Investigation, Design & Construction of a Composite Road Embankment Raroa Road, Wellington

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Summary Cracking and subsidence was observed in the Raroa Road carriageway following a period of heavy rain. The site is located 500m from the main Wellington Fault. The topography of the site is steep. Remedial works have been designed and constructed to reinstate the 200m length of road to modern stability requirements including seismic design. Remedial works include a combination of a geogrid reinforced slope, fill buttress, drainage improvements, cantilever pile and tied-back pile retaining walls.

The project encountered difficult and variable surface and sub-surface conditions and unforeseen groundwater flows during construction. Flexibility and contingency provided in the design and ongoing design involvement throughout the construction enabled the project to be completed efficiently. Modifying and fine-tuning the design throughout the project and challenging standard design and construction techniques resulted in an innovative, efficient solution.

1 INTRODUCTION

Wellington City Council engaged Tonkin & Taylor Ltd (T&T) to investigate the causes of subsidence observed in the carriageway of Raroa Road, Wellington. T&T have undertaken:

- inspection and assessment of immediate risk
- monitoring to identify the magnitude and frequency of on-going deformation
- sub-surface investigation
- assessment of the risk of further movement
- review of remedial options and associated costs
- design of three specific retaining solutions
- supervision of construction

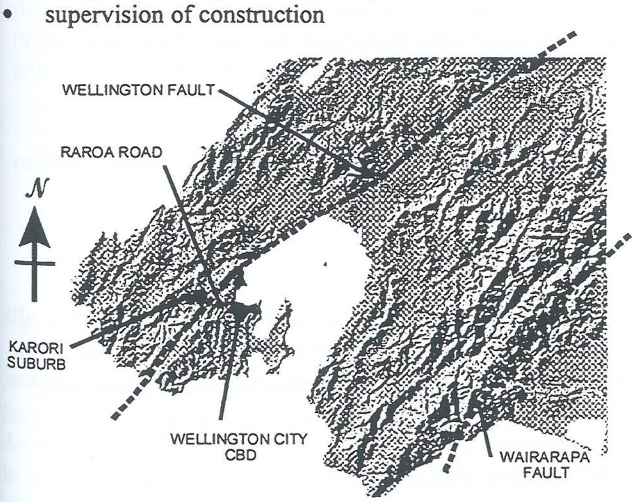

Figure 1: Location Plan

2 GEOLOGICAL AND GEOGRAPHICAL SETTING

New Zealand straddles the boundary of the Australian and Pacific tectonic plates. Caught in the wrenching action between the two plates sliding past each other, the Wellington region has fractured into blocks and long splinters of crust. The Wellington and Wairarapa Faults mark the edges of the larger blocks with many smaller faults crisscrossing the region. Minor earthquakes are common in the Wellington area. Since European settlement major earthquakes have been recorded in 1848, 1855 and 1942. The 1855 earthquake, the result of a rapture of the Wairarapa fault, is estimated to have been magnitude 8 (Richter) or higher (Begg Mazengarb) [2].

The active Wellington Fault bisects the City of Wellington. The signature earthquake for the Wellington fault is a magnitude 7.6 event with a predicted return period of 500 years. The last movement of the Wellington fault is dated at 340 to 480 years (Begg Mazengarb)[2]. This major fault lies 500m to the west of the Raroa Road site.

The site extends along a 200m section of roadway that ascends the side of the Aro Valley. The Aro valley follows the alignment of a minor dormant fault. The basement rock is interbedded greywacke sandstone and argillite with complex structure (Ruscoe) [4]. The steep side slopes of the underlying basement rock range from 30° to 60°. The floor of the valley is infilled with well-graded alluvial gravel. The side slopes are mantled with generally 0m to 1.0m thickness

