

USE OF A ROCK FILLED TYRE WALL TO PROVIDE PASSIVE SUPPORT TO A LARGE BRIDGE ABUTMENT

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ABSTRACT

This paper describes the geotechnical assessments undertaken to demonstrate the global stability of an alternative abutment solution, implemented to accommodate construction of a large railway bridge for a corridor widening project.

Excavation for a crane and piling platform to facilitate installation of pier foundations and erection of the bridge superstructure resulted in the reduction of passive earth in front of piles supporting a stem wall that would eventually retain abutment fill. The construction program required backfilling of the abutment concurrent with the construction of the adjacent central pier, resulting in the implementation of a rock-filled tyre wall to restore passive support. The tyre wall, however, was closer to the abutment and significantly truncated the theoretical passive failure wedge. Geotechnical assessment of global stability was undertaken by staged analysis with finite element software. The new configuration was demonstrated to be structurally sound and geotechnically stable in both the temporary and permanent condition. This permitted the final access track to be relocated to a position that provided further economy in construction. The analytical model was also used to determine structural design actions in response to observations of wall movement during the backfilling process.

It can be noted that certain details, including project name and location, have been omitted from this paper to respect the confidentiality of affiliated parties.

1 INTRODUCTION

To increase capacity on an existing rail corridor, a new two-track bridge was built adjacent to an existing brick arch bridge, as shown in Figure 1. The new bridge comprises three spans, totalling 74m in length across two piers and approximately 11m above an underlying large creek which becomes a major drainage path during storm events. Site constraints forced the arrangement of the new bridge as follows:

- The alignment of the two piers was set by the existing adjacent bridge structure configuration.
- Based on economic preference to use precast girders, a continuous super structure was adopted which dictated the lengths of the outer spans (No. 1 and No. 3).
- The lengths of spans No.1 and No. 3 set the location of the bridge deck pile locations.
- For continuity with the existing structure abutment face and the site levels required to maintain the waterway volume, two large retaining walls were required at either side of the creek to maintain the abutment backfill.

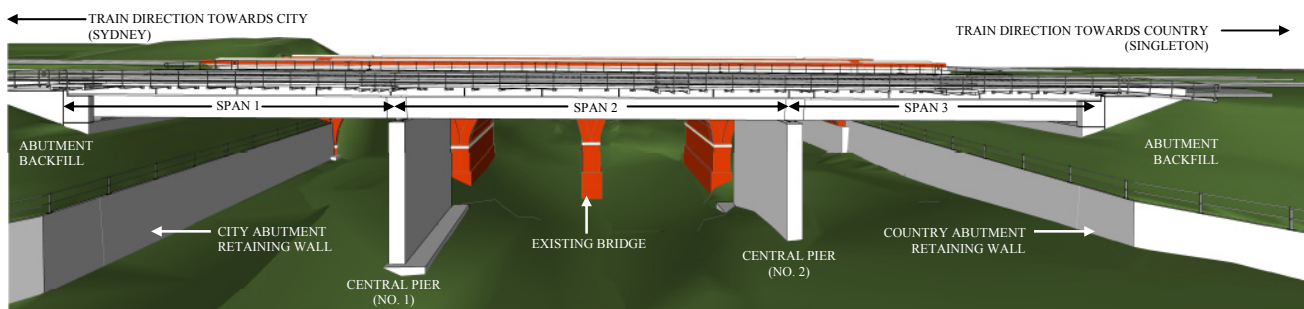


Figure 1: Perspective looking south

This paper focuses on the city side abutment where the sloping backfill was retained by a 4.35m high cast in-situ reinforced concrete stem wall, supported by a continuous 1050mm wide by 1200mm deep pile cap, over 750mm diameter piles spaced at 2.5m centres. The bridge plan is shown in Figure 2 and extracts of the 3D models are shown in Figures 3 and 4.

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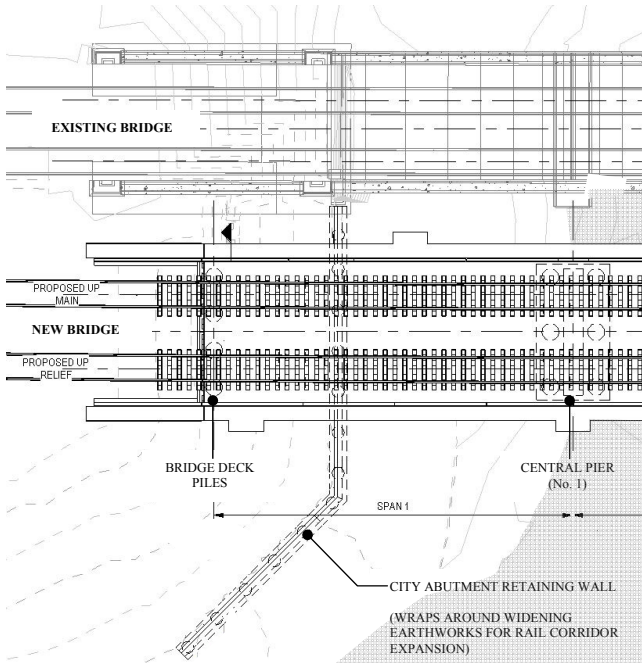


