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Design of Mechanically Stabilised Earth Walls for Bridge Abutments on the Mackays to Peka Peka Expressway, New Zealand

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ABSTRACT

The new 18 km long, four lane MacKays to Peka Peka Expressway located in the Kāpiti Coast Region of New Zealand is currently under construction. On this project there are 18 bridges which incorporate Mechanically Stabilised Earth Walls (MSE walls) in some form within the bridge abutments. These walls are designed for the high seismicity of the Kāpiti Coast region with a design peak ground acceleration (PGA) of up to 0.98g under ultimate limit state design events. This exceeds typical design practice and presents a significant challenge to wall designers. Furthermore extensive sections of the route are prone to flooding, liquefaction and have complicated geometry (for example highly skewed bridges with piles going through the MSE wall reinforced block). This paper discusses the philosophy and approach adopted for the walls and how these design challenges were overcome.

Keywords: Retaining Walls, Seismic Design, Displacement, Liquefaction

1 INTRODUCTION

The Mackays to Peka Peka (M2PP) Expressway is an 18km four lane expressway. It is the first section of the Wellington Northern Corridor project under construction. The expressway passes through the Kapiti Coast region which is located 60km north of Wellington. Once complete it will separate local and highway traffic, resulting in safer and faster trips through the region. The project includes 18 bridges which utilise MSE walls for the bridge abutments as well as a number of other minor MSE structures including abutments for shared cycleway, walkway, bridleway and maintenance access bridges and freestanding MSE walls. This paper addresses the walls associated with the highway bridge structures.

A number of interesting design challenges were encountered by the team undertaking the MSE wall design on this project. The purpose of this paper is to describe these challenges and the innovative solutions that were adopted to overcome them. The challenges included:

1. The high design seismic loading. The Kapiti Coast area has one of the highest seismic loadings in New Zealand. This, in conjunction with the low annual probability of exceedance required for M2PP expressway, results in peak ground acceleration values approaching 1g.
2. The potential for flood events to cause inundation of the MSE walls. This presents a risk of wall undercutting or erosion of the dune sand used in wall construction. Such erosion could cause significant damage to, or even collapse of, the roadway.
3. The potential for liquefaction within sands underlying the wall. Liquefaction of underlying soils, particularly if occurring while strong shaking was ongoing, has the potential to result in large lateral displacements of bridge abutments.
4. The potential for differential settlement of compressible soils beneath the MSE walls. Differential settlement might result in poor driver comfort and damage to rigid elements such as precast wall panels and flood protection measures causing increased maintenance costs.
5. The bridge design requires foundation piles to pass through the reinforced block. This presents challenges in maintaining integrity of the reinforced block where the reinforcement is disrupted.
6. The adoption of highly skewed abutments for some bridges. A number of bridges along the alignment were skewed by up to 35 degrees. This results in geogrids clashing with other design elements making it difficult to achieve design embedment.

2 SEISMICITY OF THE AREA

The NZTA commissioned a site specific seismic hazard assessment (SSHA) for the expressway (M2PP Alliance 2014a). The SSHA determined design PGA levels, for the design of the main expressway bridges (1 in 2,500 year return period) of 0.98g for the southern part of the site (NZS1170.5 Site Subsoil Class C) and 0.74g for the central and northern sectors (NZS1170.5 Site Subsoil Class D).

3 GROUND CONDITIONS

The M2PP expressway is mostly underlain by dune sands and peats with alluvial sands and gravels at depth. Dune sands inter-finger with peat deposits in an unpredictable manner below the alignment. The water table is generally shallow, between 0 to 1m below ground, locally mounding where dunes rise up to 20m above surrounding ground. The high water table and seismicity make sand layers prone to liquefaction.

4 MSE WALL DESIGN

Uniaxial polyethylene geogrids were selected as MSE walls reinforcement for their historical good performance under seismic loads, ready availability, and familiarity of designers and constructors with product and commercial considerations. Dune sand was selected as the engineered fill due to its availability within the alignment, free draining characteristics and minimising truck movements along local roads. This was an important consideration due to the close proximity of residential zones.

MSE walls were designed in general accordance with AASHTO (2007, 2010) using the program MSEW 3.0 (Adama Engineering Ltd 2013) for external and internal stability. Figure 1 shows the six possible failure mechanisms analysed. Global stability was checked using Slope/W (Geo-Slope International Ltd 2014), a limit equilibrium program used for calculating circular slip surfaces. Seismic displacements were estimated using this program following (Jibson 2007). Plaxis 2D (Plaxis 2013) (a finite element analysis program) was used to provide a comparison against limit equilibrium methods and to assess the distribution of displacements within the walls.

