

Geotechnical Design of Non-Conventional Bridge Abutments at the Hakao Gully in Tauranga, New Zealand

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ABSTRACT

The Takitimu North Link (TNL) Stage 1 project delivers 6.8 km of new expressway: connection between State Highway 2 (west of Te Puna) to State Highway 29 (Takitimu Drive) and upgrade to the connecting stretch of State Highway 29 into Tauranga. The project is part of the government's New Zealand Upgrade Programme (NZUP).

The Minden Gully Bridges are a set of three separate structures designed and constructed for this project, comprising of mainline, on-ramp and off-ramps bridges, all located within the same 25-40 m high, steeply sloped Hakao gully. The bridge abutments were formed as piers, connected to the ground behind with settlement slabs bearing on reinforced soil slopes. The geology of the site was found to be highly variable between the east and west sides of the gully, as well as some variation from north to south. As such, various refinements and sub-layering were completed as part of the geotechnical design to adequately capture site soil conditions. The complex geology and topography of the site, together with the selected bridge abutment arrangements, called for specific geotechnical design for each bridge, including achieving compatibility between each geotechnical element and soil-structure interaction for the design of shear pile ground improvement.

Keywords: bridge, gully, abutment, shear piles, reinforced soil slope

1 INTRODUCTION

The TNL Stage 1 project is being delivered by Fulton Hogan Limited and HEB Construction Limited as a Joint Venture (JV) under a design and construct contract. The design of Takitimu North Link Stage 1 is being delivered by Beca Ltd (Beca), as lead designer, with Holmes Consulting Group Limited Partnership (Holmes) as a subconsultant. This project includes the design and construction of eight bridges, three million cubic metres of earthworks, 29 culverts, eight stream diversions, seven wetland areas, and a continuous and separate shared 'active mode path'. This project will significantly improve safety and ease congestion for the existing State Highway 2, in addition to supporting urban growth, providing more transport choices, and building resilience.

The Minden Gully Bridges are located adjacent to a full grade-separated interchange at Minden Road, designed to carry the mainline, an on-ramp lane and an off-ramp lane on three separate bridges over the Hakao gully. The bridges are typically 130-140 m long, three-span with foundations composed of 710 mm diameter driven steel pile casings, infilled with concrete. Refer Figure 1 for site plan.

Liquefaction assessments, slope stability assessments and pile design under seismic design loads were undertaken for the bridges as part of the design; however, this paper will focus on the discussion regarding performance of the bridges under static design criteria.

2 BRIDGE ARRANGEMENTS

2.1 Bridge Form

Balanced bridge forms and suitable lengths for the end spans were important for the structural design. As such, the abutments of all three bridges are composed of a piled abutment with 6-7.5 m long settlement slabs that extend back to a reinforced soil slope. With this arrangement, more detailed geotechnical design was required compared to conventional abutment design.

The abutments and piers are founded on 710 mm diameter driven steel pile casings infilled with concrete and partially reinforced. Oversized casings 5 m in length with a clear annulus between the sleeves and the piles were proposed for the abutment piles to reduce the demands caused from superstructure thermal and time dependent shortening strains, as well as reducing seismic demands. The abutment arrangement is shown diagrammatically in Figure 2.