Figure 2: Bridge plan

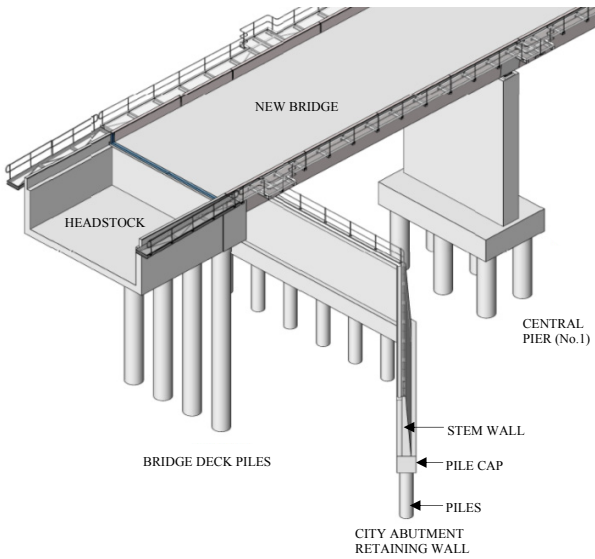


Figure 3: Bridge and abutment (no soil shown)

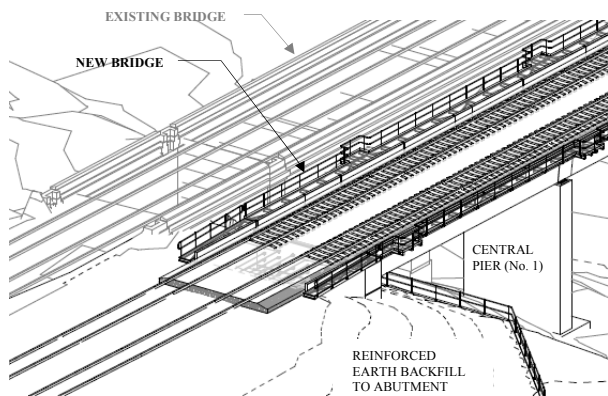


Figure 4: View of completed bridge and abutment

2 DESIGN TO ACCOMMODATE CONSTRUCTION LOGISTICS

2.1 THE REDUCTION IN ABUTMENT WALL PASSIVE RESTRAINT

The city abutment overlies residual soil with piles supported by a 3m total socket length in weathered and medium-to-high strength sandstone. A 4.5m wide access track was proposed in front of the abutment, retained by a rock-filled tyre wall, as shown in Figure 5.

During the initial stages of construction, excavation work was required to facilitate construction of a crane and piling platform adjacent to the No.1 central pier. The location of the platform was necessary to minimise construction work over the creek and enable the No. 1 central bridge pier to be constructed in parallel with the city side abutment. Once the abutment was completed, large cranes were then required to lift the bridge deck in place. The headroom clearance and swing for the piling rig and bridge deck cranes dictated the excavation level and lateral extents of the crane platform. Consequently, the required excavation removed the residual material in front of the abutment wall to the underside of the pile cap which was originally intended to provide lateral restraint to the abutment piles and pile cap. Figure 5 and 6 show the original design intent and the as-constructed temporary works geometry, respectively.

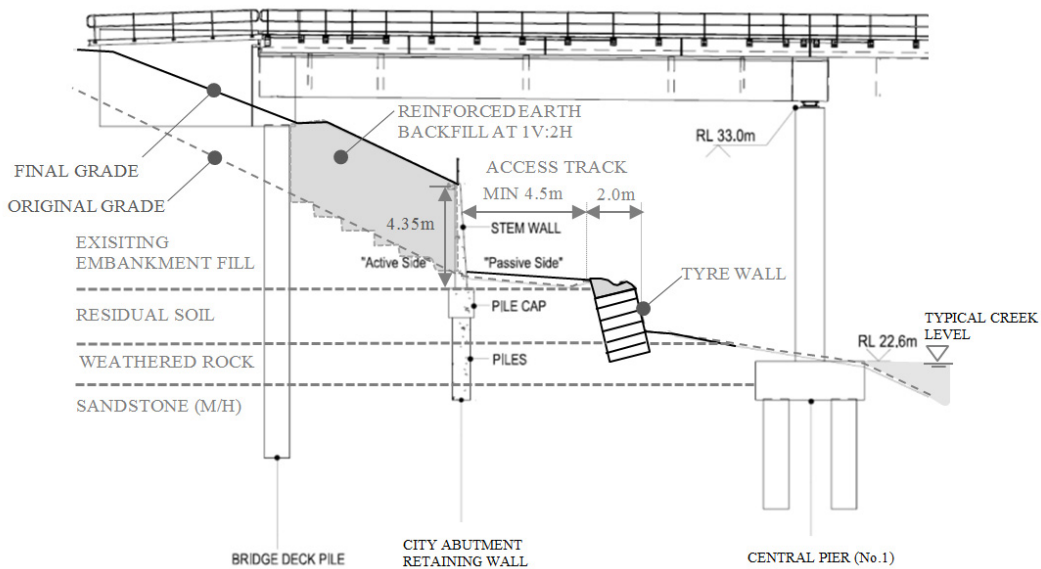


Figure 5: Bridge Elevation as originally proposed with high level access track on top of the proposed tyre wall

