms being able to pick up.

Another factor that may have contributed to the reduction of the settlement is the usage of Aerolite, a lightweight fill to replace the original select fill for the construction of the RSS blocks after Issued For Construction (IFC) submission. The aerolite fill has a lighter unit weight varying from 16.7kN/m^3 to 19.6kN/m^3 and a slightly greater internal friction angle of 36 degree based on direct shear test results as compared to the assumed design value of 34 degree (limited by TfNSW). The original select fill for the RSS blocks (see the blocks in green colour as shown in Figures 4 and 5) has a greater unit weight of 22.0kN/m^3.

During construction, the contractor backfilled a small, isolated zone behind the rear toe of the L-Shape wall at each approach embankment (see the light brown colour blocks immediately behind the blue colour blocks as shown in Figures 4 and 5) using a low quantity of Type A fill (i.e., EGB A2 fill sourced from Excell Gray Bruni Harpley Estate). This EGB A2 fill has a cohesion of 9 kPa and an internal friction angle of 29 degrees based on the consolidated undrained (CU) triaxial test results. The original selected Type A fill for these two isolated zones has a cohesion of 5 kPa and an internal friction angle of 34 degree from the CU triaxial tests as presented in Table 4.

In order to confirm the suitability of using lightweight Aerolite fill and EGB A2 fill, re-assessments were undertaken to evaluate their potential adverse impacts on the global stability, and the sliding, bearing and overturning capacities for the L-Shape walls. The re-assessment results indicated the hybrid retaining system maintains adequate factors of safety against the potential global instability, sliding, bearing capacity and overturning failures. A review was also undertaken by the specialist RSS wall supplier (RECO), and they confirmed that the use of lightweight Aerolite fill as an alternative select fill has an acceptable impact on the internal stability and the durability of the RSS blocks.

Figure 7: Casting of concrete base slab for south L-Shape wall (photo taken in June 2024)

Based on the monitoring data from the vertical inclinometer installed in front of the south L-Shape wall, it has been observed the ground lateral displacement increased with time during the construction of the hybrid retaining system and the approach embankment. A maximum lateral displacement of 13 mm has been recorded up to 14 February 2025. As expected, this value, i.e., 13 mm, is found to be less than the recorded maximum lateral displacement value of approximately 20 mm by the prisms for the south hybrid retaining system. The vertical inclinometer installed in front of the north L-Shape wall was found to be faulty after November 2024, hence, no adequate measured results could be presented here for discussion.

The two installed vibrating wire piezometers have not picked up any meaningful porewater pressure readings, which could be attributed to the relatively dry ground conditions.

Based on the above discussions, the measured wall movements up to 14 February 2025 are found to be well within the estimated range. This indicates that the performance of the hybrid retention exceeds the anticipated behaviour, though it remains reasonably close to the design estimation. Nonetheless, survey monitoring will continue until the End of Defect Liability Period. At that point, the monitoring data will be further reviewed, and back analyses will be undertaken. The findings from the relevant back analysis will be published separately in the future.

9 DISCUSSIONS / CONCLUSIONS

A hybrid bridge abutment retaining system consisting of L-shape wall and RSS block supported by ground improvement works has been progressively developed and constructed as a result of value engineering. This retention system has been proven to be an innovative, cost-effective and relatively low risk solution to support an 11 to 12 m high embankment approaches for Ison Road Overpass project. All issues identified and challenges encountered in relation to potential global instability, bearing capacity failure, sliding capacity failure, overturning failure and likely excessive settlement have been addressed through a close collaboration with all stakeholders involved, encouraging engineering innovations to be proposed and implemented. It also provided an opportunity for the geotechnical designers to make good use of advanced design tools such as finite element modelling using PLAXIS 2D and subsequent validation using first principles and hand calculations. The concern of incompatibility and integration of the proposed L-shape wall, RSS block and bridge has also been addressed by implementing smart ideas such that the hybrid retaining system and the bridge will behave independently. It is the authors' view that this innovative hybrid retaining system can be implemented in other similar projects moving forward.

10 ACKNOWLEDGEMENTS

The authors wish to thank Major Road Projects Victoria (MRPV), and Decmil Southern Pty Ltd (DECMIL) for their permission to publish this paper and collaboration towards a successful project delivery. The authors also wish to extend their thanks to the civil and structural designers from pitt & sherry for their support. The views expressed in this paper are those of the authors.

CRediT authorship contribution statement

Daiquan Yang: Writing – original draft. **Alex Ting:** Writing – review & editing. **William Eom:** Writing – review & editing. **Peter Kerian:** Writing – review & editing. **Vincent Timoney:** Writing – review & editing.

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