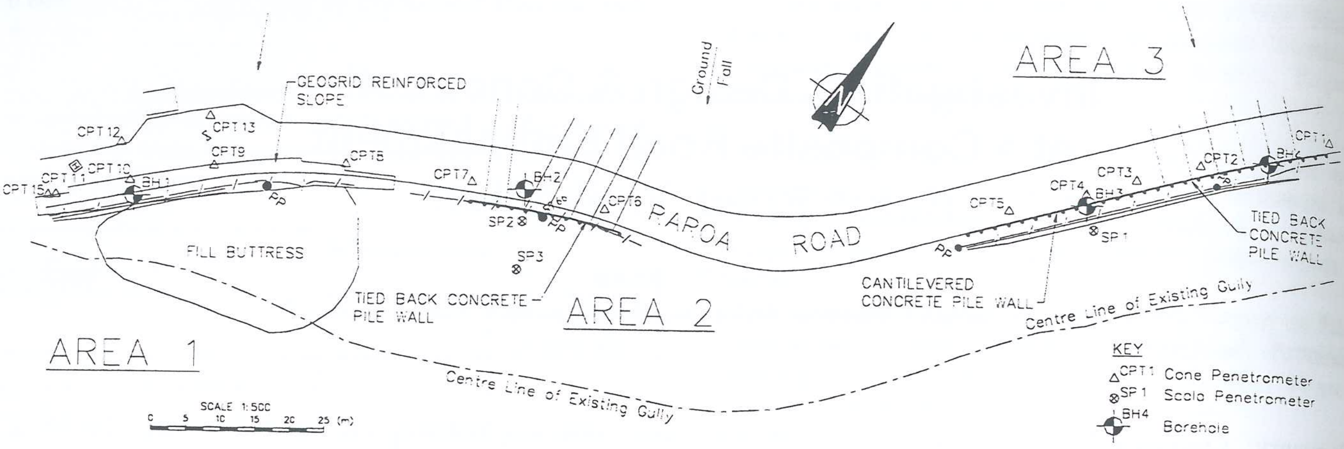

Fig. 2: Remedial Works Site Plan.

of colluvial and residual soils. Investigations indicate that the road embankment was formed early last century as a sidling cut to fill with local and imported greywacke rock placed as fill. The fill appears to have been placed directly on to the original slope. The original topsoil horizon is still in place and there is no evidence of benching or under-drainage. The fill is nominally compacted and generally consists of loose, fine to coarse silty gravel.

Artesian ground water was encountered in the basement rock with bored drains under the fill embankment encountering spring flows of up to 8 litres per minute.

The cut slopes above the road range from 35° to 70°. The fill embankment below the road stands at 35° to 45° and extends to the valley floor some 20 to 30m below the level of the road. At several locations the top of the fill embankment was retained with low concrete crib walls up to 3m in height.

Raroa Road is one of three arterial routes that link the city centre (on the down-thrusted side of the Wellington fault) and the suburb of Karori (on the up-thrusted side of the fault).

3 SUBSIDENCE

Deformation and subsidence of the carriageway was noticed following a large rainstorm event in October 1998. The deformation included significant cracking and subsidence in the downslope lane of the carriageway, resulting in arcuate depressions and cracking patterns with up to 50mm vertical offset.

Three discrete areas (Areas 1, 2 and 3 on Figure 2) along the road alignment were identified as having significantly deformed. Two of the three areas were supported by cribwalls. The cribwalls had noticeable deformation in both the horizontal and vertical alignments.

4 INVESTIGATION

4.1 Movement Monitoring

A monitoring regime was initiated to identify if further movement was occurring and if a major circular type failure beneath the carriageway was imminent. Only very minor movements were recorded following subsequent heavy rain and it was decided to keep the road open and maintain regular monitoring while a remedial investigation and design was carried out.

4.2 Sub-surface

Investigation of the site included four machine drill holes, 15 CPTs and 6 Scala penetrometer tests. Standpipe piezometers were installed in the boreholes to provide information on groundwater conditions.

The investigation confirmed that the road was constructed on silty, gravel fill, founding on a steeply inclined rock interface. At the western end of the road (Area 1), the depth of fill and colluvium underlying the carriageway was up to 15m. Typically 5m to 7m depth of fill underlies the remaining sections of road (Areas 2&3).

The existing cribwalls were constructed using 900mm concrete units connected with steel pins.

4.3 Stability Analysis

Site observations and stability analysis confirmed that all three sites were marginally stable under existing conditions. There was a high risk of further movement under a moderate seismic event or major rainstorm.

5 REMEDIAL SOLUTION

Findings of the investigation and stability review were discussed with Wellington City Council. Given the importance of the road as an arterial route, a low risk (high security) type solution was recommended. For the 200m length of road identified as having a moderate to high risk of instability, remedial designs were adopted.