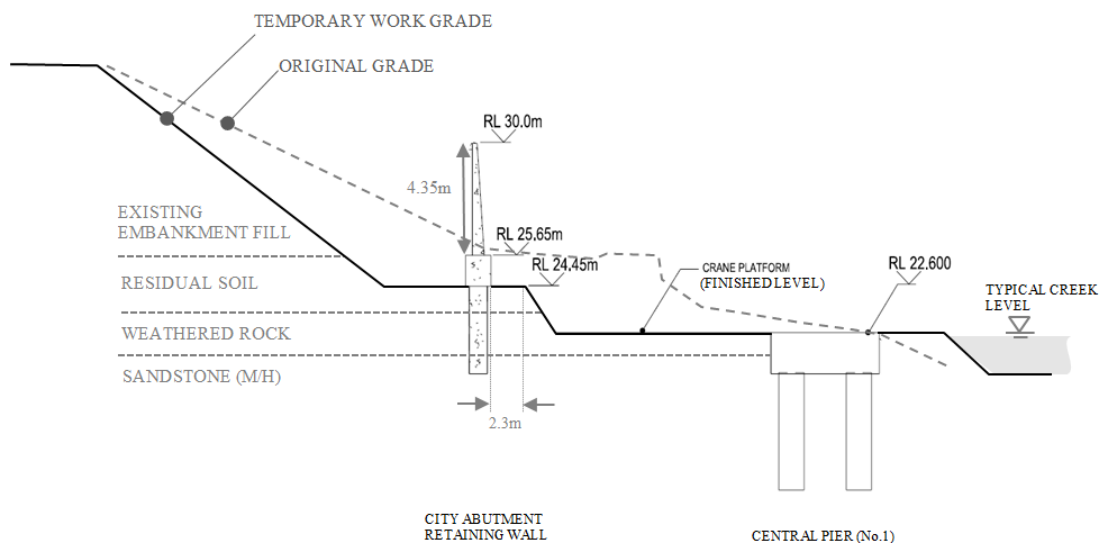


Figure 6: Geometry with the as-constructed crane/ piling platform during early site works

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In order to reinstate some passive backfill in front of the pile cap to permit commencement of the stem wall backfilling, a temporary retaining wall was proposed, closer to the abutment pile cap than originally designed. This retaining wall was comprised of interlocking recycled tyres, filled with imported single sized clean ballast material. To install the tyre wall and keep the required crane platform width, the face of the remaining in-situ material was trimmed and excavations were required to “key” the tyre wall into the underlying weathered rock to provide a competent foundation. The construction of the tyre wall was completed prior to any backfilling occurring to the abutment stem wall.

An important design consideration with the change in the abutment geometry was the comparative reduction in passive resistance on the temporary stability of the system, particularly during backfilling. With reference to classical soil mechanics theory, when the failure strength of the soil is mobilised, shear failure will occur within the soil mass at angles (θ) to the major principal plane, creating a network of shear planes at angles to the horizontal, both behind (the “active” condition) and in front (the “passive” condition) of the wall. The geometric limit of this wedge is calculated based on the internal effective friction angle and defines the maximum extent of material subject to failure. Outside this limit the soil is considered un-stressed. This theoretical active and passive “wedge” is illustrated in Figure 7. The “wedge” shown is based on an average value of the friction angle over the geological stratigraphy. In reality, the ground is not homogeneous and the theoretical passive “wedge” would not actually develop in such a linear fashion.

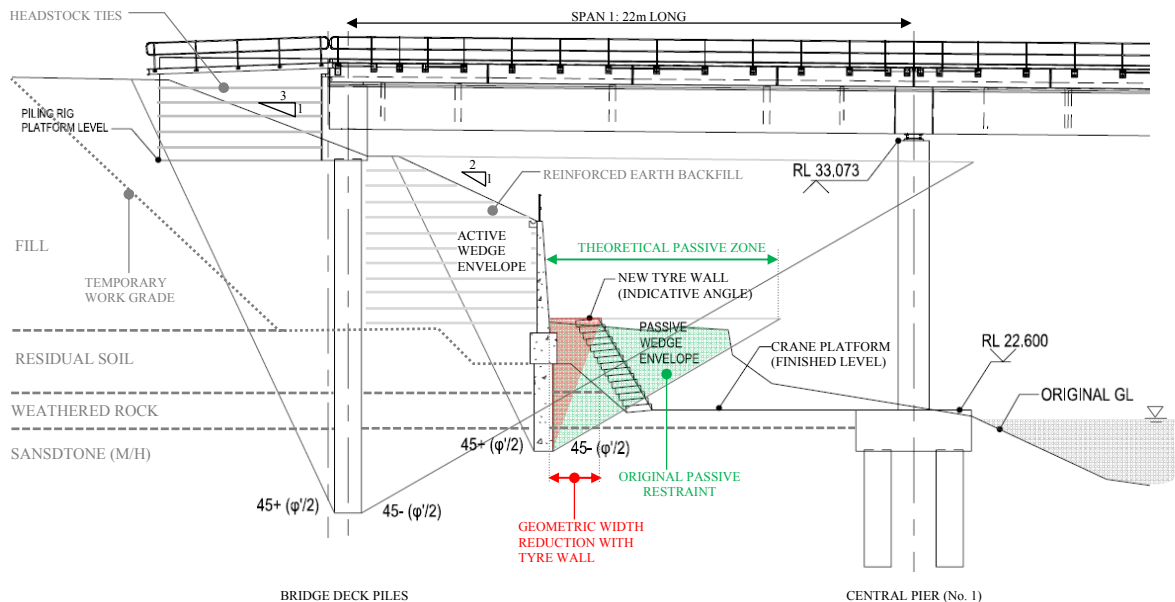


Figure 7: Altered passive support to the city abutment with illustration of the theoretical earth pressure envelopes

To conform to the original design, a large amount of fill material would have been required in addition to construction of another tyre wall (as shown in Figure 5) so a low level access track was instead adopted. Consequently, the temporary tyre wall, as shown in Figure 7 became the permanent support solution.