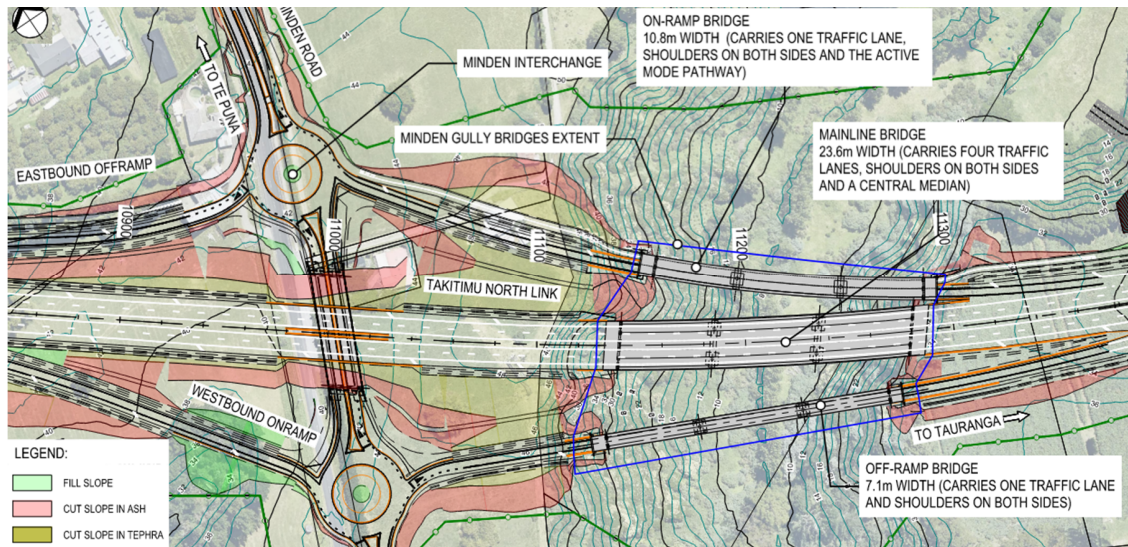


Figure 1: Site Plan

2.2 Settlement Slab Loads

The free end of the settlement slab rests on a concrete slab, with an expansion joint provided between these two elements. Through this arrangement, the reinforced soil slope is subject to axial and lateral loads from the settlement slab. These loads are provided by the structural engineer and comprise of:

- Axial – from the dead load of the concrete slab and settlement slab.
- Axial – from live loads (traffic on the settlement slab).
- Lateral – from thermal contraction of the bridge due to changes in temperature throughout the day. It arises from friction between the settlement slab and the abutment concrete slab. It is a component of the dead load and has a load direction out-of-the-slope (i.e., longitudinal to the bridges alignment).

Various load combinations for these loads are considered in the stability assessments and reinforced soil slope design. These load combinations are as follows:

- Static ULS (long-term) case
- Static SLS (short term): HO (overload) traffic case
- Static SLS (short term): elevated groundwater case
- Static SLS (short term): combined thermal effect and traffic case
- Seismic cases

Behind the settlement slab, typical traffic loads from the Bridge Manual (Waka Kotahi NZ Transport Agency, 2018) are applied: HN (normal) traffic loading of 12 kPa and HO (overload) traffic loading of 24 kPa. The arrangement of loads is shown diagrammatically in Figure 2.

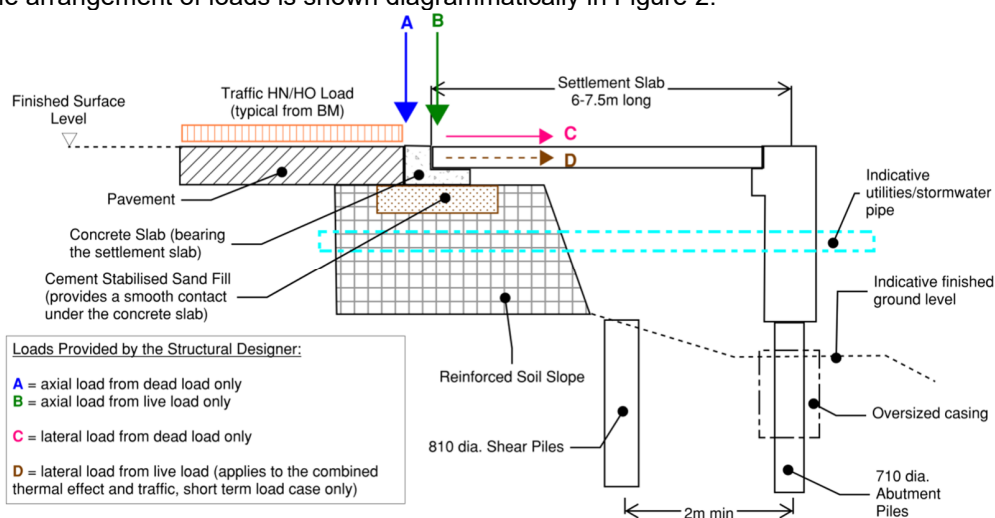


Figure 2: Typical abutment arrangement and settlement slab loads (shown diagrammatically)

3 GEOLOGY SETTING

The geology of the site generally comprises of Matua Subgroup - Interbedded (unit T3b) soils overlying Pakaumanu and Waiteariki Ignimbrite with surficial layers of Colluvium (unit T1b) on the gully slopes and Alluvium (unit T1a) deposits at the gully floor.