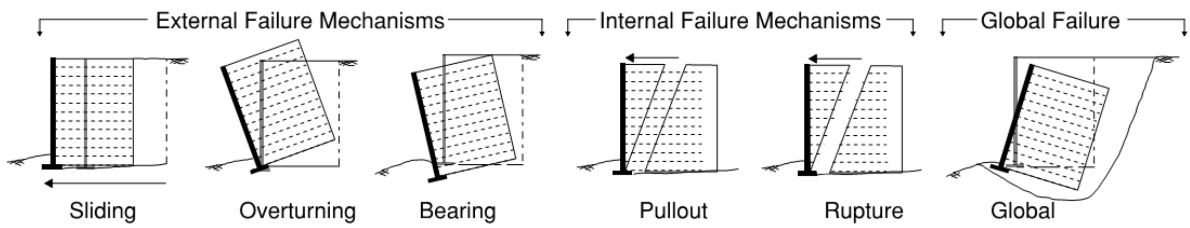

Figure 1. External, Internal and Global Failure Mechanism for MSE walls

5 GEOMETRY

In general, the abutment retaining walls comprise MSE blocks 5m to 9m high with the wall face sloping at angles to the horizontal between 70 and 90 degrees. Walls are reinforced in the longitudinal and transverse directions. In general terms there are two abutment types, a vertical abutment where the MSE is encapsulated within precast concrete panels or a spill through abutment where the grids are terminated within the embankment (refer to Figure 2 for typical wall geometry.)

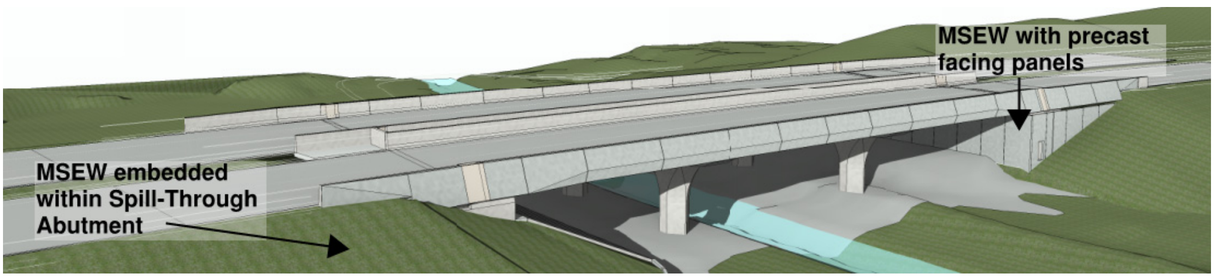

Figure 2. Typical MSE Wall Geometry (Wharemauku Stream Bridge)

6 FLOOD DESIGN

MSE walls adjacent to rivers are prone to inundation in flood events. Inundation of walls can cause a number of adverse effects, for example (1) Undercut of the wall, (2) erosion of wall fill and (3) rapid drawdown (additional hydrostatic load acting on walls). Figure 3 shows the design measures used to mitigate these effects. Specifically these measures included; (1) Rock riprap was placed upstream and downstream of each bridge with a minimum of least 600mm wall toe embedment below the underside of the riprap (frequently greater) to mitigate potential toe erosion, (2) A 1500mm wide cement stabilised backfill zone around the perimeter of the walls, (3) A geotextile to prevent horizontal migration of engineered fill, (4) A geotextile together with a 150mm thick filter layer to reduce vertical migration of engineered fill clogging up drainage layer, (5) A subsoil drainage pipe to lower flood levels as quickly as possible, (6) Slopes adjacent to walls were quickly vegetated to mitigate erosion from concentrated water flow. Areas that had not yet been vegetated suffered erosion during intense rainfall. Walls were also designed for rapid drawdown. External and internal failure mechanisms were checked with additional hydrostatic forces acting on wall.