The final design included the provision of a 0.5m thick Reinforced Concrete (RC) wall in front of the tyre wall. This RC wall was constructed after the abutment stem wall was fully backfilled and the bridge deck was lifted into position. The purpose of the RC wall was notionally to protect the tyre wall against scour during flood events. The comparison between the original and final design is shown in Figure 8 and 9, respectively.

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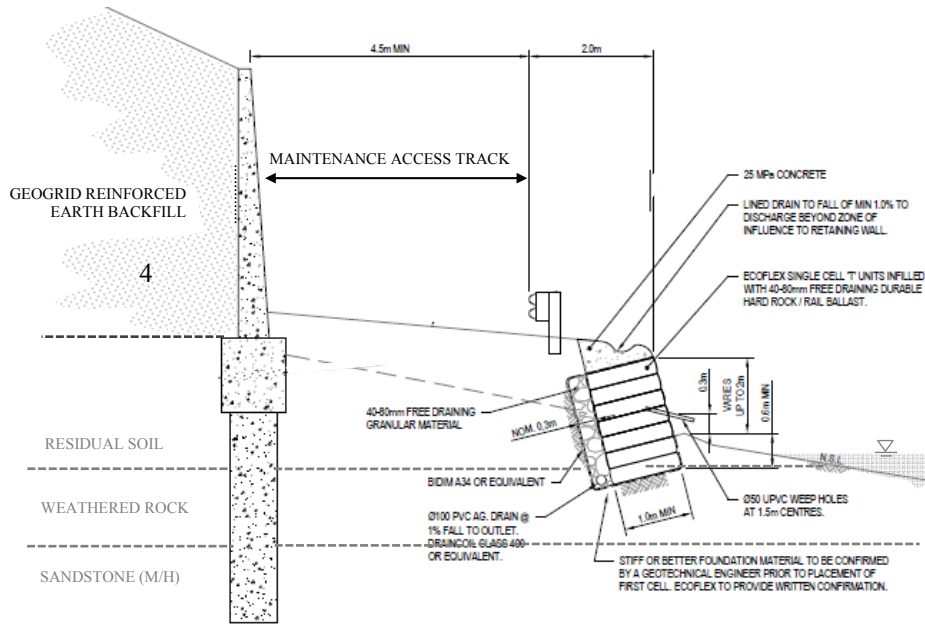


Figure 8: Original permanent retaining wall design in front of the city abutment

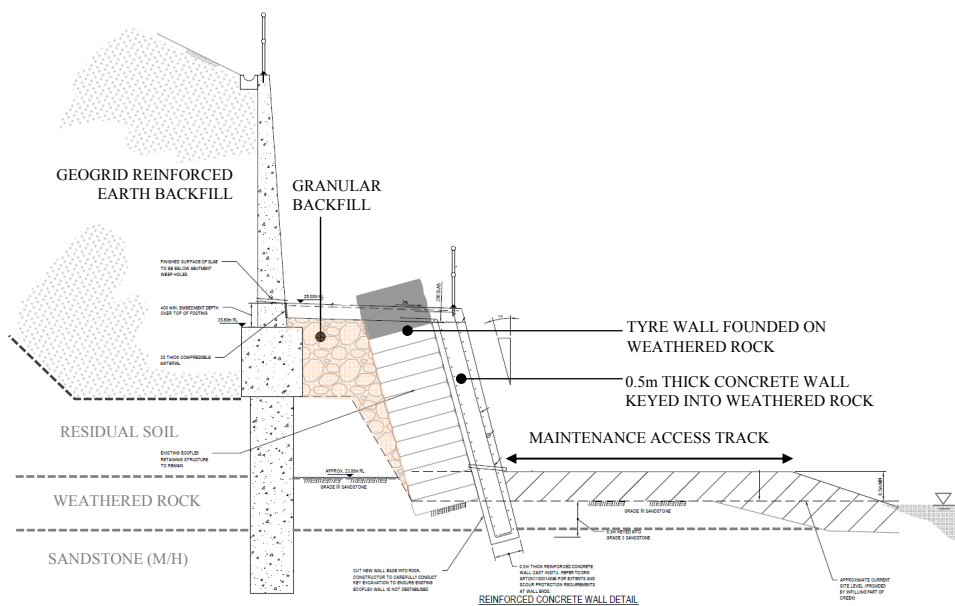


Figure 9: Modified permanent retaining wall configuration with the 0.5m thick concrete wall

Inspection of Figures 7 and 9 suggests complex soil- structure interactions effects will impact the development of active and passive earth pressures. Finite Element Analysis (FEA) was therefore utilised to further explore the following key influences:

1. The bridge deck piles and proximity to the abutment wall

- The horizontal train brake loading interacts with the laterally restrained bridge headstock, consequently altering the bridge pile active wedge.
- Bridge deck loading as a result of the vertical train loads (and potentially a smaller horizontal braking component) provides additional destabilising actions to the abutment wall system.