The Matua Subgroup - Interbedded is described as highly variable deposit comprising fluvial pumiceous and rhyolitic silts, sands, and gravels, lacustrine and estuarine muds, lignites, and peats, intercalated with airfall tephra and thin distal ignimbrites. The Pakaumanu Ignimbrite is described as non-welded to partially welded ignimbrite. The Pakaumanu Ignimbrite is divided into two units. The finer unit (unit T5a) is a completely weathered to highly weathered ignimbrite comprising firm to stiff clayey silt to silty sand, and the coarser unit (unit T5b) is a highly weathered ignimbrite comprising medium dense to dense silt to sand. In general, the finer unit typically underlies the coarser unit. Finally, Waiteariki Ignimbrite (unit T6) is typically highly to moderately weathered, partially welded to welded Ignimbrite (Briggs, 1996).

A series of ground investigations were undertaken in stages between 2011 and 2021, comprising of 15 machine boreholes (maximum depth of 55.7 m bgl), 10 CPTs, 4 sCPTs, 5 test pits, and 7 hand augers. Geological sections were generated from the three-dimensional site-wide ground model developed within the Leapfrog Geo software package. Longitudinal geological sections were taken along the bridge alignment and selected north and south edges of each of the three bridges, such that variation in topography and geology could be taken into consideration in the design. One of the longitudinal geological sections, located at the centreline of the mainline bridge, is shown in Figure 3.

4 ENGINEERING SOIL PROFILE

Varying soil strengths and behaviour within the same geology unit were encountered between the east and west sides of the gully. Refinements and sub-layering of these units were carried out to adequately capture site soil conditions therefore providing robust design for the lateral capacity of the bridge and shear piles, as well as slope stability assessments.

4.1 Matua Subgroup – Interbedded

The Matua Subgroup – Interbedded (T3b) was interpreted to have variable undrained shear strengths between the gully floor and slopes. Therefore, three different undrained shear strengths are assigned to the gully floor, east and west gully slope.

A pocket of Matua Subgroup – Interbedded was identified beneath the Pakaumanu Ignimbrite along the alignment of the on-ramp bridge. This soil sub-unit was described as predominantly sandy; hence this unit is interpreted as having “sand-like” behaviour and exhibits different behaviour to the shallower Matua Subgroup – Interbedded unit which mainly comprises of silts and clays. This unit was named Matua Subgroup – Interbedded – North (T3bN) and is only applicable at the on-ramp bridge. Although not discussed further within this paper, the presence of this material posed an interesting challenge to the piling works for the on-ramp bridge Pier B. It also provided further evidence of the complexities of ground conditions at this site.

4.2 Pakaumanu Ignimbrite

The Pakaumanu Ignimbrite – Coarse (T5a) unit was found to be variable across the gully. To account for this variability, the Pakaumanu Ignimbrite – Coarse was divided into three sub-units:

- Pakaumanu Ignimbrite – Coarse – Upper (T5a1): this sub-unit was identified at the west gully slope and interpreted as predominantly clayey, with relatively low strength (N-SPT ranges between 0 and 6) and interpreted to overlie a more competent unit of the T5a2. Consequently, this unit was modelled as exhibiting softer lateral soil springs compared to the other sub-units.
- Pakaumanu Ignimbrite – Coarse – Middle (T5a2): this sub-unit was identified as predominantly medium dense to dense silty sands with median N-SPT value of 35.
- Pakaumanu Ignimbrite – Coarse – Lower (T5a3): this sub-unit was interpreted as having similar behaviour with T5a2 with higher strength considering median N-SPT value of 50+. If T5a2 and T5a3 are not separated, the slip circle will pass through N-SPT=50+ layers. Separating this unit with T5a2 allows for higher soil parameters to be assigned, hence resulted in realistic slip shapes and sizes for the slope stability assessment. Therefore, a compatible solution between the design of bridge piles, shear piles, and the slope seismic displacements was achieved. Additionally, this sub-unit was targeted for the toe of the bridge abutment piles, providing adequate fixity as required by the structural design.