5.1 Area 1

5.1.1 Design Philosophy

Properties (see Figure 3)

- 15m depth of variable fill and colluvium
- narrow infilled valley with original valley centre under the carriageway
- an existing cribwall 3.0m in height above a 35° fill batter slope
- assumed existing failure mechanism is a circular shear plane encompassing the existing cribwall.

The remedial solution adopted to provide adequate factors of safety for both static and seismic cases was:

- construct a flexible gravity type retaining structure to support the edge of the carriageway

- fill the valley to form a buttress to support the new retaining structure.

The significant depth of fill and colluvium (15m) and depth to a suitable bearing stratum ruled out piled or anchored wall solutions. A geogrid reinforced fill with a 68° wrap around face (see Figures 3 & 4) was constructed to support the carriageway. Using a deformation tolerant retaining system reduced the difficulties with variable founding properties and also reduced the design requirements for ultimate limit state seismic design.

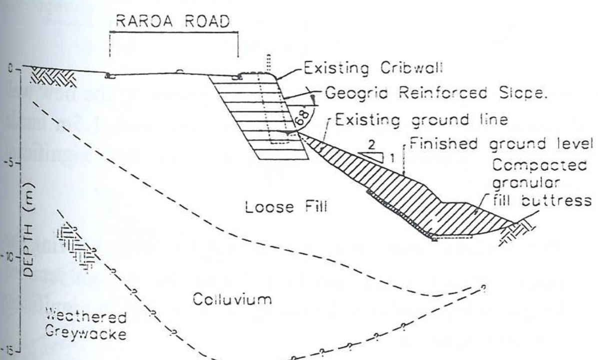

Fig. 3: Remedial Works Area 1. Schematic Cross Section

A major design consideration was to maintain at least one lane of traffic during construction as there were no alternate detour routes close to the site. In order to provide one lane in safe operating condition, the depth and width of the retaining wall excavation was limited. A relatively low retaining wall and large fill buttress was adopted to achieve this goal.

5.1.2 Seismic Design

Seismic design was undertaken using the following assumptions:

- The ultimate limit state horizontal ground acceleration was taken as 0.4g (approximately 1000 year return period)
- The serviceability limit state horizontal ground acceleration was taken as 0.2g (approximately 150 year return period)

The structure is designed to prevent yield in the serviceability limit state with a FOS greater than 1.1. The probable displacement of the wall in the ultimate limit state is estimated to be between 25mm to 100mm. Probable displacements have been calculated using Newmark [3] and Ambraseys and Menu [1] charts. Deformations of this magnitude are unlikely to result in structural failure of the reinforced slope or buttress however some cracking of the road surface would be expected. These predicted magnitudes of deformation are considered to be acceptable for the ultimate limit state design case.

5.1.3 Modifications During Design

Because construction was to be undertaken over winter, both the buttress and the geogrid-reinforced fill were constructed with compacted granular hardfill. The existing fill materials were not reused because of the high organic and clay contents.

The use of hardfill instead of clay fill or re-using existing fill cost an extra 5% of the original (summer) estimated contract value. However, the use of hardfill resulted in a higher strength fill, easy construction, lesser fill testing and monitoring requirements. Only two days were lost due to adverse weather conditions over the two months of construction. In hindsight, the correct decision was made and resulted in efficiencies in completing the contract.

The contract was let as a measure and value contract with design build elements. This enabled different geogrid suppliers to submit specific designs. Regan Brothers, Ltd, using Maccafferri New Zealand as geogrid supplier and designer won the contract. The geogrid used was "Fortrac 35/20-20", at 500mm vertical centres with a wrap-around face.