- Considering the geometry, there is an interaction with the passive wedge of the bridge deck piles and the active wedge of the abutment wall, as shown in Figure 7.
- Therefore, concern was that the reduction in passive support to the abutment wall may potentially affect the global stability and structural actions of the overall bridge abutment.

2. Geogrid and retaining stem wall interaction

- Typically, for an un-propped cantilever retaining wall, the movement induced under active earth pressure loads is significantly less than the corresponding movements required to mobilise fully passive conditions.
- Since there was a reduction in the passive wedge, less volume of soil is available to be “mobilised”. As such, fully passive conditions (soil failure) may result with comparatively less lateral wall movements.
- Despite the above, the geogrids within the abutment backfill provides a beneficial effect which serves to lessen the wall movements by reducing active pressures, which consequently may lessen the material mobilisation in the passive wedge- depending on the resulting overall equilibrium state.

3. Influence of the tyre wall

- The reduction in backfill volume coupled with the flexible behaviour of the tyre wall may potentially reduce the passive confining pressure as the wall is able to deform.
- The interaction of the passive tyre wall is critical to the stability of the abutment by providing restraint to the piles and pile cap. Considering strain compatibility, movement of the tyre wall impacts the behaviour of the imported backfill and hence the abutment wall piles

4. Influence of the 500mm concrete wall (long term)

- The performance of the tyre wall in the long term impacts the final configuration of the access track and the long term retention system proposed (as shown in Figure 9).
- The angle of the concrete wall, the dead load of the concrete and the keying in of the wall to the underlying rock may provide some additional passive restraint to the tyre wall and piles/ pile cap.

All of the above factors had a significant effect on the resulting structural actions- particularly the shear behaviour of the underlying piles in the sandstone rock.

2.2 MODELLING THE REDUCTION OF ABUTMENT PASSIVE RESTRAINT

FEA was used to quantify the interaction effects and global stability of the abutment wall.

Modelling was initially undertaken to assess the impact of the reduction in passive support to the abutment during the backfilling of the stem wall. The revised geometry was shown to be adequate for short term load cases in terms of global overturning and sliding to allow backfilling to commence on site. The long term design geometry with the 0.5m inclined RC wall (as shown in Figure 9) was also investigated for suitability and shown to be adequate.

An extract of the 2D Finite Element Model is shown in Figure 10. The model comprised 43 stages which incorporated every stage of the construction sequence. The spacing of the abutment piles was such that soil arching could be neglected and plane strain analysis was considered sufficient. All structural elements, including those supporting the bridge (i.e. piles and anchored concrete headstock) were modelled as solid concrete volume elements with a linear elastic material model and the corrected stiffness to reflect the per metre spacing (where applicable).

During construction, a simple but strict monitoring regime involving routine survey measurements was implemented on the abutment stem wall. This wall monitoring data provided further understanding of the actual abutment wall behaviour during backfilling.

On several occasions, the stem wall movements reported on site were in excess of those calculated. The geotechnical model was therefore progressively refined to reflect actual site conditions in an attempt to match site survey data to within a reasonable degree of accuracy. Interrogation of the model was primarily driven by a structural concern of the supporting abutment piles and the criticality of the bridge construction on the overall project competition. The FEA model became a tool for validating and assessing the construction methodology observed, with output structural forces in the supporting abutment piles extracted to compare with design capacities.

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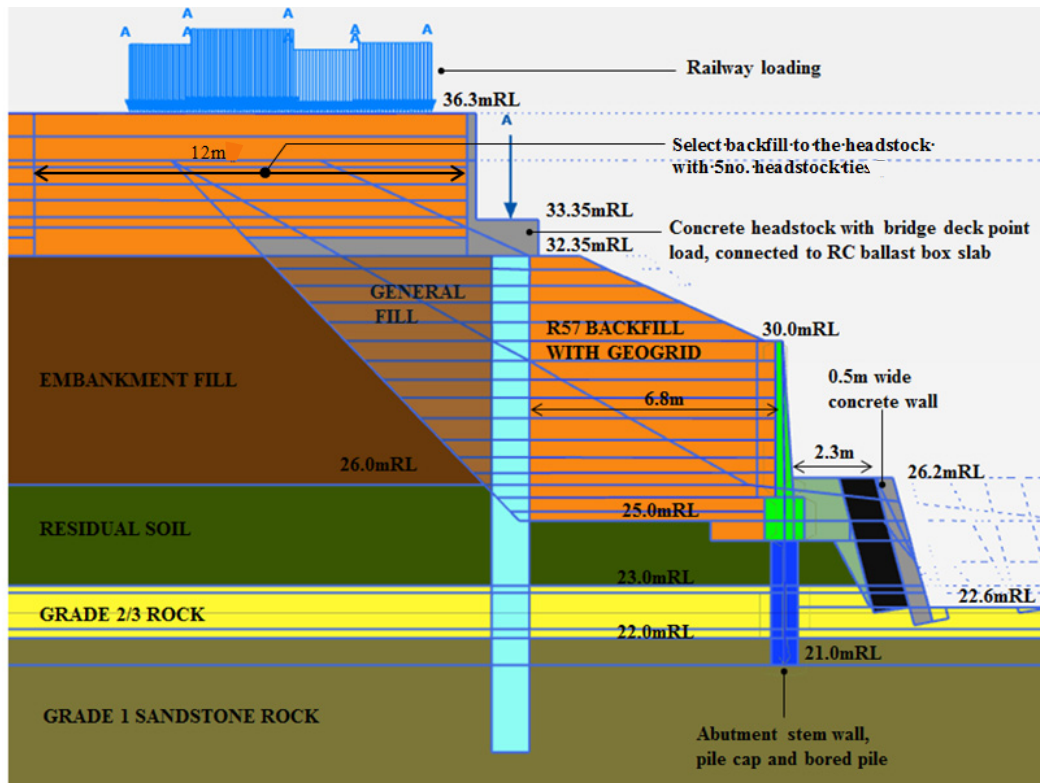