The Pakaumanu Ignimbrite – Fine (T5b) unit was found to have differing soil descriptions within the site and hence was divided into two sub-units:

- Pakaumanu Ignimbrite – Fine – West (T5bW): this sub-unit was interpreted as predominantly clayey with median N-SPT value of 20.
- Pakaumanu Ignimbrite – Fine – East (T5bE): this sub-unit was interpreted as predominantly sandy with median N-SPT value of 50+.

4.3 Waiteariki Ignimbrite

The Waiteariki Ignimbrite (T6) unit was found to have differing soil descriptions and strengths. Therefore, this unit was divided into two sub-units:

- Waiteariki Ignimbrite – West (T6W): this sub-unit was interpreted as having “clay-like” behaviour with a median N-SPT value of 19 and applicable to the west gully slope only. While the slip circle is unlikely to reach this sub-unit, separating this sub-unit allows for modelling soft lateral springs at depths and assigning lower end bearing and skin friction capacity for the abutment piles.
- Waiteariki Ignimbrite (T6): this sub-unit was interpreted as having “sand-like” behaviour with a median N-SPT value of 50+.

4.4 Engineering Long Section

Longitudinal engineering sections were developed from the geological long sections based on the refinements discussed above. Figure 4 shows the development from geological section to engineering section for the Mainline bridge at the centreline.