The design was modified during tender evaluation to minimise excavation widths in order to maintain one lane of traffic. For the critical loading case, greater geogrid embedment lengths are required at the top of the wall reducing in length with depth. It is typical practice (mainly to simplify details and specifications) to use a single geogrid embedment length for the full wall height. By challenging this typical practice and constructing a tapered wall with longer geogrid at the top shortening up at the base, the excavation width at the base of the excavation was

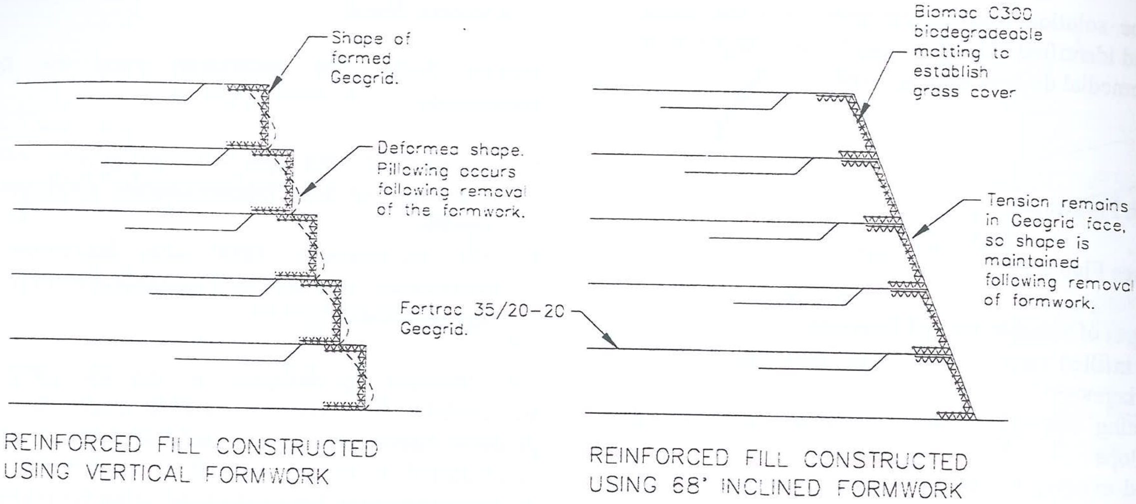

Fig. 4 Geogrid Reinforced Slope Face Detail

significantly reduced, allowing a flatter temporary batter and reducing the risk of instability of the temporary excavation.

5.1.4 Construction

Initially, the templates (formwork) used to form the face were 500mm high vertical sections with each successive layer setback 300mm to form an overall face angle of 68°. This practice was found to be unacceptable because the square edges of the geogrid provide minimal tension in the grid and the hardfill soon loosens up and the face bulges to form an unsightly pillow type appearance.

The templates were modified to provide a 68° face slope. Each successive layer was constructed immediately above the other. This was successful as tension in the front face is maintained and resulted in a superior finish. The template, as a consequence of the angled face was stiffer and much easier to handle and extract from each lift.

5.2 Areas 2 and 3

The two lower sections of road were retained with cantilevered pile retaining walls. The piles are bored, cast insitu reinforced concrete. Each pile is placed at 3D centres in order to provide soil arching between adjacent piles and negate the requirement to install rails or lagging.

The new retaining structures at both Areas 2 and 3 are rigid and not tolerant of deformation. Both structures are therefore designed to resist an ultimate limit state seismic load of 0.4g without yielding.

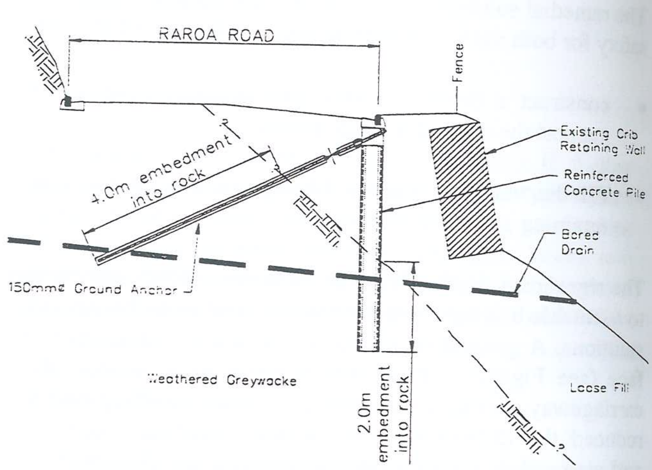

Fig. 5: Typical Section (Area 3).

5.2.1 Area 3

Where existing cribwalls were present (Area 3) the new wall is aligned under the existing kerb and channel, 1.5m inside the existing cribwall. This alignment has two significant benefits.

- The rock/fill interface dips at 40° to 50°. Moving the piles upslope significantly reduces the design retained height which reduces loadings and results in significant cost and time savings.
- Installing the piles upslope of the existing cribwall minimised the excavation required. Construction was rapid and access to both traffic lanes could be maintained for the majority of the works.