Figure 10: 2D Model Configuration (full geometry at the final and permanent construction stage)

Three key modelling alterations that were made to enable matching the calculated displacements with the reported stem wall movements were:

1. **Construction plant behind the wall**, represented as a vertical surcharge, was shown to be a key variable in the stem wall movements- particularly the proximity of large vibrating plant directly behind the wall.
2. **Modelling the abutment stem wall backfill in a way that retained the effects of the stress history.** The use of the (small strain) Hardening Soil Model (HSM) allowed for the variation in soil stiffness on unloading/reloading (i.e. the effects of compression hardening) and the irreversible plastic strains due to successive compaction plant loading.
3. **Modelling the individual layers of geogrid** as plate elements with sequential backfill lifts and the compaction plant altered the active pressures on the stem wall as construction progress.

By modelling the abutment wall and associated earthworks together, a better understanding of the global soil structure behaviour was gained.

3 MODELLING RESULTS

Comparison was made between:

- The original design geometry, as shown in Figure 8
- The revised design geometry, as shown in Figure 9

All assumptions and calibrations, such as the backfill sequence and compaction plant loading, were applied to both models.

The results in the permanent loading condition indicated that if the full passive resistance had of been left in front of the abutment, the stem wall deflections and notably the shear forces in the pile would have been reduced- as shown in Figure 11 and 12 respectively. Peaks shown in the results indicate changes in stiffness at the stem wall and pile cap; the pile cap and supporting abutment piles and the change in rock between a weathered to more competent sandstone.

The abutment pile behaviour was shown to be sensitive to the rock head level, rock stiffness and the passive support. However, the impact of the passive wedge reduction was shown to be minor and the restraint provided by the interlocking tyres was sufficient to maintain stability. The abutment wall configuration was robust enough to allow for the alterations on the passive side without overstressing the underlying piles. Analysis indicated that it was actually the proximity and size of the vibratory compaction plant to the back of the wall which was the principal reason for the higher than expected movements observed during the stem wall backfill.

The cumulative total displacement contour plot is shown in Figure 13. This is shown for the final analysis check using the factored (serviceability) design parameters but applying reduction factors to mimic creep in the geogrids and a reduction in the concrete stiffness to allow for cracking (and moment redistribution) within the concrete elements of the abutment system. The new RC wall was shown to have negligible deflections as the majority of movement had already taken place during the backfilling stage of construction, prior to casting the wall. The intent of the 0.5m thick RC wall in front of the tyre wall was therefore to provide long term integrity to the abutment by preventing damage due to erosion or undermining.

By demonstrating the alternative solution, economic benefits were provided including allowing the construction program to proceed as desired and removing the need to import fill and re- built the original design geometry.

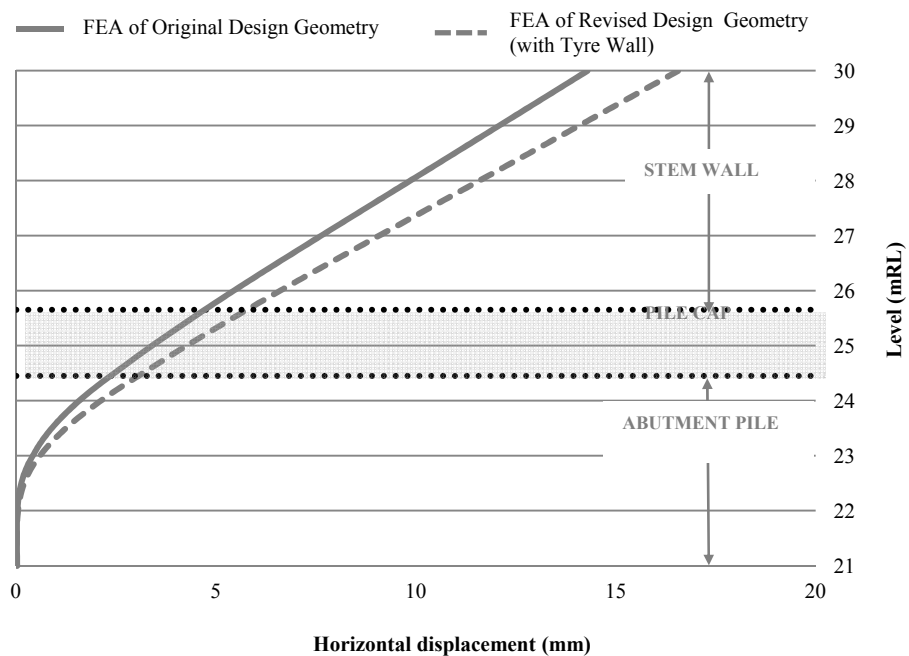


Figure 11: Cumulative horizontal displacement comparison (all construction stages)