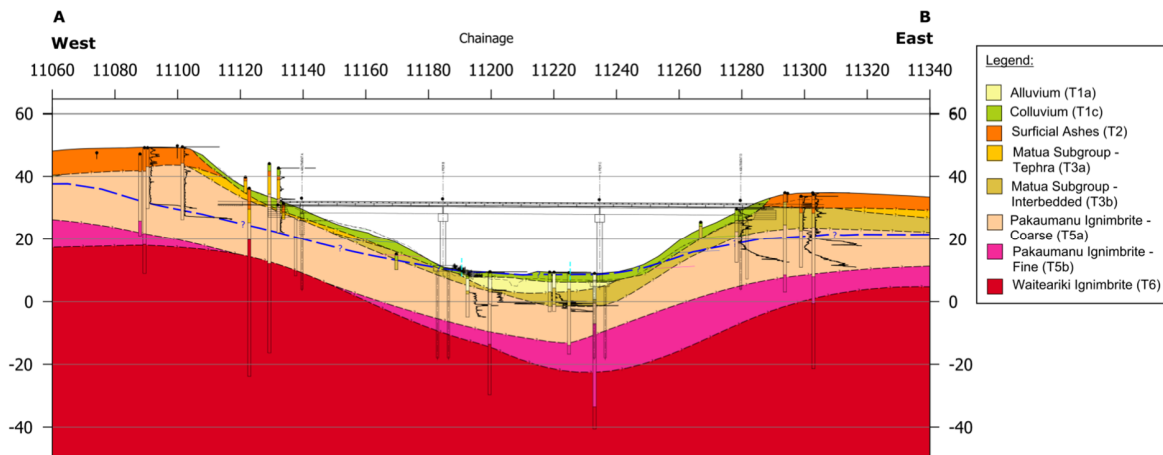


Figure 3: Geological Section of the Mainline Bridge, taken at the centreline

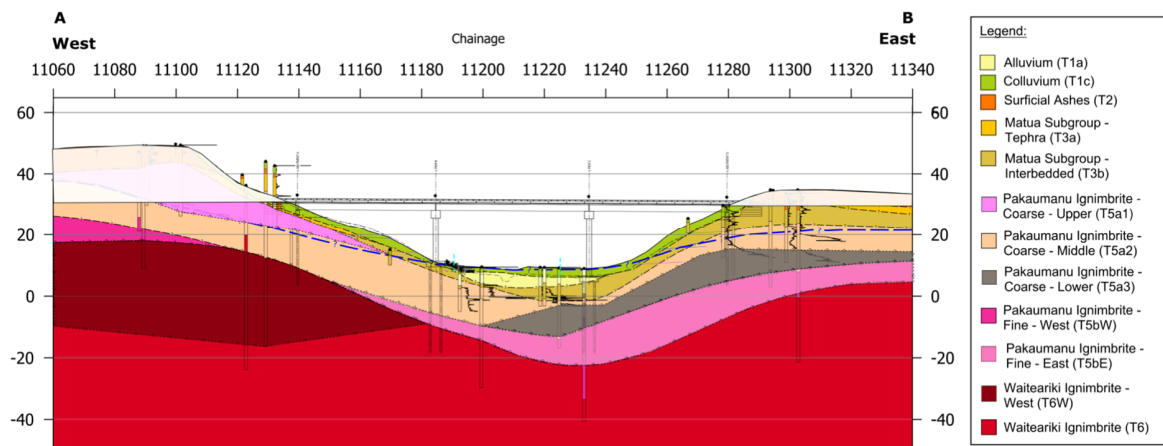


Figure 4: Engineering Section for the Mainline Bridge, taken at the centreline

5 SLOPE STABILITY ASSESSMENT AND SHEAR PILES GROUND IMPROVEMENT

Stability analyses were carried for static, seismic, and post-seismic design cases using the Geostudio Slope/W software package to assess the performance following requirements described in the Bridge Manual. Different engineering long sections between the abutments were considered to capture variability in topography affecting the critical sections. Figure 5 shows an example where the critical slope stability assessment locations for each abutment are different for the Off-ramp bridge.

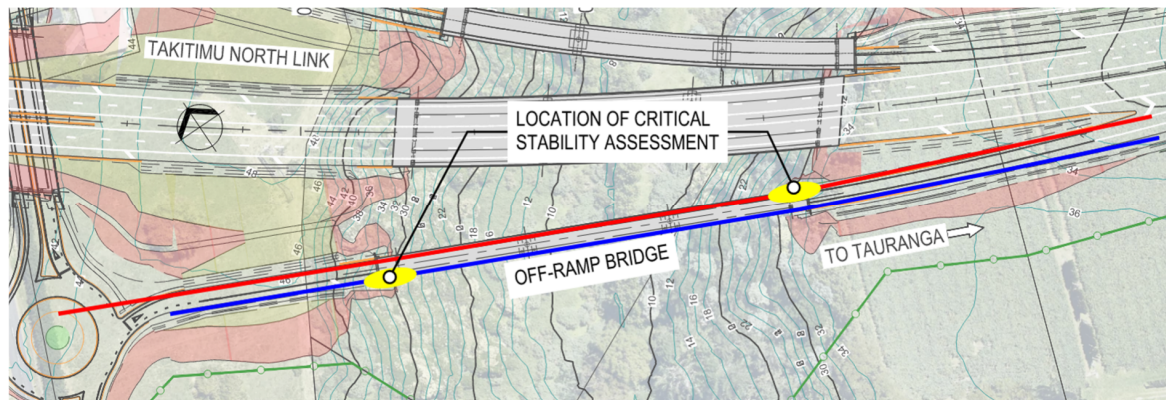


Figure 5: Location of critical stability assessment for the Off-ramp bridge

The settlement slab loads discussed in Section 2.2 are applied at the top of the reinforced soil slope. A row of 810 mm diameter driven steel pile casings at each abutment is required as shear piles to increase the long-term static factor of safety of the slopes at the abutments to the required value of 1.5, noting that the bridge piles are not considered to provide any pinning effect onto the abutment soil. The slope stability analyses were used to determine the required shear capacity of the shear piles.

It is noted that the factor of safety of the slopes at the abutments under static long-term conditions, without the shear pile ground improvement, is 1.3 and the target factor of safety of 1.2 is achieved under all static short-term conditions without the shear pile ground improvement. Hence the static long-term case became the critical case for slope stability assessment.

Although, theoretically, no slope displacements are anticipated to develop for slopes with a minimum factor of safety of 1.5, a compatibility assessment was carried out in conjunction with the structural designers. Soil-structure interaction analyses was undertaken to consider the required shear resistance onto the structural model by applying a displacement to the shear piles, thus confirming a compatible solution. The required displacement was applied as individual displacements to the pinned ends of the soil link elements (i.e., below the slip circle). The shear piles were modelled by the structural designers in CSi Bridge software package, using nominal lateral soil spring values developed by the geotechnical design using LPILE software package. Figure 6 shows illustratively the soil-structure interaction within the shear piles.