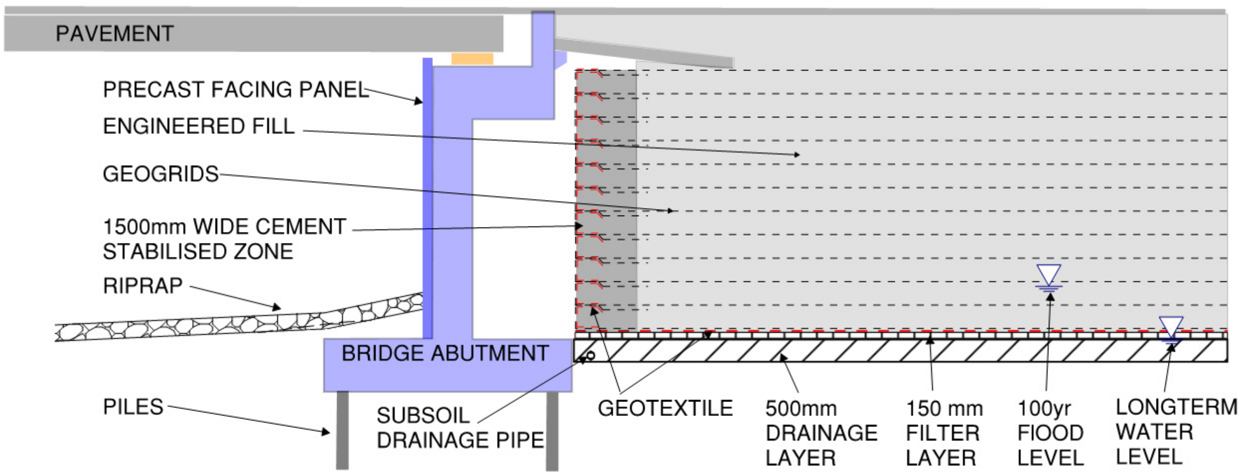

Figure 3. Flood Protection Design Elements

7 SEISMIC DESIGN

Seismic design was particularly difficult for this project due to high design accelerations and the potential for liquefaction and cyclic softening of foundation soils. It was not considered technically or economically feasible to prevent seismic induced displacements because of the site's high seismic acceleration and potential for strength loss in foundation soils. Displacement based design (allowing for movements of wall), with displacement limits in the region of 30 to 200mm effectively lowered the seismic forces considered in the wall design. Where displacement was excessive, ground improvement was specified. In this way ground improvement costs were optimised and a more efficient wall design achieved.

Seismic displacements were estimated using Slope/W following (Jibson 2007) method based on Newmark sliding block theory (Newmark 1965). The stability of the walls was checked in transverse and longitudinal directions and the distribution of displacements within the walls was assessed.

8 PILES THROUGH REINFORCED ZONE

8.1 Geogrid Layout

Some bridges required a secondary load path to provide vertical support to the superstructure to address the potential for loss of pile support following large abutment movements in a maximum considered earthquake (MCE) event. The secondary load path is provided by MSE walls extended beneath the bridge abutment and around the foundation piles. A consequence of this approach is that piles were required to extend through the reinforcement zone. The presence of piles reduces the coverage ratio locally at the wall face, furthermore because piles move laterally in a seismic event, an additional reduction must be made in the coverage ratio. Designing walls for this reduced coverage ratio required careful detailing, use of stronger grids locally and cement stabilisation.

Details A and B of Figure 4 show the detailing around piles. Namely (1) longitudinal grids (tie strips) placed between piles connecting to wall constructed behind piles, (2) Geotextile placed horizontally through tie strips in front of piles to support soil between precast panels and piles, (3) Cement stabilisation of all reinforced fill below bridge abutment capping beam (4) widening of the abutment beam to spread the deck load through a wider block of MSE.