Locating the wall on the inside of the footpath leaves the pedestrian footpath unprotected by the new wall. The new wall will however significantly reduce the loadings applied to the existing cribwall, which may prolong the design life of the aging cribwall. When the existing cribwall deteriorates, pedestrian access can be maintained either by lowering the footpath alignment or cantilevering a suspended timber path from the new retaining wall piles.

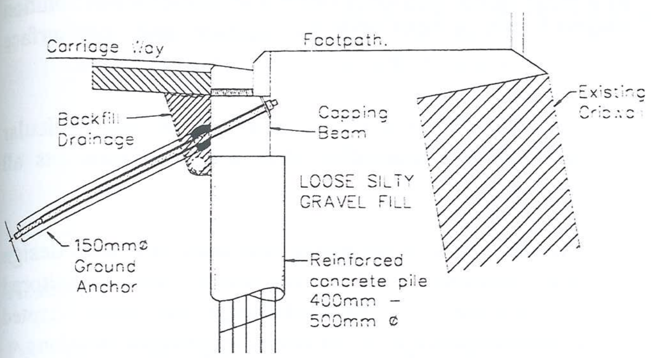

Fig. 6: Remedial Works Area 3, Typical Section

At Area 3, the design retained height of the wall varies between 2 and 5m depth. Where the designed retained heights of the walls are greater than 3.5m tieback anchors have been used to reduce the bending and overturning moments in the piles. The use of tieback anchors resulted in a cost saving to the principal of 30% over a simple cantilever type wall.

5.2.2 Area 2

In Area 2 there was no existing crib wall and the rock level (between 5m and 7m depth) was less steeply inclined than in Area 3. Minimal advantage was gained by moving the wall alignment upslope so Area 2 piles are aligned along the downslope edge of the footpath.

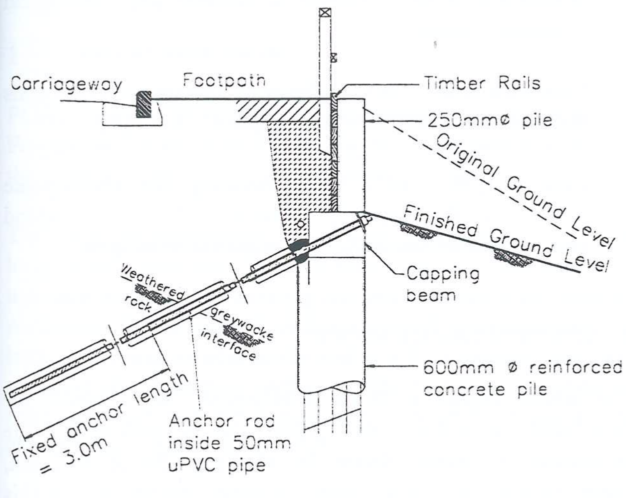

Fig. 7: Remedial Works Area 2

Tied-back reinforced concrete piles were used (see Figure 7) for Area 2. A tieback beam is set 1.2m below the footpath level with 600mm diameter concrete piles at 2m centres below the beam. Above the beam, the pile diameters were reduced to 250mm diameter mainly for aesthetics but this reduction in pile diameter also economises on materials where the bending moments are minimal.

The soil behind the 250mm diameter piles is supported with timber rails. The small wedge of fill in front of the timber rails was removed to facilitate anchor installation and also to relieve some loading from the top of the fill slope.

Downslope of the retaining wall there is potential for further instability in the unsupported fill slope. The Area 2 wall is designed to retain 5m depth. This depth of retention was established by estimating the worst probable long-term regression of the slope below. The regression profile was estimated by taking the deepest probable failure surface of the fill slope, and projecting a 26° line (tangent) from the base of the failure surface. This resulting regression line has a FOS under saturated conditions of 1.3 or greater. If such regression occurs, remedial lagging (sprayed concrete) will be required between the palisade wall piles.

5.2.3 Ground Anchors

Ground anchors have been designed to derive pull out resistance from the weathered greywacke rock. The anchors are RB 32 Reidbars set in a 150mm-diameter cement grout. The anchors with an ultimate tensile load of 430kN were designed assuming an ultimate rock-grout bond stress of 300kPa.