4 MODELLING DISCUSSION

In comparison to the original geotechnical calculations, there were significant analytical differences which may in part explain the reason for the global stability which is not immediately apparent by inspection. In the original geotechnical assessment (i.e. where no FEA was undertaken), the piles were designed using limit state equilibrium that assumed fully mobilised active and passive states. FEA incorporates the notion of plasticity, altering the theoretical active limit by allowing for changes in the material stress paths, the beneficial inclusion of the geo-grids to lower the driving force on the wall and the resulting pressures from non-linear geometry and general soil structure interaction within the complex system.

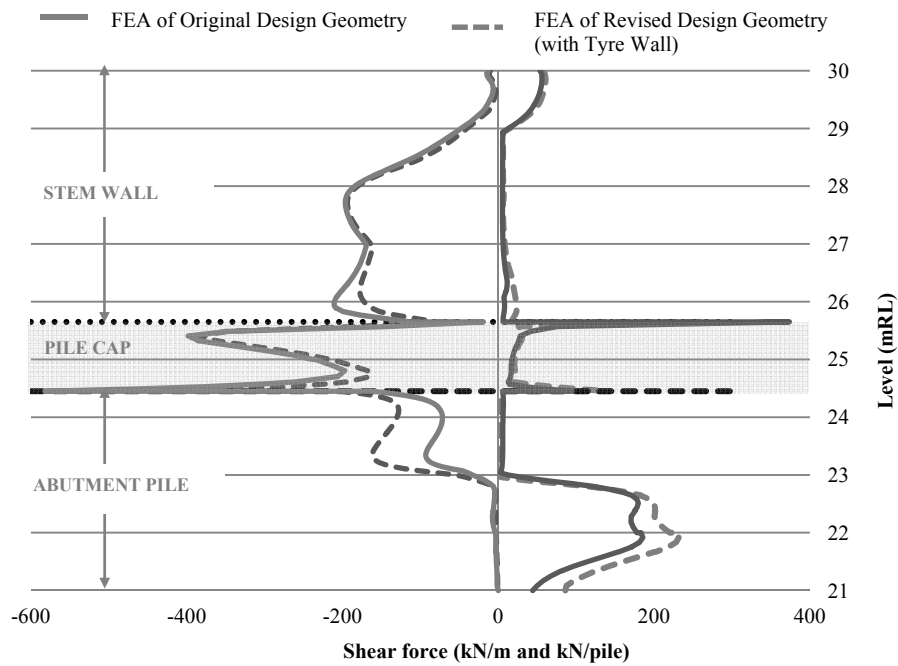


Figure 12: Shear force envelope comparison (all construction stages)

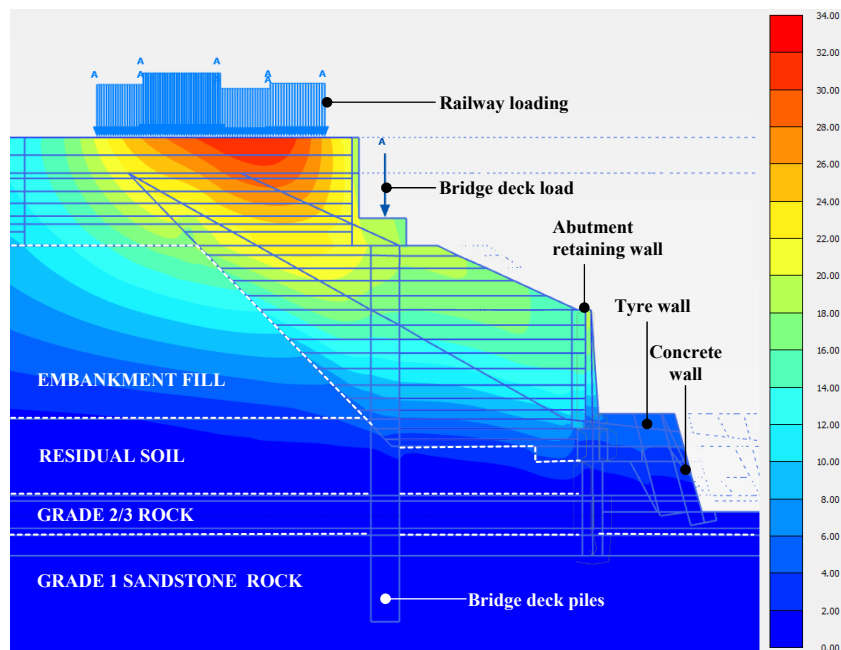


Figure 13: Cumulative displacement contour (displacements in mm)

The overall system stability can be understood as follows:

- The train loading to the headstock and the bridge deck loading to the bridge deck piles can be de-coupled from the abutment retaining wall. This is principally due to the following:
 1. Adequate restraining of the headstock with passive tie bars mitigates lateral movements from the horizontal train braking loading and the lateral earth pressure effects from the vertical train loads.
 2. Vertical loading from the bridge deck is transferred down the pile which is bearing on competent sandstone rock.