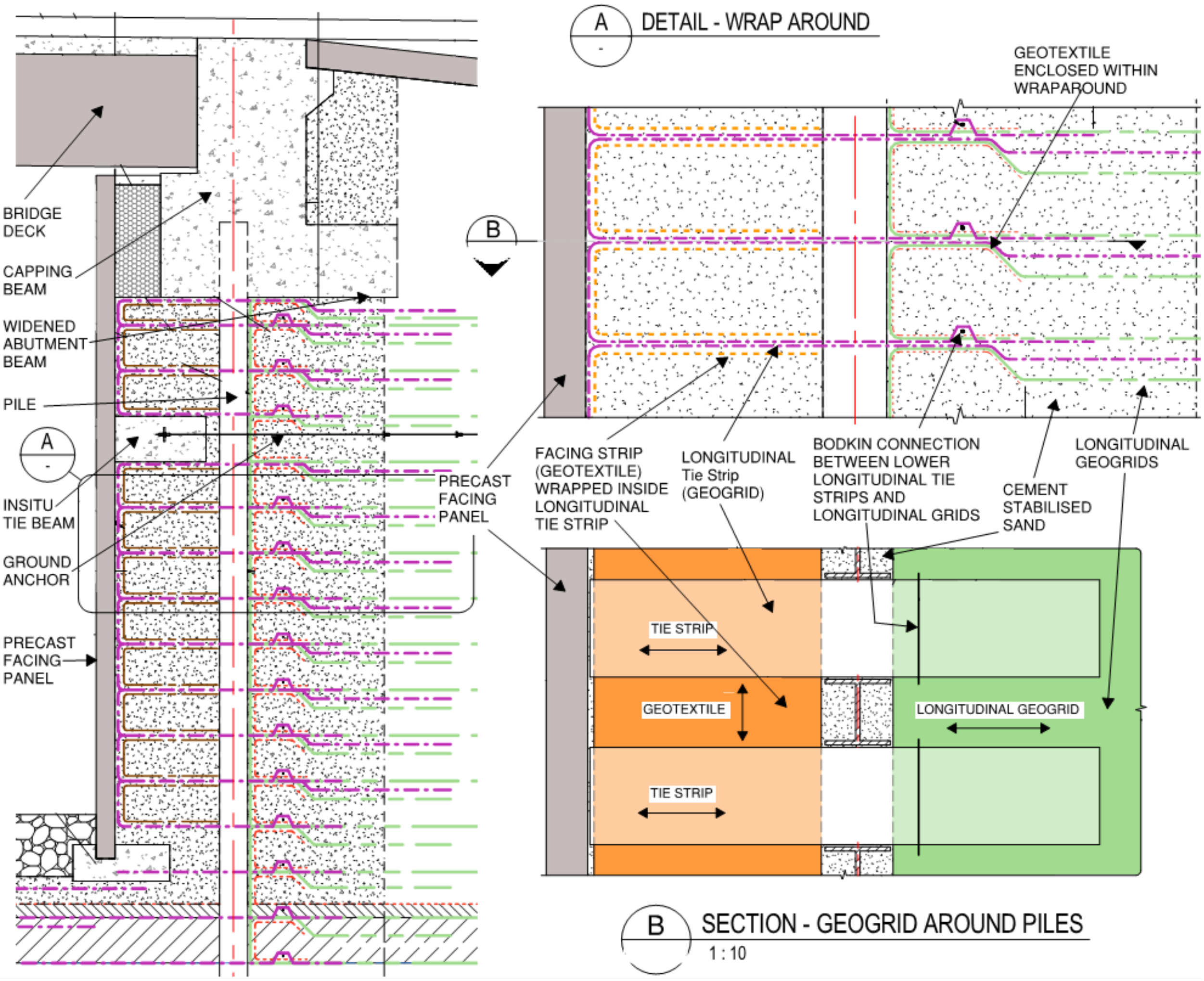

Figure 4. Section Piles through Reinforced Zone MSE Wall

9 GROUND IMPROVEMENT

Many bridges overlay soils that may liquefy during large earthquakes potentially resulting in large lateral displacement and bearing failure of walls. Furthermore settlement of compressible soft soils and liquefaction settlement could result in differential settlement resulting in poor driver comfort along the expressway and damage to rigid elements such as precast panels in front of walls and flood protection measures.

The extent of ground improvement was calculated from global stability analyses in Slope/W. The two main assumptions in this analysis were shear strength of bridge piles was ignored and improved ground was modelled using nominal soil parameters. It was assumed that ground improvement contributes only to the prevention of liquefaction, refer to Figure 5. To mitigate these adverse effects three ground improvement techniques were undertaken below MSE walls.

9.1 Densification of Soil

Liquefaction is more likely where loose to medium dense sands are present below the water table. To mitigate the effects of liquefaction the soil can be made denser by using rammed or vibro-compacted stone columns. This method is most effective for sandy soils with a relatively low silt content. It was the preferred ground improvement method on the M2PP project.

9.2 Shear Stiffening the Soil

Where sandy soils have a relatively high silt content densification methods have been found to be less effective. To mitigate liquefaction in these areas low strength concrete lattices on a regular grid were installed. This stiffens the soil in shear and reduces the risk of liquefaction.

9.3 Excavate and Replace

Where significant thicknesses of peat or fine grain deposits are present below bridge it was removed and replaced with engineered fill to reduce differential settlement effects on the walls. Excavation was carried out below the water table using GPS guided excavators and backfilled with cobble to boulder size material.