The anchors have a long non-bonded length so that a majority of the grout column will work in compression. Because the grout column is in compression tensile cracking of the grout is minimised and the elasticity of the anchor is controlled by the stiffness of the grout in compression. Anchor loads are therefore transferred more evenly to the founding rock at low strains minimising the risk of gradual unraveling of the rock/grout bond.

The anchors are in a relatively low corrosive environment however two to three levels of corrosion protection are provided for steel rods and components. Levels of protection are:

- all rods and components are galvanised
- the grout column is designed to be continuous to the surface
- the anchor tendon over the free length is coated in grease and enclosed in a uPVC tube
- exposed surfaces are coated in grease and wrapped in Denso tape.

A foam cushion was detailed between the top end of the grout column and the tieback beam so that when the anchors are stressed and tested, the grout/rock bond is stressed rather than putting the grout column/concrete beam joint into compression.

All anchors have been proof-loaded to 90% of yield and then locked off at design working stresses. The tieback wall analysis assumes rotation of the wall about the anchors. The anchors need to be pretensioned because the concrete piles have low elastic properties (stiff) compared to non-tensioned anchors. If the anchors were non-tensioned they may stretch taking up the load, this may result in the rigid concrete piles attracting higher proportions of the design load and possibly failing.

5.2.4 Construction

The construction works for the concrete pile walls (Areas 2 and 3) was let as a separate contract to the geogrid reinforced slope (Area 1). Construction proceeded generally as anticipated with the exception of the following:

- Minor variations in depth to rock between test locations were encountered. These variations were accommodated by varying pile details and introducing additional anchors for some sections of the Area 3 wall.
- Large quantities of groundwater were unexpectedly encountered in a crush zone in the centre of Area 3. One ground anchor intercepted artesian ground water within the greywacke. The water was flowing from the top of the anchor hole at approximately 2 to 5 litres per minute. After three attempts, the anchor holes (in this area) were successfully grouted with grout return to the surface.
- Two anchors failed under test loading. Both anchors were extracted and re-drilled. The anchors were unable to take load because two couplings joining sections of anchor rod failed. Both couplings had been modified by the drilling contractor to fit down the inside of a smaller drill casing. These modifications weakened the couplings.

5.2.5 Sub-surface Drainage

Due to the artesian groundwater encountered during anchor installation, inclined bored drains were drilled into the slope to relieve water pressures.

The drains were drilled from below the existing cribwall inclined upwards at 5° (see Figure 5). There was concern that the drains would transfer groundwater from the rock and into the more permeable gravelly fill, reducing slope stability below the new retaining wall.

To overcome this a permanent casing was drilled through the fill and socketed 1.0m into the rock. The hole and casing were then cement grouted. A smaller diameter inclined bored drain was advanced through the grout and into the rock, forming a sealed pipe through the fill material. Two bored drains were installed and each has a constant flow of about 8 litres per minute.

6 CONCLUSIONS

The client received an innovative and cost-efficient solution despite difficult and variable surface and sub-surface conditions. Elements that have contributed to this are:

- Different design solutions were used to suit particular ground conditions rather than a one solution fits all approach
- Flexibility and contingency was built into each design and specification. Soil conditions were monitored during construction and each design was easily adapted to accommodate local variations in ground conditions.
- The engineer throughout the project has provided design input. Each stage has been reviewed, fine-tuned and modified to suit construction methods used and the conditions encountered.
- "Standard" construction and design techniques for the geogrid-reinforced slope were challenged and reviewed for this application. Large benefits were achieved by providing small amendments to "standard" procedures.

7 REFERENCES

1. Ambraseys N.N. and Menu J.M., 1988, "Earthquake Induced Ground Displacements", Earthquake Engineering and Structural Dynamics, Vol. 16.
2. Begg J.G. and Mazengarb C., 1996, "Geology of the Wellington Area", Institute of Nuclear and Geological Sciences Limited
3. Newmark N.M., 1965, "Effects of Earthquakes on Dams and Embankments", Geotechnique 15, 139-160.
4. Ruscoe Q.W., 1974, "Microzoning for Earthquake Effects in Wellington", Bulletin 213, New Zealand Department of Scientific and Industrial Research.

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