- The abutment retaining wall is impacted by the lateral earth pressures from the reinforced backfill and the widening earthworks plus temporary effects from compaction plant. However, the combination of the geogrid layers with the compacted backfill forms a coherent mass which serves to reduce the driving forces acting on the stem wall, thereby preventing a fully active wedge developing.
- Since the geogrids and backfill form a monolithic entity behind the abutment wall, in the long term, the abutment wall piles are essentially not loaded to the limit of the conventional lateral earth pressures. The combination of the geogrids and rock socket fixity keeps the wall from rotating enough to passively fail (mobilise) the backfill- tyre wall arrangement. As such, the passive wedge comprising the tyre wall and the inclined concrete scour protection wall are primarily there for protection against erosion. Small horizontal shears and deformations observed in the tyre wall during construction were considered to have no impact on the long term global stability. Flexibility within the system allows for re-distribution of stress as the tyre wall finds an equilibrium state.
- The piles for the abutment retaining wall are founded in rock and the shallow rock profile prevents any deep slip- circle failure mechanism occurring. Lateral loading on the abutment wall is shared between the passive resistance of the tyre wall and the pile socket in rock. Shear capacity of the pile governed the stability analysis criteria during the backfilling and with reference to the calculated structural forces, the piles were demonstrated to remain within the allowable design limits.

By using an analysis technique which took into account the interaction effects of the structural elements, the loading conditions and the underlying geology, the revised geometry was demonstrated to not be detrimental to the overall abutment (and bridge) stability.

4 CONCLUSIONS

Space constraints and programme requirements during early site works led to over excavation of the residual soil and rock material in front of the abutment wall, typically used to provide restraint to lateral movement. Analysis was undertaken to check that the stability of the abutment during temporary works and in the permanent condition. The abutment was shown to be structurally and geotechnically adequate, in terms of strength, serviceability and durability. The abutment wall was shown to be structurally robust enough to permit the change in geometry on the passive side, where shear forces developing in the supporting abutment piles were found to be the critical design action.

Conclusions drawn from this project experience and the modelling process are:

1. The reduction in the passive wedge was shown to have minimal effect on the abutment performance. This was demonstrated by using advanced soil models; incorporating structural interactions, the actual geometry and survey data obtained at numerous stages throughout the backfilling process. Detailed analysis more realistically calculated the lateral earth pressures compared to conventional limit state design approaches.
2. The temporary retaining wall comprising of recycled tyres was shown to be a cost effective, easy to install solution with sufficient load carrying capacity.
3. The reinforced concrete wall primarily provides scour protection to the tyre wall.
4. Monitoring of the wall movements with trigger levels at the top and bottom is considered a minimal requirement prior to any backfilling commencing.
5. The type and proximity of vibratory compaction rollers has a significant effect on earth pressures developed in the backfill and hence lateral wall movements- particularly for un- propped walls.
6. The underlying pile behaviour is impacted by the rock level, rock stiffness, the degree of restraint provided by the backfill/ tyre wall and most importantly the lateral stresses developed during backfilling. FEA demonstrated a reduction in lateral stress within the high- quality backfill (modelled using the HSM) and that the geogrid reinforcement was sufficient to mitigate the impact of vibrating compaction plant observed close behind the stem wall.

Value may be provided on future bridge projects by anticipating constructability requirements and better accounting for the benefits of soil reinforcement and favourable geology with holistic FEA modelling. Economic benefits by reducing the passive support requirement (i.e. lowering the volume of imported fill) may be applicable in other situations after sufficient soil- structure interaction analysis has been undertaken. An understanding of the risk and reward (in terms of

programme speed and level of analysis work) needs to also be considered during the process of achieving an economically designed structure.

5 ACKNOWLEDGEMENTS

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6 REFERENCES

ARTC RCP 01: *Earthworks Construction Procedures*

ARTC BDS 20: *Configuration Standard- Bridge Ends*

Australian Standards AS 5100.3- 2004 *Bridge Design- Foundations and soil supporting structures*, Section 8.2

AS5100.2-2004: Australian Standard *Bridge Design- Part 2 Design Loads* (Section 13)

AS5100.3 Supp 1- 2008: Australian Standard *Bridge Design- Part 3 - Foundations and Soil Supporting Structures- Commentary* (Section C8.2)

BS 8002:1994: *Earth Retaining Structures* (no longer current by cited in UK Building codes)

BSI PD6694-1: *Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004*

Eurocode 7: *Geotechnical Design- Part 1: General Rules* (BS EN 1997-1:2004)

Ingold, T. S. (1979), "Effects of compaction on retaining walls", *Geotechnique*, Vol. 29, No. 3, pp. 265–283

Roads and Maritime Services (RMS) QA Specification R58: *Construction of Reinforced Soil Walls (Contractor's Design)*

RTA QA Specification R57: *Design of Reinforced Soil Walls*

UK CIRIA C580: *Embedded retaining walls- guidance for economic design*