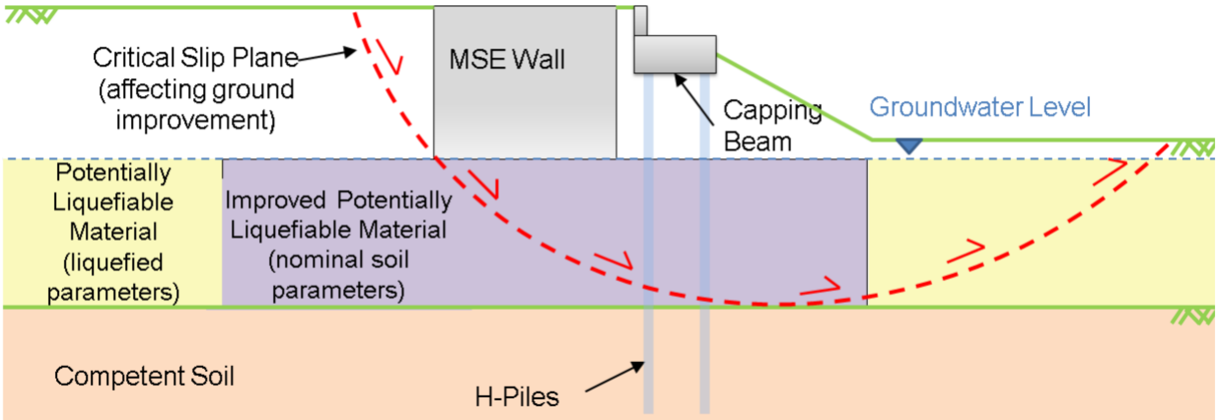

Figure 5. Ground Improvement of Liquefiable/ Compressible Materials

10 SKEWED BRIDGES

A number of bridges were highly skewed at up to 35 degrees because of the direction of existing rivers and local roads. This made it difficult to achieve the full design embedment in acute angle corners. Adequate embedment would not to be achieved if grids were placed perpendicular to the wall's face and so were skewed relative to precast side panel. Grids were locally offset where they overlapped existing grids to ensure frictional strength between grids and reinforced fill was achieved. Figure 6 shows the arrangement of grids in acute angle corners.

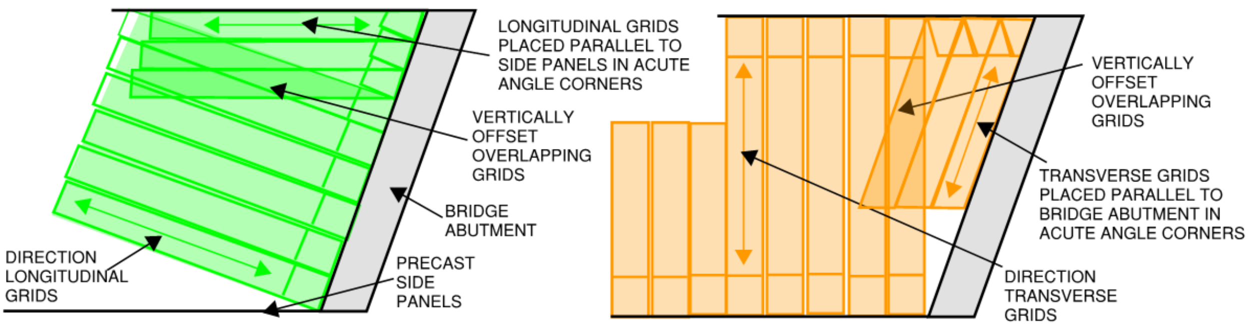

Figure 6. Arrangement of Longitudinal and Transverse Grid in Acute Angle Corners

11 CONCLUSIONS

The design of MSE walls on the M2PP expressway had six principle design challenges. The conclusions for overcoming these challenges are the following:

i. The project had high design peak ground accelerations up to 1g resulting in large seismic forces acting on walls. Displacement based design methods were used. This resulted in more economic MSE walls with less geogrids and ground improvement.

ii. The MSE walls are constructed from dune sand that is vulnerable to erosion during flood events. Therefore it was necessary to incorporate a number of design measures specifically to address the potential for erosion. These measures included the use of cement stabilised backfill immediately behind the wall face, carefully detailed drainage and comprehensive erosion protection.

iii. Many MSE walls overlay sandy soils that may liquefy and compressible soft soils. Ground improvement was adopted to control seismic lateral displacements and reduce differential settlement of walls.

iv. A secondary load path was required at some bridges if pile support could be lost because of large abutment movements during an MCE earthquake. This resulted in piles extending through the MSE reinforcement zone. Careful detailing of geogrids around piles and wall facing was required because of a reduction in reinforcement coverage ratio.

v. Some bridges were skewed because of the direction of existing rivers and local roads. This resulted in MSE walls with acute angle corners. Geogrids were skewed in these corners to achieve required pull out embedment and sliding resistance.

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