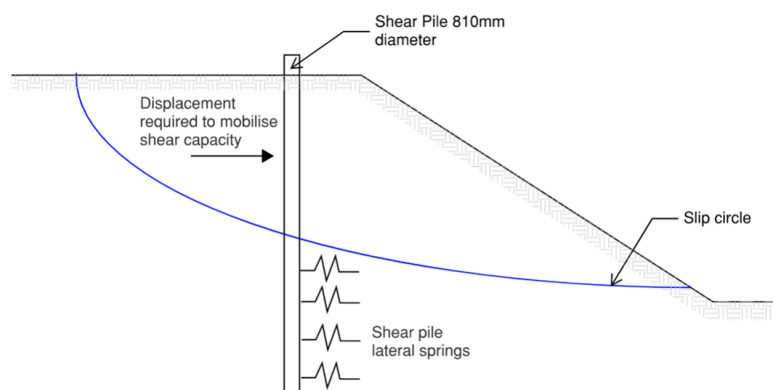


Figure 6: Soil-structure interaction in shear piles

An estimated displacement of 25-100 mm is required to mobilise the required ultimate shear capacity. Although it is considered unlikely that these displacements will occur (considering the static factor of

safety without shear piles is 1.3), this soil displacement profile was considered as a static demand for the abutment piles after the required shear resistance has been generated.

6 REINFORCED SOIL SLOPE

The reinforced soil slope runs for the full width of the settlement slab and returns parallel to the carriageway. It has a 70° face angle (from the horizontal) with a gabion basket finish on the face (achieved via Terramesh facing units infilled with rock). HDPE geogrids provide reinforcement to the slope. GAP65 hardfill (cement stabilised at the wrap arounds) was specified as backfill to provide a robust soil structure.

The reinforced soil slope design was carried out using MSEW+ software package. The relevant AASHTO (2020) load factors were applied onto the settlement slab loads discussed in Section 2.2, as well as traffic load behind the settlement slab and pavement fill. The short-term combined thermal effect and traffic case was found to be the critical case for the reinforced soil slope design due to high lateral force generated from the settlement slab. The direct sliding between geogrid and hardfill generally dictated the minimum geogrid length requirements. As such, cement stabilised sand fill (Refer Figure 2) was proposed to provide an even and homogeneous surface under the concrete slab to reduce the lateral load onto the reinforced soil slope. Additionally, the top two grids were proposed to be longer than the lower grids to support the loads from the settlement slab.

The set out proposed for the reinforced soil slopes are unique to each bridge due to the irregular interface with the existing gully slopes, presence of utilities and stormwater pipes crossing through the slopes, the longitudinal and transverse fall of the road, as well as the bridges' skew angle. Additional Terramesh blocks were proposed locally at some locations to manage these variabilities, hence the height of the reinforced soil slope varies between 3 m and 4.5 m.

Continuous reinforced soil slope was proposed for the mainline and on-ramp east abutments due to the limited space between two abutments. This design was achieved by taking into consideration the difference in elevations, potential clashes with the adjacent bridge structure, and the existing gully slope. The set out for the reinforced soil slope also influenced the set out for the shear piles. The shear piles were proposed to have minimum 2 m spacing from the bridge piles (Refer Figure 2), however at some location this is only achievable if the shear piles are below the toe of the reinforced soil slope due to limited room. The design identified the affected shear piles and proposed specific top of shear piles elevation for piles below the reinforced soil slope.

7 CONCLUSION

The geotechnical elements of the Minden Gully Bridges have been successfully designed by taking the variability of ground conditions, topography, and unconventional abutment form into consideration. Refinements of engineering soil profile and sub-layering were carried out to adequately capture site soil conditions for geotechnical design, particularly for achieving compatibility between the bridge pile design, shear pile design, and slope stability assessment.

The loads imposed by the settlement slab onto the reinforced soil slopes are considered for the slope stability assessment and reinforced soil slope design. Different critical cases were encountered between the two analyses, where static long-term was the critical case for the slope stability and the short-term combined thermal effect and traffic case for the reinforced soil slope design. Bespoke reinforced soil slope set out was provided for each abutment as part of the design to handle irregularities of the gully profile as well as various design constraints.

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